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SECTION I

SUMMARY AND RECOMMENDATIONS

SUMMARY

This report is being written to serve as a General Sewer Plan (GSP) for the cities of Chehalis and Napavine and Lewis County Sewer District No. 1 (LCSD No.1). The City of Chehalis owns and operates a Wastewater Treatment Plant, which also accepts sewage from Napavine and LCSD No. 1. A regional sewer operating board, which is made up of an elected official from each entity, is responsible for oversight of the partnership.

The GSP has been prepared to address key issues that are a direct result of the Upper Chehalis River Dry Season Total Maximum Daily Load (TMDL) Study issued by the Washington Department of Ecology (DOE) in 1994. The TMDL Study determined that there is no loading capacity in the Centralia Reach of the Chehalis River during dry weather conditions for carbonaceous biochemical oxygen demand (CBOD) and ammonia. As a result, DOE issued the City of Chehalis, the City of Centralia and Darigold NPDES permits in 1996 that prohibited any discharge to the Centralia Reach from May 1 through October 31. The City joined Centralia and Darigold in a lawsuit to block implementation of the TMDL and to appeal the NPDES permits. Through mediation, a Consent Decree was negotiated among all of the parties, which forms the basis for a revised NPDES permit. The revised permit will still prohibit discharge to the Centralia Reach, but provides that the limitations be based on river flow (year-round) instead of simply encompassing the calendar period of May 1 through October 31.

The Chehalis NPDES permit also has discharge limits for three metals (zinc, silver and copper). The permit stipulates both interim and final limits for the three metals. The final limits, as written, will be very difficult to meet for some of the metals. The City needs to conduct further sampling and analysis of the metals' actual effects on water quality in order to refine the limits.

This GSP present alternatives to 1) modify or replace the existing treatment plant to comply with the conditions of the revised NPDES permit, 2) find an acceptable end use for the wastewater during dry weather conditions, and 3) increase solids handling capacity of the existing plant.

The GSP presents a work plan for further metals studies that must be completed to refine the final metals limits so that the City can meet them. A typical secondary treatment plant does not remove any metals. Advanced treatment plants designed to remove metals are very expensive to construct and operate.

The Plan also presents recommended improvements for the Chehalis collection system, including pipe replacement for inflow and infiltration (I/I) reduction and pump station upgrades. There are also upgrades recommended for some of Napavine's pump stations. Finally, the Plan identifies new sewer line extensions and new pump stations that will be required to provide sewer service throughout the Urban Growth Areas of Chehalis and Napavine, as well as to LCSD No. 1 by the year 2025.

GENERAL SEWER PLAN

Chehalis, Napavine and LCSD No. 1 provide sewer service to 8,671 people in Lewis County, which is located in Southwest Washington (See Figure IV-3). Chehalis has owned and operated a sewage collection system since the 1890s and sewage treatment facilities since 1949.

The plant has undergone five major upgrades. The first, in 1957, constructed tanks for additional primary and secondary clarification capacity, added a second trickling filter and a flow splitter box. The second major upgrade was in 1970, which added two aeration basins, sludge storage basin, sludge floatation thickener, another chlorine contract tank and expanded the lab/control building. The third major upgrade was in 1980, which constructed new headworks facilities, including grit removal and other modifications. The City completed the fourth major plant upgrade in 1988, which added a second secondary clarifier and increased peak hydraulic capacity of the raw sewage pumps and WWTP to 13.0 MGD. The capacity was provided to allow the

City time to implement a major I/I removal program. The latest major upgrade was in 1995, which increased the secondary treatment capacity of the plant to 7.5 MGD by adding a new primary spitter box and filter feed pumps.

In 1977, interceptors were constructed to accept wastewater from Napavine and LCSD No. 1. The City started nitrifying the trickling filter effluent in 1995 to reduce the amount of ammonia that is discharged to the Chehalis River during the summer.

The existing plant consists of a headworks that includes a screen and grit removal, two primary clarifiers, two trickling filters, two secondary clarifiers, chlorination and dechlorination equipment for disinfection and two aeration basins that serve for nitrification during the summer and are used for influent equalization storage during the winter. The solids treatment train consists of two anaerobic digesters (one is used for anaerobic digestion and one is used for sludge storage), four sludge storage basins and 20,000 square feet of covered drying beds for sludge dewatering. After treatment and disinfection, the wastewater is discharged into the Chehalis River at river mile 74.3.

Chehalis, Napavine and LCSD No. 1 have all experienced sustained growth during the last five years, especially Npavine, which experienced an annual growth rate of 6.2% for sewer connections between 1992 and 1997. For the same period, Chehalis' connections grew at an annual rate of 1.9%. This plan uses a planning horizon of 27 years to make sure that the planning effort is adequate for 20 years after the plant improvements are constructed. This plan uses average annual growth rates of 1.39% for Chehalis, 3.9% for Napavine and 2.0% for LCSD No. 1, which are based on the input of the jurisdiction's elected officials. Using these growth rates, the projected population in the service area served by sewers in the year 2025 is 14,588.

Chehalis, Napavine and LCSD No. 1 have separate collection systems. Each entity is responsible for maintenance of its own collection system, except that Chehalis maintains the sole LCSD No. 1 pump station. The Chehalis collection system consists of approximately 278,000 feet of mainline gravity sewers, ten pump stations and approximately 16,800 feet of force main (see Section IV). The Napavine collection system contains approximately 41,000 feet of mainline gravity sewers, five pump stations and approximately 9,500 feet of force main. The LCSD No. 1 collection system contains approximately 28,000 feet of mainline gravity sewers, 1,250 feet of force main.

Flow monitors were installed at key manholes in the collection system during January through April of 1998 to determine the amount of inflow and infiltration (I/I) that is presently entering the collection system. The results show that there are still four of the twelve sewage basins within the service area that would benefit from I/I rehabilitation projects. The four basins that would benefit from I/I projects are all in Chehalis. The results of the flow monitoring program are discussed in Section VI of this report.

It has been determined that Chehalis would need to install approximately 35,000 feet of 8-inch sewer main and one new pump station to serve the urban growth area identified in its comprehensive plan (see Section IV). Napavine will need to install approximately 4,000 feet of 8-inch sewer main and one pump station to serve the urban growth area identified in its comprehensive plan. LCSD No. 1 does not need to upgrade the collection system in order to serve anticipated growth within the district. To implement these sewer extensions, developers will presumably have to pay the initial high capital costs. It is anticipated that actual construction of the new pump stations and interceptors would occur over time as each area is developed.

The Cities of Chehalis and Napavine should formally adopt the 1998 *"Criteria for Sewage Works Design*" as published by the Washington State Department of Ecology (DOE), the 1998 "*Standard Specifications and Standard Plans for Road, Bridge and Municipal Construction*" as published by the Washington State Department of Transportation (WSDOT) and American Public Works Association (APWA) for all sewer work within the service area. After the above referenced criteria are adopted by the cities and this Plan is approved by DOE, then DOE approval of future sewer extensions is not required, provided the cities send DOE an assurance that each proposed sewer extension conforms with the approved plan and the adopted design and construction standards.

WASTEWATER TREATMENT PLANT

The existing plant is a 4.0 MGD trickling filter plant that uses extended aeration for nitrification during the summer months. The plant performs well, but has difficulty meeting the current NPDES permit limits for total suspended solids (TSS) and biochemical oxygen demand (BOD) percent removal requirements during wet weather conditions. Two factors contribute to this; one is that the influent is diluted by I/I in the system and the second is that the WWTP does not have adequate secondary clarifier capacity. The plant also has occasional violations of BOD and TSS concentration and mass (pounds per day) limitations, which appear to be caused by changing from the nitrification process back to wet weather operation of the trickling filter, as well as impacts from I/I. The plant has had one permit violation for zinc concentration and none for copper or silver. The plant site is located in the floodplain and floodway of the Chehalis River and experiences difficulty with flooding on a regular basis.

The plant must be upgraded or replaced in order to meet the conditions of the Consent Decree and new NPDES permit. The new permit limitations are stricter than the plant can meet without major modifications, including flood protection, new secondary clarifiers and equalization storage for high flows.

This report also looks at several alternatives for an end use of the treated effluent during dry weather conditions since continued discharge to the current location will not be allowed unless the river water quality is enhanced and the TMDL is modified. The end use options include moving the outfall location seven miles downstream below the confluence with the Skookumchuck River, ceasing dry weather discharge altogether and reusing the effluent and enhancing the water quality of the river to allow continued discharge at the current outfall location all year long. The option to join in a regional plant with Centralia is also considered in this report.

After consideration of numerous treatment plant options, this report narrowed the field to three treatment plant options that comply with the TMDL and the NPDES permit requirements. The first option is to modify the existing treatment plant so that it operates as an extended aeration plant during dry weather conditions and a complete mix activated sludge plant during wet weather conditions. The second option is to replace the existing plant with a new sequencing batch reactor (SBR) treatment plant at the existing site and the third option is to build a new SBR at a new site. Other types of treatment plants were also evaluated. The analysis for both treatment and end use options is presented in Section VII of this report.

This report also evaluates the solids process train that includes sludge conditioning, stabilization and ultimate utilization. Options are presented and evaluated for thickening, stabilization, dewatering and land application or trucking of unstabilized biosolids to the new Centralia plant for processing and utilization (see Section VII).

Since it is not practical to provide treatment for metals, this report focuses on providing further analytical data that will be used either to raise the final metals limits or get them removed from the NDPES permit altogether. The metals work plan is presented in Section III of this report.

RECOMMENDATIONS

It is recommended that Chehalis, Napavine and LCSD No. 1 review this "General Sewer Plan" and submit it to DOE for approval. The GSP presents recommendations for improvements to the collection systems. It also presents a recommended enduse and treatment plant process. After the GSP is approved, work will begin on the Facilities Plan for the preferred WWTP and end use option. The recommended location of the new WWTP will be presented in the Facilities Plan.

It is recommended that Chehalis and Napavine adopt the DOE Design Standards and the WSDOT/APWA Standard Specifications and Standard Plans as minimum standards for all sewer work completed within the entire service area.

COLLECTION SYSTEM

It is recommended that several modifications be made to the Chehalis and Napavine sewage collection systems. The following improvements should be made as soon as funds are available:

1. Upgrade the Riverside pump station to provide reliability.
2. Upgrade the Prindle Street pump station to provide reliability.
3. Upgrade alarms, telemetry and flow meters.

SEWAGE TREATMENT AND END USE FACILITIES

The TMDL and resulting NPDES permit which DOE issued in 1996 placed a considerable burden on the cities of Chehalis and Napavine, and Lewis County Sewer District No. 1 by forcing them to either move the discharge point of the WWTP downstream of the Skookumchuck River, or get out of the River altogether, during dry weather conditions. After careful evaluation of numerous enduse alternatives to address the new restrictions, the recommended alternative is to produce Class A reclaimed water that will be used for poplar tree irrigation in conjunction with groundwater recharge beneath the poplar trees. This option complies with the TMDL by using the effluent for a beneficial reuse and having no surface water discharge during dry weather conditions.

An in-depth analysis of different treatment plant processes was also performed. This was necessary to determine if the existing plant could be upgraded to meet the more stringent discharge requirements, or needed to be replaced with a new plant. It was determined that the existing plant needs to be replaced with a new Sequencing Batch Reactor (SBR) treatment plant.

The existing plant site is prone to flooding which led to the discussion of whether to build the new SBR at the existing site or at a new site. A thorough siting analysis and a recommended treatment plant site will be presented in the Facilities Plan.

SECTION II

PURPOSE AND APPROACH

PROJECT PURPOSE

The General Sewer Plan (GSP) has been prepared to conform with current regulations and guidelines in the Washington Administrative Code WAC 173-240 and the Revised Code of Washington RCW 90.48. The purpose of the plan is to guide the Cities of Chehalis and Napavine and Lewis County Sewer District No. 1 (LCSD No. 1) in providing sewer service to the study area through the year 2025.

It is intended that upon completion of this document, it will be reviewed and approved by the Cities of Chehalis and Napavine, the LCSD No. 1 and the Washington State Department of Ecology (DOE). Prior to implementing the recommended improvements to the wastewater treatment and collection facilities, a more detailed Facilities Plan must be prepared. The Facilities Plan also requires DOE review and approval before design and construction can begin.

APPROACH

A primary objective of sewer system planning is to minimize adverse impacts on the environment, and protect the health and safety of the community. An additional priority is to accomplish these goals in an economical and efficient manner. Minimum requirements are set forth by the United States Environmental Protection Agency (EPA) and DOE, which the Cities of Chehalis and Napavine and the LCSD No. 1 must abide by in the management of their sewer collection and treatment facilities. The primary instrument of these requirements is the National Pollution Discharge Elimination System (NPDES) permit and the Water Quality Standards (WAC 173-201A) for surface waters of the State of Washington. The NPDES permit and Water Quality Standards are discussed in detail in Section III of this report.

DOE recently completed a Total Maximum Daily Load (TMDL) Study which includes the Centralia Reach of the Chehalis River where the Chehalis WWTP currently discharges its treated effluent. The TMDL Study was specifically conducted to address low dissolved oxygen (DO) levels in the river during dry weather conditions. The low DO levels were found to be primarily attributable to natural conditions and BOD and ammonia from both point and non-point sources. The TMDL requires Chehalis to cease discharging to the Centralia Reach during critical low flow periods in the summer. After the TMDL Study was approved by EPA, DOE issued a new NPDES permit for the Chehalis WWTP in October 1996. The new permit set forth a schedule which prohibited any discharge of treated effluent to the Centralia Reach from May 1 to October 31. Chehalis and DOE entered into a Consent Decree that changed the calendar-based restriction to a river flow-based restriction. A new NPDES permit that implements the provisions of the Consent Decree was issued by DOE in 2000. This plan will evaluate alternatives to meet the interim and final effluent limitations in the NPDES permit.

The DOE also completed a Temperature TMDL Study for the upper Chehalis River where the Chehalis WWTP currently discharges. The temperature TMDL has not been finalized as of this writing, but may require additional restrictions on discharge to the Centralia Reach.

The approach taken in preparation of this plan is to:

- Evaluate components of the sewer system with respect to compliance with the Consent Decree and applicable sections of the NPDES permit.
- Describe existing and future effluent limitations and other water quality management goals that must be met.
- Describe additional metals analysis that must be performed to obtain final metals limits that can be met without specific treatment trains for metals removal.
- Define the present and future planning areas and describe the physical and environmental conditions within the planning areas.
- Prepare a sewer system inventory and identify capacities of the wastewater collection and treatment systems.
- Prepare population projections and develop wasteload projections for flow, Biochemical Oxygen Demand (BOD), Suspended Solids (SS) and Ammonia (NH_3).

- Evaluate alternatives and estimate costs for recommended wastewater treatment plant improvements that will meet the needs of Chehalis, Napavine and LCSD No. 1 and satisfy all regulatory requirements.
- Identify a program to remove additional Infiltration and Inflow (I/I) from the sewer collection system.
- Evaluate financial options for funding recommended improvements.
- Develop a schedule (consistent with the Consent Decree) for implementing the recommended improvements.
- Assist Chehalis in preparation of a SEPA Checklist for the project.

This GSP utilizes information obtained from Chehalis including WWTP records and Daily Monitoring Reports (DMRs) from April 1995 to April 1998, as-built drawings, maps and previous planning and design related documents. Information provided by the Cities of Chehalis and Napavine and the LCSD No. 1 personnel concerning various system characteristics has been considered and included in this plan.

Many of the recommendations in this GSP are based on written interpretation by DOE staff and management regarding DOE policy, regulations, guidance and TMDL implementation. In several cases, DOE cannot provide specific interpretation without submittal and review of this GSP. For those issues where additional input is required, conservative assumptions are made based on general guidance by DOE and data contained in this GSP.

SECTION III

EFFLUENT LIMITATIONS AND OTHER WATER QUALITY MANAGEMENT GOALS

BACKGROUND

The Department of Ecology (DOE) uses two approaches to control the discharge of pollutants to surface waters. The first approach, which has been predominant in the past, is the technology-based approach. The technology-based approach requires that pollution is prevented with "all known available and reasonable methods…" (RCW 90.48.010). All known, available and reasonable or guidelines, or on a case-by-case basis through the submittal and review of an engineering report.

The second approach is the water quality-based approach. Section 303(d) of the Clean Water Act requires that water bodies not meeting water quality standards after application of technology-based pollution controls must be placed on the State's Section 303(d) list. All water bodies on the Section 303(d) list are required to undergo an analysis for the maximum pollutant loading capacity (LC) of the water body that will allow the water quality standards to be met.

Once the LC is established, the total loading available is allocated to known or suspected pollution sources. Load Allocations (LAs) are initially set for non-point sources, background/natural sources and scientific uncertainty. Remaining LC is apportioned through Waste Load Allocations (WLAs) for permitted point sources. Both LAs and WLAs may be set aside as a reserve for future growth if LC is available. The sum of all LAs and WLAs that will allow attainment of water quality standards is termed the "Total Maximum Daily Load" (TMDL).

DOE completed a TMDL Study entitled "Upper Chehalis River Dry Season Total Maximum Daily Load Study" in July of 1994 (referred to herein as the "TMDL Study"). The Chehalis

Regional WWTP is subject to unique and complex constraints and limitations based on the TMDL Study and its recommendations for implementation. This section will address the constraints and limitations placed on the Chehalis WWTP discharge by the TMDL Study, water quality standards, and other related constraints within state and federal regulations.

The upper Chehalis River basin temperature TMDL was issued in revised draft form in November 2000. The Temperature TMDL may place additional restrictions on the Chehalis WWTP discharge during critical and non-critical conditions. The temperature TMDL will be discussed in greater detail in the forthcoming Facilities Plan once it is finalized.

WATER QUALITY CLASSIFICATIONS

The Chehalis Regional WWTP discharge is located in the Upper Chehalis River Basin at River Mile (RM) 74.3. The WWTP primarily influences a segment of the river, known as "Centralia Reach", which is located between Scamman Creek at RM 65.8 and the Newaukum River at RM 75.2. Water Quality Standards for the State of Washington (Chapter 173-201A WAC) have established the status of the Centralia Reach as Class A Surface Water. Water in this classification must meet or exceed the requirements of WAC 173-201A-030(2) for all appropriate uses such as domestic, industrial, and agricultural water supply; stock watering; general recreation; commerce and navigation; wildlife habitat; and fish migration, reproduction, rearing and harvesting. Special conditions within WAC 173-201A-130(9) reduce the minimum dissolved oxygen (DO) standard within the Centralia Reach from 8.0 mg/l to 5.0 mg/l during the period of June 1 to September 15.

Municipalities are permitted to discharge sewage effluent into Class A water provided the effluent meets specified effluent limitations and water quality standards. However, if natural conditions in a water body are less than the water quality standards, then the antidegradation requirements of WAC 173-201A-070 apply. Section 2 of this statute states that "Whenever the natural conditions of said waters are of a lower quality than the criteria assigned, the natural conditions shall constitute the water quality criteria." Since water quality in the Upper Chehalis River has not met criteria in past low flow conditions during dry season periods, the antidegradation requirements are applicable under certain conditions addressed in the TMDL Study.

TOTAL MAXIMUM DAILY LOAD (TMDL) STUDY

The TMDL Study was conducted by DOE to evaluate dry season water quality and pollutant loading capacity in the Upper Chehalis River. Water quality and flow data were collected during the dry seasons of 1991 and 1992. DO readings below water quality criteria were widespread in the main stem and the tributaries of the Chehalis River. The water quality data also showed thermal stratification in the Centralia Reach during the summer months. Temperatures in the main stem and the tributaries of the Chehalis River often exceeded the maximum water quality criteria of 18°C during the TMDL Study period. Degraded water quality for fecal coliform bacteria criteria was also found in the main stem and some tributaries.

The TMDL Study evaluated the LC for carbonaceous biochemical oxygen demand (CBOD) and ammonia nitrogen (NH_3-N) that would allow DO to meet water quality criteria where possible, and allow no significant degradation of DO where background conditions would not allow the criteria to be met.

The TMDL Study utilized the WASP5 water quality computer model to evaluate the LC. Conservative assumptions were made for modeling critical conditions to incorporate modeling uncertainty. Maximum temperatures, critical low flows, and conservative reductions in non-point loading and sediment oxygen demand (SOD) were used to reduce the possibility of underestimating the impact of pollutants on the Chehalis River.

The DOE modeling determined that DO for existing critical conditions would fall below water quality criteria over much of the main stem study area in both mid-summer (DO criteria=5 mg/l minimum) and early fall (DO criteria=8 mg/l minimum). When pollutant loading in the model was reduced to assumed background levels, the model predicted DO in the Centralia Reach above the minimum water quality of 5 mg/l during the summer, but below the minimum criteria of 8 mg/l criteria during early fall. A phased TMDL was recommended in the TMDL Study for

CBOD and NH_3-N during the period May 1 to October 31.

The phased TMDL approach is advised when significant uncertainty may produce future changes in the TMDL. Some of the uncertainty in the Chehalis River TMDL comes from the following (as adapted from the TMDL Study):

- Background conditions for the model assume that implementation of Best Management Practices (BMPs) for non-point sources will result in minimum water quality standards attainment on all tributaries. Site-specific information is very limited for non-point problems, as are the potential methods that will reduce pollutant loading. The long-term effectiveness of BMP implementation should be evaluated to better estimate the potential for background water quality improvement and associated benefits.

- The SOD in the Chehalis River, especially in the Centralia Reach, was estimated for calibration of the DOE model. The actual relationships of phytoplankton and nutrients to SOD from settled biomass is unknown, as is the relationship of tributary non-point sources. Improvements in non-point source loading to the water column may also reduce SOD levels, but this is not certain. The same can be said for aerating or enhancing circulation of stratified components. Future studies should be conducted by DOE to quantify SOD, evaluate its sources and possibly conduct pilot studies to determine long-term mitigation measures.

- High river temperatures reduce DO saturation limits, as well as the capacity of the main stem and tributaries to assimilate ammonia and CBOD. To demonstrate this, DO levels improved when the model was run with temperatures set to a maximum of 18°C. Therefore, measures to improve temperature by lowering it in the main stem and tributaries may allow for greater loading capacity for ammonia and CBOD.

One of the three complex studies, suggested by the TMDL Study, for consideration in future years includes a study of sediments in the Chehalis River, especially in the Centralia Reach to make *in situ* SOD measurements and measurements of benthic (river bottom) nutrient chemistry. DOE suggests that this study should be conducted after BMPs have been widely implemented and as the phased TMDL is being reassessed. This is an excellent recommendation to minimize the public resources that will be expended to reduce the relatively small amount of point source loading to the river. The potential for effluent trading between point and non-point sources under this type of TMDL implementation will satisfy EPA priorities and may significantly reduce the costs to sewer customers. However, DOE has indicated that implementation of this strategy is unlikely. The primary influence is due to current DOE priorities to address point source compliance above non-point source compliance.

The following are two of the more significant conclusions and recommendations as published by DOE in the TMDL Study:

> *"From Mellen Street upstream to the Newaukum River no capacity exists for discharge of ammonia or BOD without a significant degradation (0.2 mg/l or more) of DO below the water quality criteria. Therefore, no point or non-point source loading above background can be allocated in the Centralia Reach."*

> *"This report* (the TMDL Study) *recommends that no WLAs be provided from the Mellen Street bridge upstream. Clearly, removing the existing Chehalis and Darigold discharges from the river in the May through October dry season will have major impacts on these dischargers. Implementation of this recommendation will need to be carefully thought-out process that will include planning, engineering, obtaining inter-government agreements, and applying for and receiving grants and loans. The concerns of other programs and jurisdictions will have to be addressed (e.g., Water Resources Program concerns over changes in river flows due to changes in permitted discharge locations). The best solution must be found that addresses the problems identified in this study, anticipates future problems, and doesn't create any new problems."*

Two alternatives for WLAs and LAs were proposed in the TMDL Study. Alternative 1 is based on Chehalis and Darigold finding an alternative disposal method for their wastewater during the dry season, such as industrial or agricultural reuse. Alternative 2 is based on Centralia, Chehalis and Darigold siting a combined outfall below the Skookumchuck River. LAs for non-point sources above the Skookumchuck River are limited to natural background levels under both alternatives.

An evaluation of model results based on additional flow conditions requested by the City is included in Appendix A. The TMDL Study model and recommendations were evaluated in a February 8, 1996 letter report by Cosmopolitan Engineering Group which is included in Appendix A. DOE did not review or comment on the letter report as a condition of the Consent Decree negotiations. There are no DOE conditions regarding the use of the Cosmopolitan Engineering Group report in the evaluation of alternatives in this report. For the sake of ensuring that local resources provide for the utmost benefit to the local environment and are applied cost effectively, the results of the letter report are considered in the alternative evaluation in Section VII of this report. It must be noted that alternatives based on the letter report findings are subject to DOE and EPA approval and modification of the Consent Decree and TMDL.

LOCALLY SPONSORED TMDL WORKSHOPS

Representatives of the City of Chehalis, City of Centralia, Darigold, various Lewis County industries and other volunteers met in September 1994 to evaluate the potential environmental and economic impact of the TMDL Study, evaluate the technical and scientific merits of the alternatives, develop first order cost estimates for the most viable alternatives, receive regulatory and funding agency input of the alternatives and maximize public input into the process.

To accomplish the work, a series of three public workshops were conducted beginning in the spring of 1995. The first workshop was specifically designed to provide public education on the issue and to "brainstorm" all potential alternatives. In the second workshop, a panel of scientific and engineering experts evaluated the merits of each of the 36 alternatives identified in the first workshop and reduced the list to nine of the most viable alternatives. Gibbs & Olson, along with CH2M Hill as a subconsultant, prepared preliminary design data and cost estimates for the remaining nine alternatives. A third workshop was held to receive regulatory and funding agency input on the nine alternatives. A final report on the workshop was prepared and presented to each of the local agencies, as well as DOE and EPA. The most feasible alternatives that were identified through this process are shown in Table III-1.

TABLE III-1
CENTRALIA REACH TMDL WORKSHOP NUMBER TWO
ALTERNATIVE EVALUATION SUMMARY TABLE

Alternative Group and Number	Alternative Name	Total Estimated Cost or Cost Range
GROUP I	**TREATMENT AND DISCHARGE**	
Alternative IA-1	Discharge to Rapid Infiltration Basins from May through October	Not Implementable*
Alternative IA-2	Lagoon Storage for Chehalis & Darigold with Centralia Discharging below Skookumchuck River	$68,700,000
Alternative IB	Centralia, Chehalis and Darigold Downstream of Skookumchuck River, via Pump Station & Force Mains	$22,130,000
Alternative IC-1	Improve Existing Wastewater Treatment Plants to Provide Nutrient Removal	$15,210,000 - $63,200,000
Alternative IC-2	Add Constructed Wetlands to Existing Chehalis & Centralia WWTP's to Provide Nutrient Removal	$55,548,160
Alternative ID	New Regional WWTP	$105,100,000
GROUP II	**TREATMENT AND REUSE**	

Alternative IIA	Application of Class D Water on Non-Food Crops at Agronomic Water Irrigation Rates	$85,530,000
Alternative IIB	Reuse of Class A Treated Effluent	$50,100,000
Alternative IIC	Application of Class D Water on Non-Food Crops at Agronomic Nitrogen Application Rates	$72,900,000
GROUP III	**IN-RIVER MEASUREMENTS FOR IMPROVING WATER QUALITY**	
Alternative IIIA	Artificial Re-aeration in the River (w/o WWTP Upgrade)	$3,850,000

* Based on land needs for all three discharges.
Note: Costs shown are for the combined flow of Chehalis, Centralia and Darigold in 1995 dollars.

Since the workshops were conducted, additional information on several issues such as flow-based discharge, amendments to reuse guidance and new river enhancement guidance have changed the basic premise of several of these alternatives. Many of the alternatives shown in Table III-1 are re-evaluated in Section VII of this report.

CONSENT DECREE

Final NPDES permits based on the TMDL Study were issued in October 1996. Among other things, the permits addressed maximum daily flow, BOD, TSS and ammonia discharges for the Cities of Chehalis and Centralia and Darigold. The three dischargers felt the effluent limits in the NPDES permits developed by DOE were unwarranted and the result of a flawed process. They therefore filed lawsuits to block implementation of the TMDL limits and appealed the NPDES permits to the Pollution Control Hearing Board.

As a result of the lawsuits, the Cities of Chehalis, Centralia Darigold and DOE developed and entered into a collaborative negotiation process to resolve the issues associated with the TMDL and NPDES limits. The collaborative process was premised on the concept that alternative limits based on river flows (rather than calendar-based periods) would be more protective of water quality and reduce the costs to the dischargers. Initially, this required water quality analyses to determine the assimilative capacity of the Chehalis River under various flow conditions and development of relationships between flow conditions in the Chehalis River and the municipal wastewater treatment plants. Relationships between river flows and climatic conditions were also needed to assess the viability of land application. A strong relationship was found between Chehalis River flows and water quality impacts from domestic wastewater treatment plant flows.

At low river flows, it was found that higher levels of treatment are needed. Fortunately, at low river flows, wastewater treatment plant flows are also low, and high levels of treatment can be provided with lower cost. At high river flow, the need for treatment is reduced, wastewater treatment plant flows are much greater and reduction in treatment requirements helps reduce the cost. Therefore, the flow-based limits have been shown to provide superior protection of river quality at a lower cost. The collaborative negotiation process resulted in the following benefits:

- Protection of water quality beyond the TMDL by changing from calendar-based to flow-based limits.
- Reduces the cost for TMDL alternative implementation.
- Greatly reduces legal costs by eliminating the need for trials.
- Development of a better relationship with DOE.

The process resulted in a Consent Decree between the plaintiffs (Chehalis, Centralia and Darigold) and the defendant (DOE). The Consent Decree is included in Appendix A and establishes interim discharge limitations, final discharge limitations, implementation timelines, controlled user rate impacts and most significantly, prohibits discharges to the Centralia Reach during low flow periods after eight years from the date the Consent Decree is entered into the court (January 14, 2000). A two-year extension to this deadline may be allowed based on funding availability and City sewer rate levels. Other key provisions of the Consent Decree include:

- DOE shall seek EPA approval of TMDL modifications.
- The City of Chehalis shall aggressively seek funding and DOE shall support the City in any funding efforts.
- Force majeure events such as acts of God, war, court orders, inability to attain permit approvals or authorizations may allow extension of compliance schedules to the extent of the delay caused by the force majeure event.
- The plaintiffs must dismiss their lawsuit and their appeals of the NPDES permits.

The effective date of the Consent Decree is the date which it is entered by the court, which was

January 14, 2000. The Consent Decree may only be modified upon written consent of all parties or through a specific dispute resolution process. The Consent Decree shall be terminated upon full implementation by all parties. The final Consent Decree document is part of the revised NPDES permit. DOE revised the TMDL to change the discharge restriction period from calendar based to flow based, per the Consent Decree. The revised TMDL was submitted and approved by EPA in March 2000.

The following is a summary of the interim water quality and technology-based effluent limitations specified in the Consent Decree and in the NPDES permit.

INTERIM EFFLUENT LIMITATIONS

The interim effluent limitations that allow the City of Chehalis to continue discharging all year long at the present outfall for the next eight to ten years are contained in the June 2000 NPDES permit (See Appendix A). This permit expired in June 30, 2000. The City is currently working with DOE on the renewed permit, which is expected to be reissued some time in 2001. Interim effluent limitations in the permit are for calendar-based dry weather (May 1 – October 31) and wet weather (November 1 – April 30) conditions. The following is a summary of the interim effluent limitations.

TABLE III-2
INTERIM EFFLUENT LIMITATIONS[a] (May 1 – October 31)

Parameters	Monthly Average[a]	Weekly Average[a]
BOD_5[b]	20 mg/l, 334 lbs/day	30 mg/l, 500 lbs/day
TSS[c]	25 mg/l, 417 lbs/day	37.5 mg/l, 626 lbs/day
Fecal Coliform Bacteria	200/100 mL	400/100 mL
pH	shall not be outside the range of 6.0 to 9.0	
Parameters	**Monthly Average**	**Daily Maximum**
Total Chlorine Residual	0.021 mg/l	0.023 mg/l
Ammonia (NH_3-N)	18.6 mg/l	36.8 mg/l
INTERIM EFFLUENT LIMITATIONS[a] (November 1 – April 30)		
Parameters	**Monthly Average**	**Weekly Average**
BOD_5[d]	30 mg/l, 1,000 lbs/day	45 mg/l, 1,500 lbs/day
TSS[e]	30 mg/l, 1,000 lbs/day	45 mg/l, 1,500 lbs/day
Fecal Coliform Bacteria	200/100 mL	400/100 mL
pH	shall not be outside the range of 6.0 to 9.0	
Parameters	**Monthly Average**	**Daily Maximum**
Total Chlorine Residual	0.023 mg/l	0.026 mg/l
Ammonia (NH_3-N)	12.9 mg/l	31.6 mg/l

[a] The average monthly and weekly effluent limitations are based on the arithmetic mean of the samples taken with the exception of fecal coliform, which is based on the geometric mean.
[b] The average monthly effluent concentration for BOD_5 shall not exceed 20 mg/l or 15% of the respective monthly average influent concentrations, whichever is more stringent.
[c] The average monthly effluent concentration for Total Suspended Solids shall not exceed 25 mg/l or 15% of the respective monthly average influent concentrations, whichever is more stringent.
[d] The average monthly effluent concentration for BOD_5 shall not exceed 30 mg/l or 25% of the respective monthly average influent concentrations, whichever is more stringent.
[e] The average monthly effluent concentration for TSS shall not exceed 30 mg/l or 35% of the respective monthly average influent concentrations, whichever is more stringent.

INTERIM METALS EFFLUENT LIMITATIONS (All Year)		
Parameters	**Yearly Average**	**Daily Maximum**
Copper	N/A	53.5 μg/l
Silver	13.5 μg/l	28.2 μg/l
Zinc	N/A	119.6 μg/l

μg/l = micrograms per liter

FINAL EFFLUENT LIMITATIONS

Final effluent limitations are conditioned on flow-based criteria specified in the Consent Decree. The flow-based criteria specifies wet weather and dry weather flow conditions as a function of river flow.

In general, flow in the Centralia Reach shall be determined by the USGS Grand Mound gage using the following conversion equation:

$y = 0.7396\,x - 28.28$

y is the flow, in cfs, in the Centralia Reach

x is the flow of the Chehalis River, in cfs, as measured at the Grand Mound gage.

cfs means cubic feet per second

"Dry weather limits" apply on the next day after the 7-day moving average flow goes below 1,000 cfs and on all subsequent days until the wet weather limits apply. Within the dry weather limits, there is a second trigger level at 200 cfs, which causes more stringent effluent limits to go into effect for some pollutants. For the 200 cfs trigger level, daily flows of the river shall be used, and direct measurements (i.e., no conversion) from the Grand Mound gage at 300 cfs shall be deemed equivalent to the 200 cfs level in the Centralia Reach.

Dry weather limits for ammonia go into effect 14 days after the 7-day moving average flow is less than 1,000 cfs, provided that the 14-day phase-in period shall be triggered no earlier than March 1 of each year (hence, March 15 is the earliest date that the dry weather limits for ammonia will apply). Additional modeling should be conducted to evaluate whether or not the March 15 date should apply for BOD and ammonia as well. However, to implement the final limits this way, the Consent Decree will have to be modified.

Final effluent limitations will take effect (according to the Consent Decree schedule) eight years from the date of the Consent Decree, or ten years if adequate funding is not available. The final effluent limitations do not allow any discharge at the present outfall location during dry weather (low flow) conditions unless river enhancement is implemented (see Section VII). Without modifications to the Consent Decree, dry weather discharge must be located downstream of the Skookumchuck River confluence. The following are the dry weather effluent limitations for the WWTP at a potential future outfall located downstream of the Skookumchuck River.

TABLE III-3
FINAL EFFLUENT LIMITATIONS (Dry Weather Flow-based)

Parameters	Monthly Average	Daily Maximum
BOD_5 (Flow<200cfs)	20 mg/l, 417 lbs/day	30 mg/l, 626 lbs/day
BOD_5 (Flow>200 & 1000 cfs)	20 mg/l, 500 lbs/day	30 mg/l, 751 lbs/day
TSS (Flow<200cfs)	20 mg/l, 417 lbs/day	30 mg/l, 626 lbs/day
TSS (Flow>200 & <1000 cfs)	20 mg/l, 500 lbs/day	30 mg/l, 751 lbs/day
Ammonia (Flow<200cfs)	-	4 mg/l, 83 lbs/day
Ammonia (Flow>200 & 1000 cfs)	-	4 mg/l, 100 lbs/day
Total Residual Chlorine	0.021 mg/l	0.023 mg/l
Parameters	**Monthly Average**	**Weekly Average**
Fecal Coliform Bacteria	200/100 mL	400/100 mL
pH	shall not be outside the range of 6.0 to 9.0	

Parameter	**Daily Maximum**	
Ammonia (Flow < 200 cfs)	4 mg/l, 83 lbs/day	March 15-November 30
Ammonia (Flow > 200 & 1000 cfs)	4 mg/l, 100 lbs/day	March 15-November 30
Ammonia	15 mg/l, 375 lbs/day	December 1- March 14

Plant Flow, Daily Maximum

When Flow < 200 cfs	2.5 MGD
When Flow > 200 cfs & <1000	3.0 MGD

Note: The monthly average effluent concentration of BOD_5 and TSS shall not exceed 20 mg/l or 15% of the respective monthly average influent concentrations, whichever is more stringent.

TABLE III-4
FINAL METALS EFFLUENT LIMITATIONS (Dry Weather Flow-based)

Parameters	Monthly Average	Daily Maximum
Copper	9.69 µg/l	10.63 µg/l
Silver	1.27 µg/l	1.39 µg/l
Zinc	69.6 µg/l	76.3 µg/l

Final effluent limitations for wet weather conditions apply to the current outfall location in eight to ten years. The final metal discharge limits may be modified, or eliminated, in the future pending the findings of further metal studies, which include a multi-faceted Comprehensive WER Study. Until this work is completed, the interim effluent limitations will be in effect. The following is a summary of final wet weather limits:

TABLE III-5
FINAL EFFLUENT LIMITATIONS (Wet Weather Flow-based)

Parameters	Monthly Average	Daily Maximum
BOD_5	30 mg/l, 732 lbs/day	45 mg/l, 2,330 lbs/day
TSS	30 mg/l, 768 lbs/day	45 mg/l, 2,330 lbs/day
Ammonia	-	15 mg/l, 644 lbs/day
Total Residual Chlorine	0.023 mg/l	0.026 mg/l
Parameters	**Monthly Average**	**Weekly Average**
Fecal Coliform Bacteria	200/100 mL	400/100 mL
pH	shall not be outside the range of 6.0 to 9.0	

Plant Flow, Daily Maximum
13.0 MGD

Note: The monthly average effluent concentration of BOD_5 and TSS shall not exceed 30 mg/l or 15% of the respective monthly average influent concentrations, whichever is more stringent. The 15% TSS limit may be lowered if the City can document several conditions in WAC 173-221-050(4)(a)(i).

TABLE III-6
FINAL METALS EFFLUENT LIMITATIONS (Wet Weather Flow-based)

Parameters	Monthly Average	Daily Maximum
Copper	10.9 µg/l	12.0 µg/l
Silver	1.29 µg/l	1.41 µg/l
Zinc	78.3 µg/l	85.9 µg/l

The Consent Decree allows the City to seek relief from the 85% TSS removal requirement based on WAC 173-221-050. I/I removal projects may also be postponed until the City has funded the new treatment plant, as long as the sewer rates are above the hardship level. DOE has indicated that they are willing to extend the interim 65% removal limit for TSS during wet weather conditions until such time as the City has removed enough I/I to meet the 85% TSS removal limit. Section VI of this report presents a schedule for I/I removal and the year in which the 85% TSS removal limit is expected to be met. This issue needs to be discussed with DOE in greater detail when the NPDES Permit for the final limits is issued.

WATER QUALITY ANALYSIS

This report does not present a water quality analysis for parameters affecting dissolved oxygen because it has already been completed by DOE through preparation of the TMDL Study, as well as subsequent water quality modeling referenced in this section. However, a water quality analysis for chlorine and metals is presented below.

WATER QUALITY-BASED LIMITATIONS

The Washington State Department of Ecology (DOE) has designated beneficial uses of Washington's surface waters, and adopted and defined surface Water Quality Standards (WQS) in accordance with EPA's recommendations. The Washington State Water Quality Standards (Chapter 173-201A WAC) are the state regulations to protect the beneficial uses of the waters for Washington State.

The WQS allow DOE to authorize mixing/dilution zones around a point of discharge to establish water quality-based effluent limits. Both "acute" and "chronic" mixing zones may be authorized at the point of discharge for pollutants that can have a toxic effect on the aquatic environment. The concentration of pollutants at the edge of these mixing zones may not exceed the numerical criteria or limits for that type of zone. Mixing zones can only be authorized for discharges that are treated using "all known, available, and reasonable methods of prevention, control, and treatment" (AKART).

Modifications to the metal limit criteria for site-specific conditions have been defined in

Chapter 173-201A WAC as follows:

> *These ambient criteria [are based upon] the dissolved fraction…of the metal…The metals criteria may not be used to calculate total recoverable effluent limits unless the seasonal partitioning of the dissolved to total metals in the ambient water are known. When this information is absent, these metals criteria shall be applied as total recoverable values, determined by back-calculation, using the conversion factors incorporated in the criterion equations. Metals criteria may be adjusted on a site-specific basis when data are made available to the department clearly demonstrating the effective use of the water effects ratio approach established by USEPA, as generally guided by the procedures in USEPA* ***Water Quality Standards Handbook****, December 1983, as supplemented or replaced. Information which is used to develop effluent limits based on applying metals partitioning studies or the water effects ratio approach shall be identified in the permit fact sheet…and shall be made available for the public comment period required…,as appropriate.*

If any Water Quality Criteria parameter is exceeded, interim permit limits are provided as a means for the permitee to meet final limits. The final limits for a discharger are set so that Water Quality Criteria can be met in the receiving water. Water Quality Criteria, adopted by DOE, are defined in the Environmental Protection Agency's Interim Final Rule (IFR; Federal Register, May 4, 1995, Volume 60, Number 86, 40 CFR Part 131).

Interim metal discharge limitations in the current Chehalis NPDES permit were set by DOE using very limited metal sampling data. To better define and provide more reliable copper, silver and zinc data for the WWTP effluent and the Chehalis River, a more intensive sampling program was begun in 1996. This sampling program was necessary so the City could provide input towards the revision of metal limits for their new NPDES permit, which is being revised as a result of the Consent Decree.

In July 1997, Gibbs & Olson (G&O) proposed to conduct a more intensive metals monitoring program to provide more accurate data, by using "Clean Metal" techniques as established by EPA, during the dry season of 1997; and to better determine the WWTP's ability to remove these three metals from the influent wastewater stream. A sampling and analysis protocol was presented in *the City of Chehalis Monitoring and Quality*

Assurance (QA/QC) Plan (G&O, 1997). This plan was presented to DOE and approved as being adequate for the intended scope and met QA/QC requirements for the "Clean Metal" Study. "Clean Metal" techniques minimize metals contamination that may wrongfully show up as high metal concentrations when measured at the parts per billion (ppb) level. Past G&O experience has shown that "Clean Metal" testing has greatly reduced the calculated impact metals have on WQS; and in some cases, the information has been used to demonstrate to DOE that no metal limits were required.

The clean metal testing program provided more data points during the dry season, which gave better statistical representation of effluent metal concentrations during "dry weather" conditions, and provided site-specific data to develop metal Partitioning Ratios for copper, silver and zinc for the Chehalis River. The Partitioning Ratio for each metal is defined as the ratio of the concentration of dissolved metal to the concentration of total recoverable metal. Frequently, the Partitioning Ratios based on site-specific data are lower than the ratios inherently assumed when using EPA's Water Quality Criteria. All of these objectives were to provide the City of Chehalis with information to adequately assess whether or not the Chehalis WWTP is meeting Water Quality Standards and to set discharge limits accordingly.

EVALUATION OF THE REASONABLE POTENTIAL TO EXCEED WQS FOR COPPER, SILVER, ZINC AND CHLORINE

A water quality analysis for copper, silver, zinc, and chlorine is presented here. The evaluation was performed taking into account of the following:

1. Incorporation of all available metals data up to December 1998.
2. Incorporation of Darigold's flow to account for the effects of Darigold's overlapping effluent plume, which creates a combined mixing zone.
3. Presentation of use of the effect that I/I has on the WWTP flow with respect to Chehalis River flow.
4. Evaluation of dilution factors from possible discharge scenarios as a result of the

Consent Decree.

5. Presentation and use of the Partitioning Ratios determined from the "Clean Model" Study.
6. Use of a more realistic method of modeling the hardness concentration in each dilution zone.

The Water Quality Criteria are determined in WAC 173-201A-040, and by using the methods described in: *Applying Metals Criteria to Water Quality-Based Discharge Limits: Empirical Models of the Dissolved Fraction of Cadmium, Copper, Lead, and Zinc* (DOE Pub. No. 96-339); and the reasonable potential to exceed Water Quality Criteria were evaluated using procedures based on the *Technical Support Document for Water Quality-Based Toxics Control* (EPA/505/2-90-001), Box 5-2, as subsequently revised; Section #6 of the *Total Maximum Daily Load Development Guidelines* (DOE Pub. No 97-315); and the *Permit Writer's Manual: Procedures for Writing Wastewater Discharge Permits* (DOE Pub. No. 92-109, as subsequently revised).

There is more metals data available since the "Clean Metal" Study was conducted. The intention of this evaluation is to use the Partitioning Ratio from the "Clean Metal" Study and to use the entire metals data set from the period of record (5/28/96 to 9/14/98). This data set includes non-"Clean Metal" data which provides a conservative water quality analysis. The metals data for the WWTP and the Chehalis River are provided in Appendix A.

There is also considerably more hardness data available. Since January 1st of 1998, the City of Chehalis has been collecting WWTP and Chehalis River hardness almost on a daily basis. This more representative hardness data was used to determine the hardness concentrations at the edge of the mixing zone boundaries. The hardness data is also provided in Appendix A. A chlorine water quality analysis was performed to ensure that the WWTP has no reasonable potential to exceed chlorine criteria. A combined effluent discharge with Darigold was analyzed, whenever applicable.

OVERLAPPING EFFLUENT PLUMES

Because Darigold and the Chehalis WWTPs discharges create overlapping discharge plumes in the Centralia Reach, DOE has decided to combine the effluent from Darigold and the Chehalis WWTP as a single point-source discharge for any water quality analyses of the Centralia Reach. Based upon three years of Darigold data, this evaluation incorporates Darigold's flow, hardness and chlorine concentrations, whenever Darigold discharges into the Reach or provides effluent to the WWTP. The peak daily and maximum monthly Darigold flows were 0.40 MGD and 0.30 MGD (dry weather), and 0.52 MGD and 0.30 MGD (wet weather), respectively (based on flow data from April 1995 through March 1998). The lowest concentration which is 88.8 mg/l, was used as Darigold’s hardness contribution (based on 6 daily samples from December 2, 1998 through December 7, 1998). Since there was no metals data available for Darigold's effluent, metal concentrations were assumed to be negligible.

PLANT FLOWS FOR MIXING ZONE ANALYSIS

The plant flows that were used for the mixing zone analysis have been adjusted to account for the flow-based conditions. This is necessary so that the mixing zone for dry weather conditions during the winter is based on a plant flow that can be expected. For instance, when the river flow is less than 1,000 cfs in January, there is absolutely no chance that the plant flow would be 13.0 MGD. The plant flow is closely tied to river flow because they are both sensitive to rain.

Consequently, high flows measured at the WWTP frequently occur when river flows are also high, due to I/I into the wastewater system, and due to runoff into the river. Appendix A provides four plots of Centralia Reach flow, and Chehalis River near Grand Mound flow vs. WWTP flow for the dry season, and the wet season. These plots provide a means to develop a regression to relate WWTP flow (MGD) with river flow (cfs) to show that there is a strong relationship between these two flow rates.

A regression analysis for WWTP flow was performed to obtain the 99th (peak daily), and the 90th (maximum monthly) percentile concentrations. The 99th percentile was taken as 2.6 times the Root Mean Square (RMS) Standard Error; and the 90th percentile was taken as 1.7 times the RMS Standard Error plus the average value determined from the regressions made. These values were used to calculate existing dilution factors for the water quality analysis.

Future flows were determined by increasing the existing peak day, and maximum monthly flows by the projected additional flow from residential, commercial, industrial, and I/I (as presented in Section V). Projected total increases for dry season flows were 0.8 MGD, and 1.0 MGD for wet season flows, for both peak daily and maximum monthly flows. The WWTP flows used in calculating dilution factors for mixing zones are discussed below.

MIXING ZONES AND CRITICAL CONDITIONS

A mixing zone is a small volume of the receiving water inside of which chronic or acute Water Quality Standards for toxics may be exceeded. Concentrations of toxics are diluted within the volume of water allowed, and the mixing zones are established such that Water Quality Standards are met at the boundary, or edge, of the mixing zone. If the Water Quality Standards cannot be met at the edge of the mixing zone for any given toxicant, then additional treatment must be provided and/or the toxicant must be controlled at the source prior to discharge into the City's sewer system.

WAC 173-201A-100 specifies the requirements for acute and chronic mixing zones in streams as the most restrictive of the following:

- Chronic Mixing Zone shall:
  1. Not extend in a downstream direction for a distance from the discharge ports greater than 300 feet plus the depth of the water over the discharge ports, or extend upstream for a distance of over 100 feet;

2. Not utilize greater than 25 percent of the receiving water flow as measured during mean lower flow water; and
3. Not occupy greater than 25 percent of the width of the water body as measured during mean lower flow water.

- Acute Mixing Zone shall:
  1. Not extend beyond 10 percent of the distance towards the upstream and downstream boundary of an authorized chronic mixing zone, as measured independently from the discharge points;
  2. Not utilize greater than 2.5 percent of the flow of the receiving stream; and
  3. Not occupy greater than 25 percent of the width of the receiving stream.

The derivation of water quality-based limits also takes into account the seasonal variability of the pollutant concentrations in both the effluent and the receiving water. Water quality-based limits are by definition derived for the water *body's critical condition*, which represents the receiving water and waste discharge condition with the highest potential for adverse impact on the aquatic biota and existing and characteristic water body uses. For the City of Chehalis, this time period is dictated by river flow per the Consent Decree.

Since the WQS requires using the minimum dilution factor for each mixing zone determined by plume width, plume length or plume volume, a plume model needs to be developed to characterize plume dimensions as it disperses through the receiving water. Unfortunately, a river survey was not conducted to provide the necessary data to be able to model the plume dispersion. However, CH2M Hill completed a dye study for the City of Chehalis WWTP outfall in a report titled *Effluent Mixing Study and Outfall Evaluation* (January, 1993). Based on the results of this study, a determination can be made as to whether the dilution factors are restricted by plume dimensions, or by the receiving water critical flow, as per 173-201A WAC. The flow conditions which occurred during the dye study were a Centralia Reach flow of 96 cfs and a WWTP flow ranging from 0.95 MGD to 1.2 MGD (corresponding to the existing WWTP average dry weather flow of 1.15

MGD). In the study, dilution contours were plotted as far downstream from the outfall as 300 feet. Based on the plume characteristics from the dye study, the limiting chronic dilution factor was approximately 30, and the limiting acute mixing dilution factor was approximately 7.

For the receiving water, the *Permit Writer's Manual* states that the critical flow ($7Q_{10}$) for both acute and chronic conditions shall be used. The critical flow for rivers and streams is the 7-day average low flow at a 10-year recurrence interval ($7Q_{10}$). For the Centralia Reach, the $7Q_{10}$ dry weather flow used was 60.2 cfs, and the $7Q_{10}$ flow during the wet weather was 218.6 cfs. Both of these flows are used in the NPDES permit and are accepted by DOE and representative flows. The wet weather and dry weather $7Q_{10}$ flows near Grand Mound that were used are 333.8 cfs and 119.6 cfs, respectively. A Centralia Reach minimum flow of 1,000 cfs was used during future wet season conditions when the City is allowed to discharge their effluent into the Centralia Reach. As stated in the Consent Decree, flows used for permit compliance will be in accordance with the flows measured from United States Geological Survey gauging station near Grand Mound (USGS Station #12027500), as modified by the regression analysis in the Consent Decree to estimate the corresponding flow in the Centralia Reach.

WWTP flows are based on WWTP flow data from April 1995 through March 1998. With NPDES permit discharge limitations, and the consequent alternatives to meet these limitations, there were several possible cases for determining dilution factors for the water quality analysis. The most restrictive of these possibilities are outlined as follows:

1. Existing Conditions:

a) Dry weather, in the Reach: Chehalis WWTP existing 99^{th} (1.9 MGD), and 90^{th} (1.5 MGD) percentile dry weather flows; Darigold existing peak day (0.40 MGD), and maximum monthly (0.30 MGD) dry weather flows; and Centralia Reach $7Q_{10}$ dry weather flow (60.2 cfs).

b) Wet weather, in the Reach: Chehalis WWTP existing 99^{th} (3.1 MGD), and 90^{th} (2.2 MGD) percentile wet weather flows; Darigold existing peak day (0.52 MGD), and maximum monthly (0.30 MGD) wet weather flows; and Centralia Reach $7Q_{10}$ wet weather flow (218.6 cfs).

2. Future Conditions (River Enhancement):

a) Dry weather, in the Reach: Chehalis WWTP future projected peak day (2.7 MGD), and maximum monthly (2.3 MGD) dry weather flows; Darigold existing peak day (0.40 MGD), and maximum monthly (0.30 MGD) dry weather flows; and Centralia Reach $7Q_{10}$ dry weather flow (60.2 cfs).

b) Wet weather, in the Reach: Chehalis WWTP future projected peak day (4.1 MGD), and maximum monthly (3.2 MGD) wet weather flows; Darigold existing peak day (0.52 MGD), and maximum monthly (0.30 MGD) wet weather flows; and Centralia Reach $7Q_{10}$ wet weather flow (218.6 cfs).

3. Future Conditions (Land Application/Groundwater Recharge):

a) Dry weather, out of the Reach: Chehalis WWTP future Consent Decree flow limitation (2.5 MGD); and Chehalis River near Grand Mound $7Q_{10}$ dry weather flow (119.6 cfs).

b) Wet weather, in the Reach: Chehalis WWTP future peak day (4.8 MGD), and maximum monthly (3.9 MGD) wet weather flows; Darigold existing peak day (0.52 MGD), and maximum monthly (0.30 MGD) wet weather flows; and Centralia Reach 7-day average 1,000 cfs trigger flow.

4. Future Conditions (Land Application/Groundwater Recharge):

a) Dry weather: N/A (effluent would be applied to land)

b) Wet weather, in the Reach: Chehalis WWTP future projected peak day (4.8 MGD), and maximum monthly (3.9 MGD) wet weather flows, Darigold existing peak day (0.52 MGD), and maximum monthly wet weather flows (0.30 MGD); and Centralia Reach 7-day average 1,000 cfs trigger flow.

Table III-7 provides a summary of the calculated acute, chronic, and complete-mix dilution factors for each scenario listed above. Dilution factors were calculated based on the equation:

$$DF = (Q_{WWTP} + Q_{DARIGOLD} + Q_{RIVER})/(Q_{WWTP} + Q_{DARIGOLD}),$$

Where Q_{RIVER} is 25% of the critical river flow for the chronic mixing zone, 2.5% of the critical river flow for the acute mixing zone and 100% for complete-mix conditions. Darigold flow was used whenever applicable. The dilution factors for alternatives which entail discharging effluent into the Skookumchuck River and poplar irrigation or groundwater recharge) resulted in dilution factors greater than that measured in the CH2M Hill Study. However, the flows in which the CH2M Hill Study was conducted were very close to critical conditions, and do not resemble the conditions which occur during the 1,000 cfs cut-off limit. The dilution factors used in the water quality analysis are shown in Table III-7.

TABLE III-7
CHEHALIS WWTP SUMMARY DILUTION FACTORS

Scenario	Acute	Chronic	Complete Mix
Existing Conditions			
Dry Weather	1.42	6.40	22.61
Wet Weather	1.98	15.13	57.52
Future Conditions (River Enhancement Alternative)			
Dry Weather	1.36	5.23	17.91
Wet Weather	1.76	11.09	41.37
Future Conditions (Pump Below the Skookumchuck River)			
Dry Weather	1.77	8.73	31.92
Wet Weather	4.04	39.45	154.81
Future Conditions (Land Application/Groundwater Recharge)			
Dry Weather	N/A	N/A	N/A
Wet Weather	4.04	39.45	154.81

At the request of DOE, the City is currently conducting a wet weather mixing zone analysis. The results of this analysis will be presented in the Facility Plan and may affect the dilution factors presented herein. The mixing zone analysis may also lead to the eventual replacement of the single port outfall diffuser with a multi-port diffuser.

PARTITIONING RATIOS

One primary objective of the "Clean Metal" Study was to collect adequate data to develop Partitioning Ratios for the Chehalis River. Sampling requirements for the study were based on DOE's *Total Maximum Daily Load Guidelines* (DOE Pub. No. 97-315), Section #6. Sample collection followed the procedures identified in Method 1669: Sampling Ambient Water for Trace Metals at EPA Water Quality Levels (EPA, 1996); and testing procedures followed Method 1638: Determination of Trace Elements in

Ambient Waters by Inductively Coupled Plasma-Mass Spectroscopy (EPA, 1995), *Standard Methods for the Examination of Water and Wastewater* (APHA et al., 19^{th} ed., 1995), and *Methods for Chemical Analysis of Water and Wastewater* (EPA Technology Transfer, 1974).

The "Clean Metal" Study provided enough data to calculate site-specific Partitioning Ratios for copper, silver and zinc for the Chehalis River. The guidelines for calculating site-specific Partitioning Ratios are found in DOE's *Total Maximum Daily Load Guidelines*, Section #6. Table III-8 provides a comparison of the Partitioning Ratios inherently assumed using EPA's Water Quality Criteria and the site-specific Partitioning Ratios determined by the "Clean Metal" Study. The data from the "Clean-Metal" Study used to calculate the site-specific Partitioning Ratios for the Chehalis River is included in Appendix A.

TABLE III-8
COPPER, SILVER AND ZINC PARTITIONING RATIOS DEFINED BY EPA, DOE AND SITE-SPECIFIC PARTITIONING RATIOS FROM THE "CLEAN METAL" STUDY

Partitioning Ratio Defined By:	Copper	Silver	Zinc
EPA	0.960	0.850	0.978 Acute 0.986 Chronic
"Clean Metal" Study	0.747	0.491	0.619

From Table III-8, it can be seen that the site-specific ratio of dissolved metal to total metal concentrations is much less than that used by EPA's Water Quality Criteria which means that the final metal limits are set too low. As a result of the "Clean Metal" Study results, the Water Quality Criteria were adjusted to incorporate the lower Partitioning Ratios for the Chehalis River.

WWTP, DARIGOLD AND CHEHALIS RIVER CONCENTRATIONS

Since there is no available data for metals concentration in Darigold's effluent, the concentration of metals in Darigold's effluent was assumed to be zero. Combined effluent concentrations have been adjusted to reflect the discharge of both the Chehalis

WWTP and Darigold WWTP, whenever applicable. Table III-9 provides a summary of all the concentrations used to perform the water quality analysis for each scenario. The effluent concentrations are based on the 95th percentile and the river concentrations are based on the 90th percentile, or 1.74 times the geometric mean for data sets smaller than 20 values. All hardness concentrations were based on the 10th percentile. These percentiles are recommended for use in DOE's *Permit Writer's Manual*. Dry weather and wet weather data were sorted according to the NPDES permit requirements based on Chehalis River flow.

TABLE III-9
CHEHALIS WWTP SUMMARY OF WATER CONCENTRATIONS USED TO PERFORM WATER QUALITY ANALYSIS

Source	Hardness (mg/l)	Copper (ppb)	Silver (ppb)	Zinc (ppb)
WWTP Effluent				
Dry Weather	70.0	41.7	4.66	112
Wet Weather	40.0	36.0	9.48	110
Ambient Chehalis River				
Dry Weather	25.2	3.33	0.076	2.46
Wet Weather	17.4	3.10	0.072	3.64
Darigold Effluent				
Dry Weather	88.8	N/A	N/A	N/A
Wet Weather	88.8	N/A	N/A	N/A

HARDNESS AT THE EDGE OF EACH MIXING ZONE

Traditionally, metal standards are based exclusively on the hardness of the receiving water. However, to be more representative of actual conditions, the combined hardness of the discharged effluent and the receiving water must be used at the edge of the acute and chronic-mixing zone boundaries, respectively. Since metal toxicity is greater in less hard waters, a water quality analysis also needs to be performed under complete-mix conditions to ensure that there is no reasonable potential to exceed WQS under all mixing zone scenarios. The more realistic approach of using hardness data was used in this evaluation and hardness concentrations were calculated incorporating the hardness of receiving water, the WWTP effluent and Darigold's effluent. A mass balance was performed to determine the hardness at the edge of each mixing zone boundary. Table

III-10 provides a summary of the hardness concentrations at the edge of each mixing zone boundary that were used for the water quality analysis.

TABLE III-10 CHEHALIS WWTP HARDNESS CONCENTRATIONS (mg/l)			
Scenario	**Acute**	**Chronic**	**Complete Mix**
Existing Conditions			
Dry Weather	59.0	32.7	27.3
Wet Weather	32.4	19.3	17.9
Future Conditions (River Enhancement)			
Dry Weather	58.1	33.8	27.7
Wet Weather	33.3	19.8	18.0
Future Conditions (Pump Below the Skookumchuck River)			
Dry Weather	50.5	30.3	26.6
Wet Weather	24.2	18.1	17.6
Future Conditions (Land Application/Groundwater Recharge)			
Dry Weather	N/A	N/A	N/A
Wet Weather	24.2	18.1	17.6

REASONABLE POTENTIAL TO EXCEED WATER QUALITY CRITERIA

After determining the dilution factors, the next step is to determine the Water Quality Criteria for those pollutants of interest. For municipal plants like the Chehalis WWTP, DOE has normally been focusing their interest on chlorine and ammonia. Since ammonia has already been addressed in the TMDL, an analysis for ammonia is not required. Water quality analyses were conducted for copper, silver, zinc and chlorine. WAC 173-201A-040 lists the Water Quality Criteria to use for water quality analysis. Quality Criteria is provided in Appendix A. Table III-11 shows the calculated Water Quality Criteria that must be met at the edge of the mixing zones and shows if there is a "reasonable potential to exceed" those criteria.

TABLE III-11 CHEHALIS WWTP SUMMARY OF THE REASONABLE POTENTIAL TO EXCEED WQS RESULTS				
Scenario	**Copper**	**Silver**	**Zinc**	**Chlorine**
Existing Conditions				

Dry Weather	Yes	Yes	No	No
Wet Weather	Yes	Yes	No	No
Future Conditions (River Enhancement Alternative)				
Dry Weather	Yes	Yes	No	No
Wet Weather	Yes	Yes	No	No
Future Conditions (Pump Below the Skookumchuck River)				
Dry Weather	Yes	Yes	No	No
Wet Weather	Yes	Yes	No	No
Future Conditions (Land Application/Groundwater Recharge)				
Dry Weather	N/A	N/A	N/A	No
Wet Weather	Yes	Yes	No	No

To determine if the discharge has a reasonable potential to exceed the acute and chronic criteria, the method specified in the document titled *Technical Support Document for Water Quality-Based Toxics Control* (EPA/505/2-90-001, PB91-127415, March 1991) was used. The Technical Support Document specifies a statistical procedure to determine if a discharge has the potential to exceed Water Quality Standards. The procedure is based on the dilution factors calculated above, the maximum concentration of a pollutant, the number of samples represented by the maximum concentration, and the ambient concentration of the pollutant in the stream. A multiplier and coefficient of variability, which are essentially "safety factors" dependent upon the variability of the data and number of samples, are then used to calculate the "reasonable potential to exceed" Water Quality Criteria. If a "reasonable potential to exceed" exists, the delegating agency (DOE) must issue limitations upon the discharger to ensure criteria are met.

The analysis results showed that Water Quality Criteria for zinc and chlorine were not exceeded. Copper and silver showed a reasonable potential to exceed WQS under all scenarios. However under all scenarios, copper exceeds criteria in the chronic mixing zone, and also under complete-mix conditions. *This is due to the fact that ambient river copper concentrations already exceed the criteria regardless of effluent discharge for some of the discharge scenarios.*

The reasonable potential to exceed WQS for copper and silver under existing conditions has already been identified by DOE, and acknowledged by the City of Chehalis. Interim metals limits have been established in the Consent Decree and the NPDES permit to allow time for the City to implement a program to meet WQS. The Consent Decree allows the City to have up to 8 years to meet WQS. Interim metal limits posed to the WWTP within the 8-year period are performance-based, and meeting these interim limits for metals should not pose a problem. The calculation sheets are included in Appendix A.

WATER EFFECT RATIO STUDIES

WAC 173-201A provides an allowance to further modify Water Quality Critieria for metals by conducting a Water Effect Ratio (WER) Study. This provision in the Water Quality Standards allows the permittee to collect site-specific data necessary to calculate WERs for metals. The WER is defined as the ratio of the concentration at which the metal is deemed toxic using site-specific receiving water and WWTP effluent versus the concentration of metal used with EPA's laboratory-made culture-media. The WER ratio is used as a factor which is applied to EPA's Water Quality Criteria for metals.

In September and August of 1998, G&O conducted "range-finding" WERs below the Skookumchuck River, and at the existing outfall site in the Centralia Reach. The "range-finding" WER is a pared-down version of the complete WER, and is less costly. "Range-finding" WERs provide an estimate of what values to expect when a complete study is completed.

EPA's *Interim Guidance on Determination and Use of Water-Effect Ratios for Metals* (EPA-823-B-94-001), requires the use of the chronic mixing zone boundary to determine acute WERs, and the complete mixing zone boundary to determine chronic WERs. For the copper, silver and zinc "range-finding" WER testing, WERs were determined based on the premise that complete-mix conditions would provide the most stringent WER due to lower hardness concentrations. The "range-finding" hardness-adjusted chronic WER results are as follows:

- Dry Season, upstream in the Centralia Reach
  Copper: 11.0; Silver 36.6; and Zinc 3.3

- Dry Season, below the Skookumchuck River
  Copper: 6.3; Silver 9.7; and Zinc: 5.5

DOE's spreadsheets for calculating the potential to exceed WQS was used to back-calculate the acute WER values that would be required to demonstrate that no potential to exceed WQS would occur for copper, silver and zinc. The required WER values are shown in Table III-12.

TABLE III-12
ACUTE WER VALUES REQUIRED TO SHOW A "NO POTENTIAL TO EXCEED" WQS

Scenario	Copper	Silver	Zinc
Existing Conditions			
Dry Weather	2.18	1.16	0.67
Wet Weather	2.50	4.77	0.81
Future Conditions (River Enhancement)			
Dry Weather	2.30	1.24	0.71
Wet Weather	**2.69**	**5.10**	0.88
Future Conditions (Pump Below the Skookumchuck River)			
Dry Weather	2.08	1.23	0.62
Wet Weather	2.29	3.92	0.54
Future Conditions (Land Application/Groundwater Recharge)			
Dry Weather	N/A	N/A	N/A

Wet Weather	2.29	3.91	0.54

Note: Bold values indicate the highest WER required to meet WQS.

By taking the lowest WER value of each metal from the "range-finding" results, we can anticipate WER results for copper, silver and zinc to be: 6.3, 9.7, and 3.3, respectively. A comparison of the "range-finding" chronic WER values to those in Table III-12 shows that copper and silver may have no reasonable potential to exceed acute WQS once a completed WER study is completed.

In all scenarios, a copper chronic WER may also be required to show that there is no potential to exceed WQS in the chronic mixing zone, and under complete-mix conditions. The highest chronic WER required for copper is 1.77. The highest complete-mix WER required for copper is 1.11. It is expected that both of these exceedances can be met if a chronic WER of 1.77, or above, for copper is obtained.

The WER testing is very costly due to the scope and the high level of quality assurance and quality control. A complete WER Study cost estimate includes the cost to conduct acute WERs for copper, silver and zinc, and a chronic WER for copper. This is a worst-case cost estimate scenario and negotiations with DOE may result in not having to do a zinc WER, and/or a chronic copper WER. Consequently, the cost estimate is subject to change based upon both EPA and DOE's review of the WER QA/QC Workplan and is subject to any additions and/or deletions to the scope of work. The preliminary cost to do the complete WER program is $422,000. A detailed cost estimate is included in Appendix A.

The WER testing is recommended as a solution for meeting the WQS since the capital cost for implementing advanced treatment to remove metals is very expensive. In addition to increased O&M required to run the advanced metal removal units, the City will still need to prove that there is no metals problem in the next permit cycle of their NPDES permit. If a WER Study is conducted, the Consent Decree mandates that DOE revise the permit upon the completion of additional metals data after a copy of the WER report is submitted to them.

The City is in the process of submitting a water quality analysis for zinc that shows that there is no reasonable potential to exceed water quality standards. It is anticipated that once approved by DOE, zinc will be removed from the NDPES permit.

The City is proceeding with a WER Study for copper and silver. Wet weather sampling is planned for Winter 2001, and dry weather sampling is planned for Summer 2001. The results of the WER Study will be summarized in the Facilities Plan.

AMBIENT WATER QUALITY

Many factors, in addition to stream flow, impact the potential to meet Water Quaility Standards in a river. These factors include, but are not limited to, temperature, pH, hardness, ambient stream conditions and the presence or absence of salmonids. DOE has collected ambient Chehalis River data at Claquato and at the Newaukum River station near Chehalis. These two stations provide water quality for the WWTP receiving water upstream of the discharge location. Data from both stations were collected during 1996 and 1997 (See Appendix A). Since inadequate Newaukum River flow data was available, the lesser water quality of the two stations (in concentration) was used to determine whether or not Class A standards would still be met with the impact WWTP discharge included.

CLASS A WATER QUALITY STANDARDS

Pollutants in an effluent may affect the aquatic environment near the point of discharge (near-field) or at a considerable distance from the point of discharge (far-field). Toxic pollutants are, for the most part, near-field pollutants – their adverse effects diminish rapidly with mixing in the receiving water and thus a mixing zone is allowed. Conversely, a pollutant such as BOD is a far-field pollutant whose adverse affect occurs away from the discharge point even after dilution has occurred. In addition to evaluating compliance with the toxic substances criteria, compliance with Class A water quality standards specified in WAC173-201A-030 must also be evaluated.

CONCLUSIONS

Based on the data evaluated and the reasonable potential to exceed analysis, it is recommended that the City proceed with the WER Study for copper and silver. Based on the WER "range-finding" Study, the results are likely to show that copper and silver will have no reasonable potential to exceed WQS once a complete WER Study is completed.

Coupled with conducting the Water Effect Ratio Study for copper and silver; the active partnership between the City and all industrial dischargers who may be discharging silver should continue to try to find ways to reduce the overall silver loads to the WWTP, and to reduce the peak silver concentrations that have been measured in the system. The completion of the WER and a silver source reduction/management program should adequately allow the WWTP to meet WQS for metals.

RIVER ENHANCEMENT

The TMDL Study presents an intriguing argument for evaluating river enhancement as a potential water quality management program for the Centralia Reach. A July 23, 1998 letter from DOE to Chehalis (included in Appendix A) discusses the process and constraints for addressing river enhancement alternatives such as aeration.

DOE will consider allowing aeration of the river to meet water quality based limits under the conditions and requirements as follows:

- Completion of a thorough environmental, economic and engineering analysis of all best available technology.
- Completion of SEPA review documenting that aeration (or other river enhancement) is in the "overriding public interest" compared to other treatment or reuse alternatives.
- Aeration (or other river enhancement) must be applied to significantly improve the overall levels of dissolved oxygen in the Centralia Reach, not just to provide assimilative capacity for a discharge from Chehalis.
- Other entities, such as the Confederated Tribes of the Chehalis Reservation and the Washington State Department of Fish and Wildlife (WSDFW), must review and accept all alternatives being considered.

- A perpetual maintenance and monitoring agreement (or similar assurance) must be in place to ensure that sure an aeration system would be operated and maintained for consistent effectiveness.
- Provide potential cost recovery by DOE for review and evaluation of computer modeling.
- Public involvement.

Other general considerations of the letter include concerns regarding EPA approval and negative impacts to other dischargers. Strategies to evaluate river enhancement methods and alternatives are discussed in Section VII.

WATER RECLAMATION AND REUSE

Option 1 of the TMDL Study recommended reclamation and/or reuse as a potential alternative for water quality management during critical periods. Recent discussions with DOE and DOH in a July 21, 1998 meeting identified potential reclamation and reuse options and constraints. The minutes from the meeting are included in Appendix A. Unresolved issues from that meeting regarding groundwater recharge effects on groundwater quality and natural wetland enhancement are addressed through the analysis of reclamation and reuse alternatives in Section VII.

The City is also proposing to produce a Class A reclaimed water to be used for stream flow augmentation. Under this option, the wastewater would be highly treated and used to augment the flow in the Centralia Reach during dry weather conditions.

SECTION IV

CONDITIONS IN THE PLANNING AREA

INTRODUCTION

The planning area is located in the western portion of Lewis County, Washington along the Interstate 5 corridor from the City of Chehalis to south of the City of Napavine. A vicinity map is provided as Figure IV-1.

Three entities, the City of Chehalis, the City of Napavine and LCSD No.1 are responsible for operation and maintenance of respective collection system components. Collection system policy, planning and financing is determined for each individual entity through the respective commissions and councils. All three entities share in funding improvements to the Wastewater Treatment Plant (WWTP) according to capacity ownership percentage. WWTP O&M costs are tracked separately and shared by each entity according to use.

The City of Chehalis is the primary agency responsible for planning, financing, operating and maintaining the Regional WWTP. The City of Chehalis coordinates Regional WWTP policy and planning decisions with Napavine and LCSD No.1 through the Chehalis Regional Sewer Operating Board (CRSOB). The CRSOB is comprised of one elected official from the each entity. Current representatives on the CRSOB are as follows:

- Mayor Robert Spahr (Chairman), City of Chehalis
- Jim Haslett, Councilman, City of Napavine
- Chuck Weiland, Commissioner, Lewis County Sewer District No.1

INSERT FIGURE IV-1 VICINITY MAP

SEWER INTERCEPTOR AGREEMENT

The rights and responsibilities for the CRSOB members regarding collection and treatment of sanitary sewage are contained in the Sewer Interceptor Agreement (SIA), which is included as Appendix B. The current SIA was established on June 22, 1994 and is an update to the original SIA established in June of 1976, and is valid through June of 2004. If the SIA is not updated on, or after that date, the SIA will continue on a year to year basis until updated.

The SIA establishes the City of Chehalis as the lead entity responsible for treatment and disposal for all sewage in the planning area. The City of Chehalis establishes rates for collection and treatment of sanitary sewage. Proposed rates must be presented to the CRSOB for review and comment prior to enactment by the Chehalis City Council. The City of Napavine and LCSD No.1 must prepare an annual report to the City of Chehalis with the number and classification of all sewer connections. The CRSOB is designated as the responsible entity for addressing any necessary dispute resolution related to SIA issues.

Interceptor line and WWTP capacity ownership is established by Exhibit A of the SIA and is shown as Figure IV-2. Equivalent Residential Units (ERUs) are used to establish a uniform basis of relative capacity allotted to each entity. Average ERU capacity established in the SIA for use in Exhibit A is 250 gallons per ERU per day. Average ERU capacity represents an estimate of 2.5 persons per average single-family residential household contributing 100 gallons per person per day. A factor of 2.5 is utilized to establish peak hydraulic flow per ERU (not including I/I). It should be noted that ERU values are system and time specific and may not correspond exactly to specific wastewater flows from each entity on a regional basis. However, if applied uniformly, the ERU value creates a basis for fair and reasonable cost sharing on a regional basis. The process to update the proportion of interceptor and treatment capacities for each entity will begin after completion of the GSP and Facility Plans. The financial analysis in Section VIII of this report presents several cost sharing options based on projected population of each entity.

INSERT FIGURE IV-2 INTERCEPTOR LINE AND WWTP CAPACITY OWNERSHIP

SEWER SERVICE AREA

Sewer service areas are shown in Figure IV-3. The existing service area is comprised of incorporated City limits, District boundaries and areas currently served by sewers within unincorporated areas.

The 2025 future service area shown in Figure IV-3 is approximately 8,700 acres. The future service area represents the Interim Urban Growth Management Areas (IUGA) for each City with minor adjustments to account for anticipated growth in LCSD No.1 outside of the City IUGAs. The IUGAs were established through County Ordinance on May 4, 1998 in accordance with the planning process under the Growth Management Act (GMA). The Cities and the County have developed comprehensive plans in conformance with GMA that establish Urban Growth Areas (UGAs) and future land use. The UGAs and any amendments to the UGAs will continue to represent the future service area (i.e., as the UGAs change, so will the future service area).

The City of Napavine updated their comprehensive plan in August of 1998 and is in compliance with GMA. The Chehalis Comprehensive Plan was adopted in July 1999. The Lewis County Comprehensible Plan is currently being appealed.

The ultimate service area identified in Figure IV-3 is approximately 12,200 acres and represents areas that may be served by the WWTP, but are not expected to contribute to WWTP flows until after 2025. Additional discussion regarding future service areas and GMA are included in Section V.

The current (1997) population within the planning area is estimated at 8,671 persons. Population projections for the 2025 service area are presented in Section V. Population projections for the ultimate service area are not presented in this report. The ultimate service area is presented specifically for use in Section VI to determine future collection system flows.

INSERT FIGURE IV-3 SERVICE AREA MAP

ECONOMICS

Since the early 1970's, unemployment in Lewis County has been consistently higher than the statewide average (Washington State Employment Security). Employment growth in Lewis County has slightly exceeded population growth in recent years by 0.17% per year (Hovee). The continuing transition of the resource based economy along with more recent welfare to work programs, will assert additional pressure for job creation throughout the County. With the implementation of the GMA in Lewis County and lack of comprehensive utility service in unincorporated areas, the Chehalis-Centralia area will likely support a majority of the Lewis County job growth to adequately meet the needs of the anticipated population growth. Due to the relatively large amount of industrial land in Chehalis and Napavine UGAs compared to Centralia, industrial development in the County will likely occur in the planning area to meet County needs for family wage jobs.

LAND USE

Currently, the County, as well as the Cities of Chehalis and Napavine designate land use within their respective jurisdictions. As stated above, each jurisdiction must prepare comprehensive plans in conformance with GMA. The comprehensive plans for the Cities will designate future land use for their respective UGAs. Land use designations are shown in Figure IV-4 and includes only general land use classifications for the purpose of this report. Some of the designated land uses may change as final GMA plans are completed, but these changes are not anticipated to have a significant affect on projected sewer service levels.

Land use within the City of Chehalis corporate limits is regulated by the City of Chehalis Municipal Code Title 17 ("Zoning"). Within that title, Chehalis has twenty-one land use

INSERT FIGURE IV-4 LAND USE

designations. These "regular" zones include: single-family residential, (low and medium density), multiple-family (medium-density), multiple-family (high density), four forms of commercial, light and heavy industrial, and ten forms of essential public facilities. The City of Napavine has similar land use regulations, but fewer land use designations.

Land use outside the City limits is governed by Lewis County, which at this time employs no land use zoning. Current "county" land uses within the service area include commercial, industrial, agricultural and rural residential. There are also a few residential subdivisions with densities close to that of urban residential areas. Table IV-1 shows acreage within the 2025 and ultimate service area boundaries for general land use classifications. All of the commercial/industrial designation has been included in the industrial classification in this table.

TABLE IV-1
LAND USE ACREAGE

TYPE	2025 ACREAGE	ULTIMATE ACREAGE
Residential	3,000	5,600
Commercial	1,300	1,800
Industrial	2,000	2,200
Areas not suitable for development	2,400	2,600
Total	**8,700**	**12,200**

OTHER SERVICES

In the planning area, utilities are provided by a combination of city managed, state regulated, federally licensed and municipally franchised providers. In Chehalis, city managed utilities are sewer, water, solid waste and stormwater. The remaining non-city managed utilities are cable television (AT&T), electrical (Lewis County PUD), natural gas (Puget Sound Energy) and telephone and cellular (Qwest Communications). In Napavine, the city managed utilities are water and sewer. Napavine receives the same non-city managed utility services as Chehalis other than natural gas. LCSD No.1 manages sewer service only. A majority of water service in LCSD No. 1 is provided by the City of Chehalis. Electrical and gas utilities within the planning area are regulated by the Washington Utilities and Transportation Commission (WUTC) while the telephone and cellular telephone services are federally licensed. Cable television services are provided under municipal franchises through AT&T.

The 1990 GMA requires all comprehensive plans to contain a Utilities Element that includes the general location, proposed location and capacity of all existing and proposed utilities (RCW36.a.070- (4)). The utility element for Napavine, Chehalis, and LCSD No. 1 (through the County plan) has been completed.

TRANSPORTATION

Interstate 5 extends north/south throughout the service area. The planning area is served by five on/off ramps with one in the immediate vicinity of the WWTP. Railroad service is available to Seattle, Portland and Grays Harbor. Railroad service to Willapa Harbor was recently discontinued. The planning area is served by the Chehalis/Centralia Airport. The Federal Aviation Administration (FAA) publishes guidelines regarding minimum setbacks for facilities and water bodies from airports. The proximity of the airport can have negative ramifications for siting equalization basins or constructed wetlands in the vicinity of the WWTP. However, the airport is already limited by the fact that it is completely surrounded by wetlands and oxbow lakes. There are also height restrictions for structures built within the airport flight path.

PHYSICAL CONDITIONS

TOPOGRAPHY

The topography of the study area is characterized by the flood plain of the Chehalis and Newaukum Rivers and the adjacent low river terraces. Flood plain areas are flat or gently sloping. River terraces form very subdued stair-steps in the terrain which branch upward to the southeast and laterally away from the main stem of the Chehalis and Newaukum and toward the southwesterly fringes of the planning area, and are adjacent to the upper reaches of creeks flowing into the Chehalis and Newaukum Rivers. The gradients of the major stream tributaries increase greatly toward their headwaters, as is typical in Western Washington. Low-lying areas range in elevation from approximately 160-feet to 300-feet Mean Sea Level (MSL). Higher areas range in elevation from approximately 300-feet to 500-feet MSL.

SOILS

Soils within the drainage basin of the existing wastewater collection system are principally flood plain deposits or gravel, sand and silt. Another large portion is occupied by glaciofluvial sand and gravel in matrix of clay and silt, which forms a well-defined terrace along the Chehalis and Newaukum Rivers. Both these soils yield large supplies of groundwater.

The low, flat flood plain areas are primarily made up of recent alluvium soils or silt layers deposited by the River. A majority of these soils are poorly drained and are subject to unfavorable high water table causing swampy conditions in wet seasons as well as fall and winter flooding.

CLIMATE

Climate in the planning area is moderate with cool, dry summers and wet, moist and cloudy winters. Average daily temperatures vary from 45 degrees in January to 78 in July. Average annual precipitation is 47 inches. Average rainfall during the period of May through October is 25 percent of the total yearly precipitation. In contrast, average rainfall from July through September is 10 percent of the total. Freezing weather seldom continues for more than a few days before warmer, moist air from the ocean moves inland.

AIR QUALITY

The Environmental Protection Agency (EPA) and DOE set regulations for air quality. The EPA has established National Ambient Air Quality Standards (NAAQS) for six "criteria pollutants": carbon monoxide, particulate matter, nitrogen oxides, sulfur dioxide, ozone and lead. The Southwest Clean Air Authority (SCAA) is responsible for regulation and monitoring of air pollution in Lewis County.

The prevailing direction of wind around the planning area is south or southwesterly during the wet winter season and northwesterly during the dry summer season. The strongest winds are generally southeasterly to southwesterly and associated with more intense winter systems. During the winter, automobiles tend to produce more carbon monoxide, and home heating produces both particulate matter and carbon monoxide, especially when wood is used as a fuel. Conversely, breezes are generally stronger during spring and summer when less carbon monoxide and particulates are produced.

In the Chehalis area, the main sources of air pollutants are automobiles, wood stoves, road dust and industrial emissions (SCAA, 1994). Industrial air pollutant sources in Chehalis and its vicinity include Centralia Steam Plant, Coast Millwork Company, Kinnear of Washington, Lakeside Industries, Hardel Mutual Plywood and Northwest Hardwoods. All six operations are well below national EPA standards. The largest emission source in the State of Washington is a coal fired power plant operated by Pacific Corp to the northeast of the planning area. All of these point sources are regulated by 5-year operating permits issued and tracked by both SCCA and EPA.

SURFACE WATERS

The major surface water resources in the planning area are the Chehalis River and its tributary, the Newaukum River.

Newaukum River

The Newaukum River is a typical, medium-sized western river flowing generally northwest and entering the Chehalis River about three-quarters of a mile west of the City of Chehalis at Corps of Engineers Chehalis River Mile (RM) 75.4. About eleven river miles upstream from its mouth, the Newaukum forks into two branches; North Fork and South Fork. The Newaukum River drains approximately 155 square miles, with an average discharge of approximately 500 cubic feet per second (cfs). The City of Chehalis diverts approximately 5 cfs for municipal use from the North Fork of the river, approximately 17 miles from the confluence with the Chehalis River. Along the river there are various small diversions made for agricultural and domestic uses.

The TMDL Study reported that flow in the Newaukum River ranged from 27 to 72 cfs during the study period, and made up about one-half of the flow below the confluence of the Chehalis River. DO and pH were mostly within water quality standards during the study with one DO measurement just below 8.0 mg/l in an early morning sample. Temperatures exceeded 18°C in three of the six measurements taken.

Chehalis River

The main stem of the Chehalis River is over 100 river miles and covers a drainage area of approximately 1300 square miles. The Chehalis River is a relatively shallow and swift-moving stream. However, near RM 74, a section of the river called the "Centralia Reach" deepens, and stream velocities decrease substantially. Throughout most of this reach, the Chehalis River is confined to a deeply cut, meandering channel averaging about 50-feet wide. The Centralia Reach is characterized as having intermittent deep pools up to 30-feet deep. During low-flow periods, stream velocities as low as 2 to 3 miles per day are common (DOE, 1984). Below the mouth of the Skookumchuck River, near RM 67, the Chehalis River becomes wider and shallower. River velocities are much higher below RM 64.

WAC 173-522-020 specifies base flows for the Chehalis River basin. During the TMDL Study, flow was measured at twelve control stations on the main stem Chehalis River and its tributaries. Of the twelve control stations, only three (Cedar Creek, Salzer Creek and the Skookumchuck River) were referenced as having flows higher than base flows during August 1992.

The Chehalis River supports a diverse variety of aquatic life. Important salmon runs include spring, fall and summer chinook, coho and chum. Salmon are present within the Chehalis River on a year-round basis. The portion of the Chehalis River downstream of the WWTP is not a prime spawning area, but does serve as a transport zone for both spawning and downstream returning salmon. Riverbanks in this area are commonly lined with deciduous trees and/or brush. The Chehalis River Basin is not glacially fed, although snowmelt makes a minor contribution to flows in the upper Newaukum River watershed (CH2M Hill, 1998).

Water quality problems have been identified in the Chehalis River basin for at least 30 years. The TMDL Study lists general causes of water pollution which include municipal and industrial WWTP effluents, septic tank effluent, urban development and storm runoff, stream bank degradation, poor domestic livestock management, forest practices and pesticide usage (agriculture). Pollutant sources identified in the study area are discussed throughout the TMDL Study.

The Centralia Reach of the Chehalis River was characterized in the TMDL Study as having numerous stratified areas during the summer months in locations with deep pools. Temperatures at the surface in these stratified areas were very high during July and August. The deep waters of the stratified areas were cooler, but were mostly found to be anoxic, especially from RM 71.0 downstream. Some of the stratified areas showed evidence of water quality degradation from local pollutant inputs, in particular: at sites north of the Chehalis/Centralia Airport (RM 70.7) and below Salzer Creek (RM 69.1).

Other Tributaries

There are two tributary creeks, Dillenbaugh Creek and Salzer Creek that are suspected to have an adverse impact on water quality within the Centralia Reach. Salzer Creek was characterized in the TMDL Study as having the worst water quality of any tributary in the upper Chehalis River basin. Dillenbaugh Creek enters the Chehalis River at approximately RM 74.5 just upstream of the WWTP. Salzer Creek enters the Chehalis River at approximately RM 69.3. Salzer and Dillenbaugh Creeks contributed approximately 2% and 1% of the total low flow within the Chehalis River, respectively during the TMDL Study.

Temperatures in the tributaries were identified above the 18°C criterion on several occasions. The most likely cause of increased water temperature identified is loss of riparian canopy vegetation. Restoration of the riparian canopy on these tributaries would likely reduce water temperatures. Dillenbaugh Creek was reported to have extensive wetlands near the mouth that may produce low DO by natural processes. All the creeks with low DO in the TMDL Study have current livestock impacts.

FLOOD PLAINS

Flood levels for rivers and streams within the planning area are available in the November 1979 Flood Insurance Study published by the Federal Emergency Management Agency (FEMA).

The entire WWTP site is located in the Chehalis River flood plain in the proximity of River Mile 74.3. The westerly portion of the site is within the FEMA designated floodway which is shown in Figure IV-5. Flood stage levels for the WWTP site, as documented in the November 1979 Flood Insurance Study, are shown in Table IV-2. Flood stage levels for a 25-year event in Table IV-2 are interpolated from the FEMA data for the purpose of addressing DOE design standards.

INSERT FIGURE IV-5 EXISTING WWTP SITE DESIGNATED FLOODWAY

TABLE IV-2 1979 FLOOD STAGE LEVELS IN THE VICINITY OF THE WWTP	
Flood Event	Chehalis River Elevation (MSL) at River Mile 74.3 Approximate streambed elevation = 138-feet
500-year	181.0-feet
100-year	179.0-feet
50-year	178.5-feet
25-year	178.0-feet
10-year	177.0-feet

FEMA is in the process of updating floodway and flood plain maps. The updated 100-year flood levels in the vicinity of the WWTP may be as high as 179.5, which corresponds to the flood of record (February 1996). A floodplain map by Pacific

International Engineering (PIE) based on the February 1996 flood of record for the planning area is included in Appendix B.

Lewis County, through a contract with PIE, is currently developing flood-stage modeling and mitigation proposals to lower the Chehalis River flood stage. PIE completed a draft report in October of 1998, which outlines mitigation alternatives that will reduce flood stage levels in the Centralia/Chehalis area. Effects from the proposed mitigation efforts identified in the report may result in a 1 to 2-foot reduction in the flood stage at the current WWTP site. Design and construction of any WWTP improvements should consider the FEMA update, as well as, the information from the County flood mitigation study.

The City has recently completed a FEMA financed program to purchase property within the floodplain and floodway near the WWTP site. These properties could be used by the wastewater utility for nuisance abatement, floodway offsets and future facilities. However, these properties cannot be used for any permanent facilities which would prevent them from passing floodwaters (i.e.: buildings, diked basins. etc).

Figure IV-6 identifies the hazard mitigation program area and property currently under City ownership in the vicinity of the WWTP.

The original facilities at the WWTP were constructed in 1948, decades prior to development of the DOE design standards and the most recent flood stage levels. Ground elevations at the WWTP site range from a high of approximately 178-feet along the southeast dike to a low of approximately 173-feet along the frontage road to the north. Ground elevations for the majority of the plant are approximately 175 to 176-feet. Consequently, many of the original facilities did not meet the basic intent of the DOE guidelines. In recognition of this, all existing mechanical and electrical equipment at the plant has been raised above the current 100-year flood stage. Most of the plant functions can be adequately controlled during a 25-year flood event. However, the trickling filters and aeration basins are subject to over-topping depending on the severity of the flood event.

Past flooding at the WWTP site would typically begin as the stormwater drainage system backed up into the plant. Since the installation of pinch valves on the plant drainage system, flooding is delayed and reportedly starts on the northeast side, which is consistent with the relatively low ground elevations. The WWTP and/or Shoreline Drive have flooded at least seven times since December 1989, according to WWTP records. Flood impacts have ranged from creating minor access problems to structural damage of facilities.

During a recent flood event on December 30, 1996, flooding at the WWTP site damaged the concrete walls of the north aeration basin. The flood stage, as measured at monitoring station (MS) #4 (near the WWTP), peaked at 177.8-feet. This event was in excess of the estimated 25-year flood stage, but far below the 100-year flood stage and many other floods that have occurred.

INSERT FIGURE IV-6 HAZARD MITIGATION PROGRAM AREA

The highest recorded flood event in the past 70 years occurred on February 8, 1996 when the recorded flood stage at MS #4 was 180.2-feet. No noticeable damage occurred to the WWTP facilities, but this and previous flooding may have been a contributing factor to the damage during the subsequent event on December 30, 1996.

GROUNDWATER

The major geological formations of the planning area are the glacial outwash of the Pleistocene Age and the alluvial terrace deposits of late tertiary to Quaternary Age. The alluvial terrace deposits are the most predominant of the planning area. Alluvial terrace deposits generally occur as a yellow-gray to yellow-brown heterogeneous mixture of gravel and sand with lesser amounts of silt and clay. Lenses of sand or clay are common, as well as, lenses of till. The thickness of the alluvial terrace deposits exceed 150-feet and are thin towards the foothills. Extensive weathering of the upper 20 to 40-feet of the formation has reduced the permeability in some areas, whereas unweathered gravel in the lower parts produces yields of approximately 200 gallons per minute (GPM) in a few wells. Widely spread, thick clay and silt sections yield little water. Water levels are generally less than 400-feet below the surface.

Groundwater yields on the river terraces are usually small and often contain an objectionable amount of iron. In spite of these objections, this source has been extensively developed for domestic and agricultural purposes because of its accessibility.

A large portion of the Newaukum River Basin contains an artesian aquifer capable of providing moderate to large quantities of water of a reasonably high quality. Artesian water is obtained by tapping the down-folded tertiary rock. The sources of this water are on the hills to the north and south of the valley where these water-bearing strata are near the surface. Here rainwater flows down permeable strata, flushing out the saline water normally found at this level and providing water under pressure in the valleys.

The TMDL Study referenced several previous studies which estimated groundwater inflows from above Bunker Creek (RM 86.0) to Prather Road (RM 59.9). Average inflow rates ranged from 0.5 cubic feet per second (cfs)/mile at the upstream end of this area to 4.5 cfs/mile near the mouth of Lincoln Creek (RM 64.2 to 62.0). The TMDL Study concluded that groundwater inputs to the main stem may constitute up to one-third of the low flow reaching the Mellen Street Bridge (RM 67.5).

Future increases in irrigation needs must be met almost entirely from groundwater sources since much of the surface water and shallow groundwater is over appropriated by DOE estimates. Because of this, it is anticipated that water reclamation and reuse may become more prevalent in the future. Records of public water supplies within the vicinity of the WWTP were obtained from the Washington State Department of Health (DOH). From this information, there are no public water supply wells immediately downstream of the WWTP and only two public water supply wells within approximately 3 miles upstream of the WWTP along the Chehalis River.

WETLANDS AND SHORELINES

Wetlands

Because a majority of the planning area is located in a wide, flat valley with very minimal slope variation, the community is bordered on the west by several small to midsize wetlands. Wetlands support regular large concentrations of wintering migratory waterfowl, fish and other wetland species.

The approximate location of known wetlands has been inventoried and mapped by the United States Department of the Interior's National Wetlands Inventory. Wetland locations are available from Washington State Department of Fish and Wildlife's (WSDFW) Public Data Release Maps. The City of Chehalis and Napavine have adopted these wetland maps and use them for guideline locations when a development proposal is submitted. The City of Chehalis had made attempts in the past to verify wetlands and update their maps accordingly.

Shorelines

Streams within the study area that are subject to the Washington State Shoreline Management Act of 1971 include the Chehalis River, Newaukum River, Salzer Creek and Dillenbaugh Creek. In addition, the shoreline of the Chehalis River is designated as Shoreline of Statewide Significance. Activities within the shorelines of these waterways are guided by the regulations contained in the Chehalis Shoreline Master Program (SMP).

The SMP contains policies and regulations that specify permitted land uses within these shoreline areas and afford protection to these areas based on the designated shoreline environments. One policy of note reads as follows: "Sewage treatment, water reclamation and power plants should be located where they do not interfere with other public uses of the water and shoreline."

FISH AND WILDLIFE, THREATENED AND ENDANGERED SPECIES

The Chehalis River Basin contains approximately 3,353 miles of stream habitat, providing a complex and diverse ecosystem. WSDFW has been contacted to provide a list of state and federally listed and proposed threatened and endangered species, candidate species and species of concern that may be present within the area of the proposed sewer service area. The planning area has a regular concentration of bald eagles, which are a state and federal listed threatened species. Other listed species that have been identified to have habitats in the area include osprey, wild turkey and the Olympic mudminnow. Spawning and rearing areas within the basin support several economically viable species of anadromous fish including chinook salmon (*Oncorhynchus tshawytscha*), coho salmon (*O. kisutch*), chum salmon (*O. keta*), steelhead trout (*O. mykiss*), cutthroat trout (*O. clarki*) and Dolly Varden char (*Salvelinus malma*).

Early findings described in a report titled *"Chehalis River Basin Fishery Resources: Status, Trends and Restoration Goals"* (USFWS, 1993) and additional reports from USFWS and the Western Washington Treaty Indian Tribes show fish populations have declined as a result of pulp mill effluents, increased temperature and/or low DO, dams and diversions, domestic animal practices, forest practices, agriculture, urbanization and industrialization, gravel mining, sedimentation and excessive commercial fishing. However, with the exception of winter steelhead in the Skookumchuck and Newaukum Rivers, fish stocks in the Chehalis River system are considered healthy.

HISTORICAL AND CULTURAL RESOURCES

Archaeological Sites

Prehistoric use of the Chehalis River Valley by Native American people was high, due to the abundance of salmon and other resources in the area. A number of archaeological sites have been discovered in the study area during excavations for construction projects, although few systematic surveys have been undertaken. Investigations of known sites have yielded valued assemblages of artifacts dating back as far as the Olcott Phase (4,000-7,000 years before present). The general likelihood of archaeological resources being present is high throughout the study area. If any construction activities encounter archeological finds, construction will need to be suspended and the State Office of Archeology notified.

Prime and Unique Farmlands

Proposed land use for the planning area does not include farmland. However, there are areas within the planning area boundaries that have soil conditions that are designated prime farmland soils by the Natural Resource Conservation Services located mainly in the rich alluvial soils adjacent to the Chehalis and Newaukum Rivers. The soils in these areas are comprised primarily of Newberg fine sandy loam, with lesser amounts of Chehalis silty clay and Cloquato silt loam. There are no designated agricultural resource lands in the planning area by the City or County under the provisions of the GMA.

ADJACENT WASTEWATER FACILITIES

The City of Centralia WWTP is located approximately 2 miles north of the northern boundary of the planning area and approximately 3.5 miles north of the Chehalis Regional WWTP. The Centralia WWTP provides secondary treatment up to approximately 7.5 MGD as presented in the City's 1998 Facilities Plan (CH2M Hill, 1998). The 1998 Facilities Plan recommends replacement of the existing Centralia WWTP at one of three sites to the north of the existing WWTP. Options for utilizing the new Centralia WWTP as a regional facility for Centralia and Chehalis are discussed in Section VII of this report.

DESCRIPTION OF EXISTING WATER SYSTEMS

A majority of the water service in the planning area is provided by the City of Chehalis and City of Napavine municipal water systems. There are numerous small water systems and private wells which provide water service to a relatively small percentage of the population located between the two cities. The majority of the smaller systems are located in the proximity of LCSD No.1. Future service areas for the Chehalis and Napavine water systems correspond to each City's UGA.

CITY OF CHEHALIS WATER SYSTEM

The City of Chehalis water system currently provides service to approximately 3,160 service connections, of which, approximately 900 are outside city limits. The service connections outside the city limits are primarily along Jackson Highway and the North Fork Road. Figure IV-7 shows the major components of the City of Chehalis water system. The City of Chehalis currently has two sources of supply, one providing water from the North Fork of the Newaukum River and the other from the Chehalis River.

INSERT FIGURE IV-7 CHEHALIS DRINKING WATER

The North Fork of the Newaukum River Source

This supply system includes intake facilities and equipment consisting of a bar screen, traveling screen, turbidity monitoring and chlorination equipment, standby power and approximately 17.5 miles of raw water transmission line. The intake site is situated approximately 17 miles from the city, approximately 10 miles east of Jackson Highway, in Section 20, Township 14 North, Range 1 East, W.M. The watershed of the intake encompasses an area of about 18 square miles predominately owned by the Weyerhaeuser Company.

A majority of the 16-inch transmission line from the intake to the Henderson Park pump station line was replaced in 1977 with ductile iron pipe. The cast iron portion of the line is believed to be in acceptable condition. Until recent years the intake operations were conducted jointly by the cities of Centralia and Chehalis and operational costs were shared by both cities. After provisions of the Federal Safe Drinking Water Act prohibited the City of Centralia from using "filtered water" from this source, they reluctantly curtailed their operations in 1993, and in 1994 were forced to abandon the Newaukum as an unfiltered supply source.

Now that Centralia is no longer using this source, it appears that Chehalis would be entitled to withdraw a much greater quantity of available water (2.8 MGD based on the City's initial 1912 right and 6.46 MGD based on the City's 1923 right). Even though this quantity of water appears to be sufficient to satisfy the 2015 peak day demand, the probable least mean monthly flow at the intake has been estimated in previous studies to be as low as 5.2 MGD.

Chehalis River Source

The Chehalis River pump station and intake were constructed on the east bank of the Chehalis River near Riverside Road at approximately Chehalis river mile (RM) 75 in 1961-62. The intake is a 10-foot square wooden crib with a layer of 6-inch rocks in the walls to act as a screen. A 48-inch diameter corrugated metal pipe extends from the intake crib approximately 50-feet to the pump station wet well located on the riverbank.

The wet well is a reinforced concrete structure, 19-feet in diameter. The station originally was equipped with one 100 horsepower (Hp) and one 150 Hp vertical turbine pump. In 1993, a third pumping unit (150 Hp) was added. An automatically cleaned traveling screen is housed in the pump station and screens the water before it enters the wet well.

The three pumps discharge into a common 18-inch steel transmission line that extends approximately 8,000-feet to the water treatment plant. The present capacity of the facility is 5.04 MGD (7.8 cubic feet per second), which is more than the projected 2015 peak day needs. The electrical service and controls were replaced within the past four years and even more recently controls were replaced and relocated above the record flood level. The existing 18-inch line from the pump station to the filter plant has a capacity of 15 cfs, which is the City's water right permit instantaneous limit.

Just like the North Fork source, the Chehalis River source faces vulnerabilities related to forest practices that take place in the upper watershed of the river and its tributaries. This source also faces potential problems related to agricultural and dairy activities that take place upstream of the intake.

It is anticipated that Chehalis will be required to use the Chehalis River intake to augment flows from the North Fork and make up differences in peak day demands beyond those that can be currently supplied (1.76 MGD in 2015). This source also provides a backup in the event of a failure or problem with the North Fork supply, in which case it would provide the entire water supply to the City.

Centralia – Chehalis Intertie

The Cities of Centralia and Chehalis have constructed an emergency intertie, connecting the two cities' water systems. The intertie is currently un-metered, but has two valves, one operated by each city. Operation requires cooperation and specific action by both cities. The purpose of this system is to provide each city with a source of water, although limited, from the other's water system, during emergency conditions.

WATER RIGHTS

Washington Water Law

The surface water code of the State of Washington, Chapter 90.03 RCW, was enacted in 1917. Before that, water rights could be established under the common law. These older water rights are often termed common law, or "vested" water rights. Common law water rights are of two types: riparian rights and appropriative rights. Riparian water rights must be used upon lands that are adjacent to the water body from which the water is withdrawn. The common law appropriation doctrine sanctions withdrawing water and using it at distant locations.

North Fork of the Newaukum River

The City of Chehalis initiated a common law appropriation of water from the Newaukum in 1912. Centralia later applied for water rights. After a series of disputes, a State Supreme Court decision in 1954 decreed that the City of Chehalis has the right to the first 2.8 MGD of flow in the river at the intake. The City of Centralia, which has ceased withdrawing water from the North Fork source, has (and may still have) a subsequent right to the next 4.8 MGD. The Court also ruled that the City of Chehalis had the right to all water in excess of 7.6 MGD. The City also has an additional certificate for 10 cfs (6.46 MGD) vested through a water right permit certificated in 1923. These water rights are sufficient to supply the projected 2015 peak day needs, however, the transmission main system cannot currently deliver the entire quantity to the water treatment plant.

Chehalis River

The City of Chehalis holds a water right permit to withdraw up to 15 cfs from the Chehalis River, dating back to 1957. The permit contains a 50 cfs minimum flow provision. The permit contained an initial completion date of May 1, 1962, which has been extended a number of times. In 1996, the City requested that the permit be certificated, based on projected 20-year use projections that included providing a proposed power generating facility with raw water. Since DOE saw the issue of supplying the raw water to the proposed power facility as an outstanding (but uncertain) factor that could significantly influence the 20-year projections, DOE elected to extend the permit for ten years, until May 1, 2006.

Water Treatment Plant

The treatment plant was constructed in 1960-61 and its components include a flash mixing chamber where coagulant is added and mixed, two slow mixing chambers (in series), a presettling basin and two (parallel) settling basins, two rapid sand filters (also in parallel) and a clearwell. The water surface elevation (maximum) at the treatment plant is 415.7-feet. Raw water from the North Fork and/or the Chehalis River may be fed into the plant. The plant provides coagulation, flocculation, sedimentation, filtration, disinfection, pH adjustment/control and fluoridation. Aluminum chloride hydroxide is the primary coagulant that is currently used at the plant. Lime is used to provide pH adjustment and the plant generally maintains a finished water pH of 7.2 to 7.4.

Post-treatment disinfection is accomplished with chlorine gas applied to provide a residual concentration ranging from 0.4 to 2.0 parts per million (ppm) in the water distribution system, with a distribution system average of approximately 1.0 ppm.

Three certified operators staff and operate the facility. They also conduct water quality monitoring, inspecting and testing throughout the water system. The plant operates 24 hours per day and is staffed at least eight hours per day during the workweek.

During the past six years, the plant has been upgraded to include a streaming current detector that provides extremely responsive coagulant feed rates that are automatically varied as the demand dictates and an emergency backup generator, which provides essential electric power during emergencies and outages. The water treatment plant currently provides finished water with a turbidity typically ranging from 0.03 to 0.09 nephelometric turbidity unit (NTUs).

Based on the maximum filter rate of 2.5 gpm per square foot of surface area (as established by DOH criteria), the water treatment plant's current capacity is 4.8 MGD or 3,360 gpm. Although the plant has a listed capacity of 4.84 MGD, the effective "operating" capacity is actually closer to 4.0 MGD. This difference is due to down time for filter backwashes and operational flow reductions that are required. Chemicals on hand include liquid aluminum chloride hydroxide, lime, fluoride, liquid chlorine, filter aid polymers and various laboratory chemicals and reagents.

Storage

The storage facilities for the main pressure zone (low-level system) consist of two reservoirs: a 5 MG reservoir located adjacent to the water treatment plant; and a 1 MG reservoir located south of the current city limits. The upper-level system is served by a 100,000-gallon reservoir. The Valley View pressure zone distribution system is served by two 67,000-gallon reservoirs (a total of 134,000 gallons).

The primary reservoirs (Main and Kennicott Reservoirs) serving the main zone have sufficient capacities to meet demands beyond the projected 2015 required levels. The other reservoirs, however, cannot meet current demand including fire flow needs, and improvements will be made as part of the City's Capital Improvement Plan (CIP). An additional 100,000-gallon reservoir will be constructed to augment the existing High Level Reservoir. In order to address the potential fire flow needs at the north end of the city, a 500,000-gallon reservoir will also be constructed.

Water Demand and Conservation

Water use in the Chehalis system is metered at several locations prior to treatment, post-treatment as the water enters the distribution system and through individual water meters. The total amount of raw water has been broken down into its components of plant operational use and loss, total entering the reservoir (distribution system), system operational losses and total demand. These water qualities were evaluated and this information was used to develop water consumption, use projections and peak need quantity forecasts in the City's 1997 Water System Plan (WSP). The 1997 WSP shows total residential water consumed divided by the number of residential connections results in an equivalent residential unit (ERU) of 183 gpd or approximately 65 gpd per capita at current housing densities.

Water use projections shown in the WSP assume water production per ERU can be reduced by 2.5 percent in 20 years by incorporating the programs presented in the City's Water Conservation Plan. Beyond use reductions that result from rate increases, the conservation plan assumes that strong public awareness and utilization of low-volume plumbing fixtures, and implementation of uniform water rates will result in long-term reduction beyond the already low demand per ERU.

CITY OF NAPAVINE WATER SYSTEM

The City of Napavine water system provides service to approximately 450 service connections, of which, approximately 10 are outside of the city limits. An elevated 100,000 gallon steel reservoir constructed in the early 1970's provides storage and system pressure. A 350,000-gallon at grade reservoir and booster pump station was recently constructed. The following lists the City's wells and capacities:

Well No. 1: capped
Well No. 2: 80 gpm
Well No. 3: 35 gpm
Well No. 4: 110 gpm
Well No. 5: 90 gpm

Figure IV-8 shows the City of Napavine's water system facilities.

Water quality has historically been excellent other than recent coliform problems. The system is completely metered. Leakage is not currently a problem as recent records indicate an average of more than 85% of water pumped is accounted for in metered consumption. The distribution system is generally adequate for domestic flows, however, most areas do not have adequate fire flows. The City is currently preparing a WSP update to address storage, hydraulic and capacity difficulties. Water use per capita reported in previous planning documents was 113 gpcd, which is projected to be approximately 300 gpd per ERU at current population densities.

LEWIS COUNTY SEWER DISTRICT NO. 1 WATER SERVICE

Water service within the LCSD No.1 is provided through a combination of private wells, small public water systems and extensions from the City of Chehalis water system. Further service from the City of Chehalis to this area is limited by both hydraulic capacity and GMA boundaries.

INSERT FIGURE IV-8 NAPAVINE DRINKING WATER

It is anticipated that future water service in this area will be provided by private and exempt wells since the majority of the basin is closed to additional groundwater withdrawals.

DESCRIPTION OF EXISTING WASTEWATER COLLECTION SYSTEM

The existing wastewater collection system consists of 16 pump stations and approximately 64 miles of mainline gravity sewer pipe ranging in size from 6-inches up to 27-inches in diameter. A map of the existing collection system is provided in Figures IV-9 and IV-10. The City of Chehalis, City of Napavine and LCSD No.1 each own and operate their respective portions of the collection system. The City of Napavine and LCSD No.1 wastewater collection systems were constructed in 1978 using concrete pipe with rubber gasket joints and PVC pipe. The City of Chehalis system began in 1907 using clay and concrete sewer pipe. The City of Chehalis has replaced about 80,400 feet of the old lines with new PVC pipe through I/I rehabilitation work. Currently, the entire collection system consists of about 97,700 feet of PVC and the remaining 233,700 feet is concrete, clay or other pipe materials.

There are 16 pump stations varying in size from the small North Kresky station, which is a 35 gpm submersible station, up to the largest, Prindle Street pump station, which has a peak capacity of approximately 7,500 gpm. Ten of these stations are wet wells with submersible pumps. The other six are wet well/dry well pump stations with pumps located in the dry wells. The largest station, Prindle, was constructed in 1948 and has been upgraded as recently as 1988.

The other three large wet/dry well pump stations are identical stations and were built in 1978. Those stations are Napavine, Rush Road, and Riverside pump stations.

A more comprehensive review and evaluation of the collection system and pump stations are provided in Section VI. That evaluation divides the collection system into twelve collection basins. Flows are estimated for each basin along with projected flows for the expanded service area.

INSERT FIGURE IV-9 EXISTING COLLECTION SYSTEM

INSERT FIGURE IV-10 EXISTING COLLECTION SYSTEM

DESCRIPTION OF EXISTING WASTEWATER TREATMENT SYSTEM

The WWTP was first constructed in 1949 and has undergone substantial upgrades in 1957, 1970, 1980 and 1995. Other minor improvements were implemented in 1988, 1993, and 1997. The existing WWTP site plan is shown in Figure IV-11. A schematic of the existing WWTP is provided in Figure IV-12. The following is a brief description of the major components of the WWTP.

1. All flow from the service area arrives at the plant in an 18-inch common force main from Riverside and Prindle pump stations and a small 6-inch force main and pump station serving Shoreline Drive. There is no gravity flow to the plant.

2. A Doppler meter, installed in February 1999, measures the influent flow where the force main enters the site.

3. The flow enters the top of the elevated headworks structure which consists of a grit chamber and rotating fine screen. The grit chamber removes sand and gravel and the screen removes plastic and rubber goods, rags and other larger debris. The screen (Hycor) apparatus compresses the screening and discharges directly into a trash bin for disposal. A bypass channel with manual bar screening is also provided.

4. The treatment process consists of primary clarification, trickling filter and secondary clarification. Two aeration basins are used for ammonia removal during the summer and flow equalization during the winter. There are two equal-sized primary clarifiers that are the spiraflow type. Each primary clarifier is 50-feet in diameter and has a sidewater depth of 9-feet. Both primary clarifiers are made of concrete and have a sloped floor and sludge collection rake arm assembly.

INSERT FIGURE IV-11

INSERT FIGURE IV-12

5. There are two unequally sized trickling filters. Filter No. 1 is 7-feet deep and has a diameter of 90-feet and Filter No. 2 is 6-feet deep and has a diameter of 66-feet. Both

use rock filter media.

6. After biological treatment in the trickling filters, the flow is pumped to two secondary clarifiers. The first secondary clarifier is the spiraflow type with a diameter of 65-feet and a sidewater depth of 10-feet. The newer clarifier is the center feed type with a flocculating center well and has a diameter of 65-feet and a sidewater depth of 18-feet. Both clarifiers are made of concrete and have a sloped floor and sludge collection hopper. Sludge is recirculated back to the aeration basins or wasted to the primary clarifiers.

7. During the summer, when the plant is operating in the nitrification mode, the flow leaving the trickling filters is sent to the north aeration basin prior to secondary clarification. The basin converts harmful ammonia into nitrate. The aeration basin has a volume of 0.95 MG with a sidewater depth of 9-feet. The basin is mixed and aerated with four two-speed fixed 15 Hp aerators. The second (south) aeration basin is not used in the summer. Flow leaving the extended aeration basin is pumped to the secondary clarifiers.

8. During the winter, when nitirification is not required, both aeration basins are drained and are used for influent equalization storage. Inflows in excess of about 7.5 MGD are routed to the equalization storage basins for treatment after influent flow decreases below plant capacity, which is usually a couple of days later.

9. After clarification, the wastewater flows by gravity to the chlorine contact basins for disinfection. There are three chlorine contact basins, but only two are currently used for disinfection. Chlorine Contact Basin 1 has been converted for use as a flow diversion structure. The chlorine is produced in the chlorine building by mixing chlorine gas with plant water. The gas is stored in 150-pound cylinders.

10. The disinfected effluent is dechlorinated with sulfur dioxide which is currently stored in 150-pound cylinders.

11. The dechlorinated effluent then flows by gravity to a single port outfall in the Chehalis River.

12. Primary solids and waste activated sludge (WAS) are pumped to the primary anaerobic digester, which has a capacity of 158,239 gallons. The primary anaerobic digester is mixed by recirculation pumps and heated to approximately 37°C. Digested sludge is then transferred to the secondary anaerobic digester, which is used for sludge storage and supernating. The secondary anaerobic digester has a capacity of 158,239 gallons and is neither mixed nor heated. Treated biosolids are pumped from the secondary anaerobic digester to the open sludge storage basin that has a volume of 342,000 gallons. Solids from the storage tank are pumped to sludge drying beds at approximately 8-10% solids. There are 14 sludge-drying beds that are all covered. However, four of the beds are used for storage of equipment. Total available drying bed area that is currently used is 20,000 square feet.

The City's operation and maintenance (O&M) manual provides descriptions of the various design parameters, sizes, locations, O&M and troubleshooting for the system. Design criteria for the plant is summarized in Table IV-3 and a list of plant equipment is summarized in Table IV-4.

TABLE IV-3
EXISTING WWTP DESIGN DATA

Rated Flow Capacity	
Secondary Treatment Capacity	7.5 MGD
Peak Hydraulic Capacity	13.0 MGD
Secondary Treatment Capacity w/Equalization Storage	9.3 MGD
Rated Loading Capacity	
ERUs	14,958
BOD_5	4,880 lbs/day
TSS	5,125 lbs/day
Influent Force Main	
Type, Diameter	18-inch DIP
Peak Pumping Capacity	15.4 MGD
Chlorine Contact Tanks	
No. 2 – 102,600 gallons (l:w) =	60:1
No. 3 – 89,900 gallons (l:w) =	

Total – 192,500 gallons	
Detention Time at 4 MGD	70 Minutes
Detention Time at 13 MGD (Peak Flow)	21 Minutes
Chlorination Equipment	
Number	1
Capacity	500 lbs/day
Chlorination Rate	15 mg/l
Control	Oxidation Reduction Potential (ORP)
Anaerobic Digester No.1	
Volume	158,230 gallons
Operating Temperature	37°C
Mixed	Yes
Sidewater Depth	22-Feet
Diameter	35-Feet
Anaerobic Digester No.2	
Volume	158,230 gallons
Operating Temperature	Ambient
Mixed	No
Sidewater Depth	22-Feet
Diameter	35-Feet
Sludge Storage Basin	
Volume	342,000 gallons
Sidewater Depth	10-feet
Dechlorination Equipment (Sulfur Dioxide)	
Number	1
Capacity	250 lbs/day
Control	Oxidation Reduction Potential (ORP) Paced
Plant Outfall	
Size	24-inch
Length	380-feet
Emergency Generator	
Size	90 kW
Fuel Type	Diesel
Sludge Drying Beds	
Type	Covered
Number	14
Size	20-Feet X 100-Feet
Total Area	28,000 Square Feet

TABLE IV-4
EXISTING WWTP PARAMETERS

Headworks	Primary Clarifiers
12 ft. X 12 ft. Grit Chamber 5.3 MGD Hycor Screen 7.7 MGD Parshall Flume 13+ MGD Bypass Bar Screen	No. of Tanks – 2 Sidewater Depth – 9ft. Diameter – 50 ft. Area/Tank – 1,963 sf. Weir length/tank – 342 ft. Volume/tank – 134,000 gal.

Trickling Filter No. 1 Rock Media Depth – 7 ft. Diameter – 90 ft. Area – 6,362 sf. (0.15 acres) 2 – 3.7 MGD Recirc. Pumps	**Trickling Filter No. 2** Rock Media Depth – 6 ft. Diameter – 66 ft. Area – 3,421 sf. (0.08 acres) 2 – 3.7 MGD Recirc. Pumps
Secondary Clarifier No. 1 Sidewater Depth – 10 ft. Diameter – 65 ft. Area – 3,320 sf. Weir Length – 477 ft. Volume – 270,000 gal.	**Secondary Clarifier No. 2** Sidewater Depth – 18 ft. Diameter – 65 ft. Area – 3,320 sf. Weir Length – 372 ft. Volume – 453,000 gal.
Filter Feed Pumps 2 at 2.1 MGD 1 at 4.3 MGD Total 8.5 MGD	**Aeration/Equalization Basins** No. of Basins – 2 L:W – 125 ft. X 125 ft. Sidewater Depth – 10 ft. Area/tank – 15,625 sf. Volume/tank – 955,000 gal. 2 – 1 MGD Dewatering Pumps
Secondary Clarifier Feed Pumps 2 at 5.5 MGD 1 at 2.0 MGD Total 13.0 MGD	**Aerators (each basin)** Type: Fixed Power: 15 Hp Number: 4 Hp/1,000 cf.: 0.5

CAPACITY OF EXISTING WWTP

A comprehensive capacity evaluation of the existing WWTP was prepared in 1993. This capacity evaluation took into account all of the plant upgrades and established firm plant capacities for flow, BOD_5 and TSS. The plant's firm capacity for secondary treatment is 7.5 MGD. Using the two equalization storage basins allows for a flow of 9.3 MGD to be treated to secondary standards. Flows through the plant in excess of 7.5 MGD receive only primary clarification and disinfection with chlorine. Peak hydraulic capacity is 13.0 MGD regardless of permit conditions. Table IV-5 shows the hydraulic capacity for major unit processes at the plant.

TABLE IV-5 EXISTING UNIT PROCESS HYDRAULIC CAPACITY		
Treatment Units	**Hydraulic Average (MGD)**	**Capacity Peak (MGD)**
Headworks Facilities	7.5	13.0
Equalization/Aeration Basins	N/A	13.0
Primary Clarifier Splitter Box	7.5	7.5
Primary Clarifiers	4.7	11.8
Trickling Filter Feed Pumps	7.5	7.5
Trickling Filter Distributor Arms	N/A	8.98
Secondary Clarifier Feed Pumps	7.5	13.0
Secondary Clarifier Splitter Box	7.5	13.0
Secondary Clarifiers	5.3	8.0
Chlorine Contact Tanks	5.0	15.1
Dechlorination Systems	7.5	7.5

Firm capacity of the plant under the current NPDES permit for BOD_5 and TSS is more difficult to quantify because of the variability in loading rates for each unit process shown in DOE's "Criteria for Sewage Works Design" (Orange Book). The trickling filter loading rate for BOD_5 is the most critical for determining the plant's BOD_5 capacity. According to the Orange Book, the trickling filter loading rate should be between 25 and 300 lbs/day/1,000 cf. Therefore, with the existing trickling filters, the BOD_5 capacity is from 1,026 lbs/day up to 19,518 lbs/day. Based on a low loading rate of 75 lbs/day/1,000 cf the BOD_5 capacity is 4,880 lbs/day.
A more realistic rate of 150-lbs/day/1,000 cf yields a trickling filter capacity of 9,750 lbs/day. In addition, the primary clarifiers remove an average of 24% of influent BOD_5. Therefore, the rated BOD_5 capacity of the plant is 13,000 lbs/day based on 24% removal in the primary clarifiers and a loading rate of 150-lbs/day/1,000 cf for the two trickling filters. No allowance is assumed for BOD_5 capacity in the aeration basins.

The TSS capacity of the plant according to the 1993 capacity evaluation is 5,125 lbs/day. This is mostly limited by the solids process train. Ammonia removal was not required in the previous NPDES permit and subsequently was not evaluated in the 1993 report. The WWTP has demonstrated adequate ammonia removal capacity from June through October in 1996-98 when extended aeration has been applied. However, the ammonia removal capacity for high flow conditions is minimal and is limited by aerator capacity.

DOE performed a facility inspection of the WWTP in June 2000. Mr. Dave Knight (DOE) issued a letter to the City on September 15, 2000 regarding concerns he had about plant capacity and operations as a result of the inspection. See Appendix B for the inspection letter and subsequent correspondence from Gibbs & Olson to the City, which responds to Mr. Knight's concerns.

EXISTING PLANT CONDITIONS

Data on the plant flows and performance are collected by the plant operators and recorded on report forms (DMRs) which the City submits each month to the DOE. A detailed analysis of WWTP flow is presented in Section V of this report. A summary of the plant influent loading data for BOD_5 (mg/l), BOD_5 (lbs/day), TSS (mg/l), TSS (lbs/day), ammonia (mg/l) and ammonia (lbs/day) from the beginning of April 1995 through the end of March 1998 is presented in Table IV-6. The following figures show the influent parameters.

INSERT TABLE IV-6

MONTHLY WWTP INFLUENT DATA

Influent

Figure IV-13: This graph shows the monthly average influent BOD_5 concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the overall average BOD_5 concentration to be 165 mg/l. The average wet weather (November 1 though April 30) BOD_5 concentration is 129 mg/l while the average dry weather (May 1 through October 31) BOD_5 concentration is 200 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 246 mg/l and is used for design calculations.

Figure IV-14: This graph shows the monthly average influent BOD_5 mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). Table IV-6 shows the overall average BOD_5 loading to be 2,370 lbs/day. The average wet weather (November 1 though April 30) BOD_5 loading is 2,534 lbs/day while the average dry weather (May 1 through October 31) BOD_5 loading is 2,207 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 3,264 lbs/day and is used for design calculations. Existing BOD_5 removal capacity is greater than the existing design loading condition.

Figure IV-15: This graph shows the monthly average influent TSS concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the average overall TSS concentration to be 161 mg/l. The average wet weather (November 1 though April 30) TSS concentration is 143 mg/l while the average dry weather (May 1 through October 31) BOD_5 concentration is 178 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 241 mg/l and is used for design calculations.

INSERT FIGURE IV-13 INFLUENT MONTHLY BOD_5 CONCENTRATION

INSERT FIGURE IV-14 MONTHLY AVERAGE INFLUENT BOD LOADING

INSERT FIGURE IV-15 Monthly Average Influent TSS Concentration (mg/l)

INSERT IV-16 Monthly Average Influent TSS Loading

Figure IV-16: This graph shows the monthly average influent TSS mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). Table IV-6 shows the overall average TSS loading to be 2,458 lbs/day. The average wet weather (November 1 though April 30) TSS loading is 2,896 lbs/day while the average dry weather (May 1 through October 31) TSS loading is 2,020 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 3,971 lbs/day and is used for design calculations. Existing TSS removal capacity is greater than the existing design loading condition.

Figure IV-17: This graph shows the monthly average influent ammonia (NH_3-N) concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the overall average ammonia concentration to the plant is 22.3 mg/l. The average wet weather concentration is 13.8 mg/l and the average dry weather concentration is 30.3 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 41.0 mg/l and is used for design calculations.

Figure IV-18: This graph shows the monthly average influent ammonia (NH_3-N) mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). The overall average ammonia loading to the plant is 303 lbs/day. The average wet weather ammonia loading is 274 lbs/day and the average dry weather loading is 331 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 493 lbs/day and is used for design calculations. The existing WWTP does not have sufficient aeration capacity to provide 4.6 lbs of oxygen per pound of NH_3-N. However, the WWTP has demonstrated adequate ammonia removal in demonstration testing from 1996 to present.

Effluent

Table IV-7 shows average, minimum and maximum effluent flow for each month, as well as for base flows during the month of July through September. A detailed analysis of WWTP of current and future WWTP flow is presented in Section V of this report.

INSERT FIGURE IV –17 Monthly Average Influent Ammonia Concentration (mg/l)

INSERT FIGURE IV-18 Monthly Average Influent Ammonia Loading

INSERT EFFLUENT FLOW TABLE IV-7

Figure IV-19: This graph shows the daily WWTP effluent flow versus time. All flow values are expressed as million gallons per day (MGD). The overall average outflow of the plant is 2.23 MGD. The average dry weather flow is 1.34 MGD while the average wet weather flow is 3.12 MGD. Average dry weather base flow, determined using the months of July through September, is 1.15 MGD. The plant is subject to extreme swings in flow due to high I/I. The highest recorded flow during the period was 13.77 MGD on November 10, 1995. However, this flow does not directly correspond to a specific flooding or rainfall event. The validity of this flow data point is also questionable since the maximum pumping capacity of the existing influent pump stations is approximately 13.8 MGD. WWTP journal entries also do not validate the flow event. The next highest flow during the period is 12.02 MGD, which occurred during February 1996 (the most significant flood event during the period). This flow is substantiated by WWTP journal entries and is considered the existing peak day flow to the WWTP for this report.

INSERT FIGURE IV-19 DAILY WWTP EFFLUENT FLOW (MGD)

Effluent flow data from Table IV-7 shows that the ratio of wet weather flow measured in the months of November through April to dry weather flow measured in the months May through October flow as follows:

$$\frac{\text{Average Monthly Wet Weather Flow} = 3.12 \text{ MGD}}{\text{Average Monthly Dry Weather Flow} = 1.34 \text{ MGD}} = 2.3:1$$

$$\frac{\text{Peak Daily Wet Weather Flow} = 12.0 \text{ MGD}}{\text{Peak Daily Dry Weather Flow} = 5.4 \text{ MGD}} = 2.2:1$$

$$\frac{\text{Peak Daily Flow} = 12.0 \text{ MGD}}{\text{Average Daily Flow} = 2.23 \text{ MGD}} = 5.4:1$$

The 230 percent increase in average outflow and the 220 percent increase in peak flow during wet weather for 1995 – 1997 verifies the collection system still experiences significant I/I.

Table IV-8 (three pages) shows the monthly averages and the daily maximum and minimum effluent measurements for the following parameters:

- BOD_5 (mg/l)
- BOD_5 (lbs/day)
- BOD_5 (% Removed)
- TSS (mg/l)
- TSS (lbs/day)
- TSS (% Removed)
- Ammonia (mg/l)
- Ammonia (lbs/day)
- pH (su)
- Chlorine Residual (mg/l)
- Fecal Coliform (#/100ml)

The plant's performance is further illustrated in a series of figures in the following analysis of WWTP performance.

INSERT TABLE IV-8 PAGE 1 OF 3

INSERT TABLE IV-8 PAGE 2 OF 3

INSERT TABLE IV-8 PAGE 3 OF 3

WWTP PERFORMANCE

This subsection will discuss the plant's ability to meet the various NPDES permit conditions. The NPDES permit limits are complicated because of different dry weather and wet weather limits, as well as, interim and final limits. The various permit conditions and limits are discussed in Section III of this report. The Consent Decree is the basis for both interim and final permit limits. This analysis is based on calendar-based interim limits since that is how the NPDES permit is written. This report does not evaluate the plant's ability to meet final permit limits because the existing plant requires major upgrades in order to comply with the final limits. The existing plant may also be replaced with a new treatment system. Options for treatment systems that will comply with the final limits are presented in Section VII of this report.

Graphs that show the plant's performance for different effluent parameters along with permit conditions are presented herein.

Figure IV-20: This graph shows the monthly average concentration of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as milligrams per liter (mg/l). The data show that the WWTP has met the interim dry weather limit of 20 mg/l monthly average in 17 of the 18 months (94 percent of the time). During the same time period, the plant has met the monthly wet weather interim limit of 30 mg/l in 18 out of 18 months (100 percent of the time).

Figure IV-21: This graph shows the average weekly concentration of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as mg/l. The limits are 30 mg/l for dry weather and 45 mg/l for wet weather conditions. The WWTP has met the permit limit in 72 of the 72 weeks (100 percent of the time) when wet weather limits apply. During the same time period, the plant has met the dry weather limit in 71 out of 72 weeks (99 percent of the time). The overall average BOD_5 of the plant's effluent during wet weather conditions is 16 mg/l. The overall average BOD_5 concentration discharged during dry weather conditions is 9 mg/l.

INSERT FIGURE IV-20 Monthly Average EFFLUENT BOD_5 Concentration (mg/l)

INSERT FIGURE IV-21 Weekly Average EFFLUENT BOD_5 (mg/l)

Figure IV-22: This graph shows the monthly average pounds per day of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The WWTP has met the average monthly dry weather limit of 334 lbs/day in 17 of the 18 months that dry weather limits apply (94 percent of the time). During the same time period the plant has met wet weather limit of 1,000 lbs/day in 18 out of 18 months (100 percent of the time).

Figure IV-23: This graph shows the average weekly pounds per day of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The data show that the plant has met the interim wet weather BOD_5 limit of 1,500 lbs/day in 72 out of the 72 weeks (100 percent of the time) when wet weather limits would apply. The plant has met the interim weekly dry weather limit of 500 lbs/day in 70 out of 72 weeks (97 percent of the time). The overall BOD_5 discharged from the plant during wet weather conditions is 418 lbs/day. The overall BOD_5 discharged from the plant during dry weather conditions is 114 lbs/day.

Figure IV-24: This graph shows the monthly average percent removal of BOD_5 at the WWTP for wet and dry weather conditions. The wet weather interim limit is 75% minimum removal and the dry weather limit is 85% minimum removal. The data show that the WWTP has met the wet weather removal limit in 12 of the 18 months (67 percent of the time) when wet weather limits would apply. The data also shows that the WWTP has met the interim dry weather removal limit in 17 of the 18 months (94 percent of the time) when dry weather limits would apply. The overall average BOD_5 percent removal during wet weather conditions is 82%. The overall average BOD_5 percent removal during dry weather conditions is 94%.

INSERT FIGURE IV-22 Monthly Average EFFLUENT BOD_5 (lbs/day)

INSERT FIGURE IV-23 Weekly Average EFFLUENT BOD_5 (lbs/day)

INSERT FIGURE IV - 24 BOD_5 percent removal

<u>Figure IV-25</u>: This graph shows the monthly average concentration of TSS discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed in mg/l. The wet weather interim limit is 30 mg/l and the dry weather interim limit is

25 mg/l. The data show that the WWTP has met the interim dry weather limit in 17 of the 18 months (94 percent of the time) when dry weather limits would apply. The plant has met the wet weather interim limit in 14 out of 18 months (78 percent of the time) when wet weather limits would apply.

Figure IV-26: This graph shows the average weekly concentration of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The limits are 37.5 mg/l for dry weather and 45 mg/l for wet weather conditions. The data show that the WWTP has met the dry weather limit in 70 of the 72 weeks (97 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim wet weather limit in 68 out of 72 weeks (94 percent of the time). The overall average TSS concentration discharged during wet weather conditions is 25 mg/l. The overall average TSS concentration discharged during dry weather conditions is 12 mg/l.

Figure IV-27: This graph shows the monthly average pounds per day of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The data show that the WWTP has met the interim dry weather limit of 417 lbs/day monthly average in 18 of the 18 months (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim wet weather limit of 1,000 lbs/day in 15 out of 18 months (83 percent of the time).

Figure IV-28: This graph shows the weekly average pounds per day of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim wet weather TSS limit of 1,500 lbs/day in 64 out of the 72 weeks (89 percent of the time) when wet weather limits would apply. The plant met the interim dry weather weekly average limit of 626 lbs/day in 67 out of 72 weeks (93 percent of the time). The overall TSS discharged from the plant during wet weather conditions is 668

INSERT FIGURE IV-25 Monthly TSS (mg/l)

INSERT FIGURE IV-26 Weekly TSS (mg/l)

INSERT FIGURE IV - 27 Monthly TSS (lbs/day)

INSERT FIGURE IV - 28 Weekly TSS (lbs/day)

lbs/day. The overall average TSS discharged from the plant during dry weather conditions is 155 lbs/day.

Figure IV-29: This graph shows the monthly average percent removal of TSS occurring at the WWTP for wet and dry weather conditions. The wet weather interim limit is 65% and the dry weather limit is 85% minimum removal. The data show that the WWTP has met the wet weather removal limit in 15 of the 18 months (83 percent of the time) when dry weather limits would apply. The data shows that the WWTP has met the dry weather removal limit in 17 of the 18 months (94 percent of the time) when wet weather limits would apply. The overall average TSS percent removal during wet weather conditions is 74%. The overall average TSS percent removal during dry weather conditions is 92%.

Figure IV-30: This graph shows the monthly and weekly geometric mean of the fecal coliform bacteria in the effluent. All values are expressed in number of colonies per 100 ml of effluent (#/100 ml). The interim monthly and weekly limits for wet and dry weather conditions are the same. The data show that the WWTP has met the monthly limit of 200/100 ml (geometric mean) in 36 of the 36 months (100 percent of the time). During the same time period, the plant has met the weekly limit of 400/100 ml in 141 out of 144 weeks (98 percent of the time).

Figure IV-31: This graph shows the daily effluent pH of the WWTP for both wet and dry weather conditions. All values are expressed in standard units of pH (SU). The daily interim limits for wet and dry weather conditions are the same. The interim permit requires effluent pH within the range of 6.0 to 9.0. The data show that the WWTP has had a pH within this range for 1,090 out a total of 1,095 days (99 percent of the time) where the pH was recorded. Of the 5 days where daily sample results were outside of the permitted range, all had a pH of less than 6.0. The average pH value of the effluent is 7.0.

Figure IV-32: This graph shows the monthly average and daily (5 days a week) concentration of residual chlorine in the effluent for both wet and dry weather conditions. All values are expressed as mg/l. The data show that the plant has met the wet and dry weather monthly average interim permit limits of 0.023 and 0.021 mg/l respectively, in 18 of the 18 months (100 % of the time). The plant has met the dry weather maximum daily limit of 0.023 mg/l in

INSERT FIGURE IV – 29 Percent Removal TSS (lbs/day)

INSERT FIGURE IV-30 EFFLUENT FECAL

INSERT FIGURE IV-31 EFFLUENT pH

INSERT FIGURE IV-32 Residual Chlorine

776 out of 780 samples (99 percent of the time) when dry weather limits would apply. The WWTP has met the wet weather maximum daily interim limit of 0.026 mg/l in 779 out of 780 samples (99 percent of the time) when wet weather limits would apply. The overall average chlorine residual discharged during dry weather conditions is 0.00 mg/l.

Figure IV-33: This graph shows the monthly average concentration of ammonia discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim dry weather limit of 18.6 mg/l monthly average in 18 of the 18 months (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim monthly wet weather limit of 12.9 mg/l in 16 out of 18 months (89 percent of the time).

Figure IV-34: This graph shows the daily concentration of ammonia discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim dry weather daily limit of 36.8 mg/l in 1,084 of the 1,084 daily samples (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim daily wet weather limit of 31.6 mg/l in 1,084 out of 1,084 daily samples (100 percent of the time). The overall average ammonia concentration during dry weather conditions is 2.5 mg/l. The overall average ammonia concentration during wet weather conditions is 9.5 mg/l. It should be noted the WWTP is currently not capable of performing nitrification during wet weather conditions.

Summary

The existing plant meets most conditions of the NPDES permit with good reliability. Most of the violations are for TSS compliance during wet weather, which is caused by inadequate secondary clarifier capacity. The plant also experiences violations in the fall when the aeration basin is drained so that it can be used for equalization storage. Tables IV-9 and IV-10 show a performance summary of the existing plant.

FIGURE IV-33 Monthly Ammonia Concentrations mg/l

FIGURE IV-34 Daily Ammonia Concentration

TABLE IV-9 DRY WEATHER PERFORMANCE SUMMARY			
Water Quality Parameter	Permit Condition	Interim Permit Limit	% Compliance

BOD_5 Concentration	Monthly Average Weekly Average	20 mg/l 30 mg/l	94% 99%
BOD_5 Mass Loading	Monthly Average Weekly Average % Removal	334 lb/day 500 lb/day 85%	94% 97% 94%
TSS Concentration	Monthly Average Weekly Average	25 mg/l 37.5 mg/l	94% 97%
TSS Mass Loading	Monthly Average Weekly Average % Removal	417 lb/day 626 lb/day 85%	100% 93% 94%
Ammonia Concentration	Monthly Average Daily Maximum	18.6 mg/l 36.8 mg/l	100% 100%
Fecal Coliforms	Monthly Geometric Mean Weekly Geometric Mean	200/100 ml 400/100 ml	100% 100%
pH	Daily Maximum Daily Minimum	9 6	100% 95%
Chlorine Residual	Monthly Average Daily Maximum	0.021 mg/l 0.023 mg/l	100% 99%

TABLE IV-10
WET WEATHER PERFORMANCE SUMMARY

Water Quality Parameter	Permit Condition	Permit Limit	% Compliance
BOD_5 Concentration	Monthly Average Weekly Average	30 mg/l 45 mg/l	100% 100%
BOD_5 Mass Loading	Monthly Average Weekly Average % Removal	1,000 lb/day 1,500 lb/day 75%	100% 100% 67%
TSS Concentration	Monthly Average Weekly Average	30 mg/l 45 mg/l	78% 94%
TSS Mass Loading	Monthly Average Weekly Average % Removal	1,000 lb/day 1,500 lb/day 65%	83% 89% 83%
Ammonia Concentration	Monthly Average Daily Maximum	12.9 mg/l 31.6 mg/l	89% 100%
Fecal Coliforms	Monthly Geometric Mean Weekly Geometric Mean	200/100 ml 400/100 ml	100% 98%
pH	Daily Maximum Daily Minimum	9 6	100% 100%
Chlorine Residual	Monthly Average Daily Maximum	0.023 mg/l 0.026 mg/l	100% 99%

METALS PERFORMANCE

The draft water quality analysis for metals is presented in Section III of this report. The following graphs show the plant's performance for copper, silver and zinc.

Figure IV-35: This graph shows the effluent copper concentration of the WWTP. The interim limit for copper is the same for both wet and dry conditions and is based on a daily maximum. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (µg/l). The data show that the WWTP has met the interim limit of 53.5 µg/l daily maximum in 29 out of 29 months (100 percent of the time) where the copper concentration was recorded.

Figure IV-36: This graph shows the monthly effluent silver concentration of the WWTP. The interim limit for silver is the same for both wet and dry conditions and is based on a daily maximum and yearly average. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (µg/l). The data show that the WWTP has met the interim limit of 28.2 µg/l daily maximum in 29 out of 29 months (100 percent of the time) where the silver concentration was recorded. The plant has met the yearly average limit of 13.5 µg/l in 3 of 3 years. The overall effluent silver concentration is 3.08 µg/l and the 99th percentile value is 11.5 µg/l.

Figure IV-37: This graph shows the monthly effluent zinc concentration of the WWTP. The interim limit for zinc is the same for both wet and dry conditions and is based on a daily maximum. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (µg/l). The data show that the WWTP has met the interim limit of 119.6 µg/l daily maximum in 28 out of 29 months (97 percent of the time) where the zinc concentration was recorded. The overall effluent zinc concentration is 75.6 µg/l and the 99th percentile value is 136 µg/l.

INSERT FIGURE IV-35 EFFLUENT COPPER

INSERT FIGURE IV-36 EFFLUENT SILVER

INSERT FIGURE IV-37 EFFLUENT ZINC

BIOSOLIDS (SLUDGE) TREATMENT AND DISPOSAL

The plant has both primary and secondary clarifiers. The primary clarifier sludge is wasted to the primary anaerobic digester for stabilization. Sludge is removed from the secondary clarifiers and pumped back to the primary clarifiers where it commingles with the primary sludge. The activated sludge is either returned through the treatment process (RAS) or wasted to the primary anaerobic digester (WAS). Sludge is sent to the digester at approximately 3 to 5% solids concentration. The solids are treated in two anaerobic digesters, which are operated in a series. The first anaerobic digester is heated and mixed. After a detention time of approximately 59 days, the sludge is transferred to the other anaerobic digester that is not heated or mixed. After another 59 days, the treated solids are sent to a sludge storage basin for thickening prior to being pumped to covered drying beds for dewatering. The thickened sludge has a solids concentration of 8 to 10%. The dried biosolids are trucked to eastern Washington where they are utilized for agricultural land application.

SEWER SYSTEM OPERATION AND MAINTENANCE COSTS

Each entity maintains separate budgets for operation and maintenance of their wastewater facilities. The budget for the regional WWTP is maintained by the City of Chehalis and costs are reimbursed to the City by Napavine and LCSD No. 1 for their proportional WWTP expenses. Using recent City expenditures, a budget for operation and maintenance of both the plant and the collection system were developed. Table IV-11 provide a line item breakdown of expenditures for the WWTP. The current cost to operate the WWTP is $1,100,680 annually. This does not include any costs associated with collection system improvements and maintenance of the collection system.

TABLE IV-11 WWTP Expenses (1998)*	
Inflation Rate Applied to Cost 3.00%	**Estimated WWTP Cost**
Salaries and Wages (S&W)	$ 407,000
Personal Benefits	$ 142,500
Office & Operating Supplies	$ 53,000
Professional Services	$ 8,000
Uniforms & Clothing	$ 6,000
Communications	$ 12,000
Travel	$ 300
Rentals and Leases	$ 1,000
Advertising	$ 100
Insurance	$ 5,000
Public Utility Service	$ 58,000
Small Tools & Minor Equipment	$ 1,000
Maintenance & Repairs	$ 36,000
Machinery & Equipment	$ 33,000
Fuel	$ 4,000
Miscellaneous	$ 11,000
Taxes	$ 12,000
Interfund Supplies	$ 200
Interfund Repairs & Maintenance	$ 1,000
Sewer System Reserve Fund	$ 43,000
Existing Debt Service	$ 110,900
TMDL Related Costs	$ 500,000
Total Expenditures	**$1,445,000.00**

* Does not include O&M costs for Chehalis, Napavine and LCSD No. 1 Collection System.

The municipal codes for the Cities of Chehalis and Napavine establish authority to charge for sewer service. LCSD No.1's authority to charge for service is established in Title 57 of the Resource Code of Washington (RCW). Rates and charges are established independently for each entity by the elected bodies through ordinances and resolutions. The rates and charges for each entity vary significantly as shown in Tables IV-12 through Table IV-17.

TABLE IV-12 CITY OF CHEHALIS CONNECTION CHARGES	
Type of Service	**Connection Charge**
All Customer Types	$2,991 (1999)
Existing Line Surcharge	$1,000 (if not previously contributed to line)
Airport Area Surcharge	$1,452 (1999)

TABLE IV-13 CITY OF NAPAVINE CONNECTION CHARGES		
Type of Service	**Connection Charge**	
Residential and Commercial	Inside City	Outside City
▪ ¾" meter	$3,500	$4,500 plus ERU Charge $750
Industrial and/or >2" meter	Determined by the Director and Council	
Multiple Units	$1,000 for each additional unit 2-20 $3,500 and $4,500 for the 21st, $1,000 for 22-40, etc.	

*Increase basic charge by $100 per year

TABLE IV-14 LEWIS COUNTY SEWER DISTRICT NO. 1 CONNECTION CHARGES (1999)			
Type of Service	**Connection Charge**		
Residential	ULID No.1	ULID No.2	Outside District
▪ Existing Stub or lot	$1,200	$3,750	Not Applicable
▪ New Lot*	$4,000	$12,000	$5,000
Multiple Units **	$1,000 each	$1,500	$1,500
Commercial & Industrial	Determined by the Commissioners		

* Not originally part of ULID No.1 or ULID No. 2.
** In addition to the basic charge for one residential.

TABLE IV-15 CITY OF CHEHALIS SERVICE CHARGES (1999-2001)*			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Single family	$25.02/($37.53)	$3.07/100 c.f.	$55.72/($68.23)
▪ Low Income/disabled	$17.89/($26.84)	$3.07/100 c.f.	$48.59/($57.54)
Commercial (per unit)	$25.02/mo.	$3.07/100 c.f.	$55.72
Industrial	$3,225 (1MG)	$0.26/lb of BOD $0.38/lb of TSS	Not Applicable

* Service charges increase by approximately 3% in 2002 and 2003

TABLE IV-16 CITY OF NAPAVINE SERVICE CHARGES			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Inside City	$26.00/mo.	$1.50/100 c.f. >300 c.f.	$36.50
▪ Outside City	$31.00/mo.	$1.75/100 c.f. >300 c.f.	$43.25
Schools	$25.00/mo.	$3.00/100 c.f. >300 c.f.	$46.00
Churches	$26.00/mo.	$1.50/100 c.f. >300 c.f.	$36.50
Commercial/Industrial			
▪ Inside City	$26.00/mo	$3.00/100 c.f. >300 c.f.	$47.00
▪ Outside City	$50.00/mo.	$3.75/100 c.f. >300 c.f.	$76.25

TABLE IV-17 LEWIS COUNTY SEWER DISTRICT NO. 1 SERVICE CHARGES			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Inside District	$18.00/mo.	Not Applicable	$12.00
▪ Outside District	$22.00/mo.	Not Applicable	$16.00
Trailer courts			
▪ Inside District	$3.00/mo./pad	$75.00/dump site/mo.	Not Applicable
▪ Outside District	$4.00/mo./pad	$90.00/dump site/mo.	Not Applicable
Nursing and Rest Homes			
▪ Inside District	$30.00/mo.	$5.00/bed/mo.	Not Applicable
▪ Outside District	$40.00/mo.	$6.00/bed/mo.	Not Applicable
Office, Daycare, etc.			
▪ Inside District	$40.00/mo.	Or $0.04/s.f. if greater	Not Applicable
▪ Outside District	$50.00/mo.	Or $0.05/s.f. if greater	Not Applicable
Restaurant, Café, etc.			
▪ Inside District	$60.00/mo.	$0.60/seat	Not Applicable
▪ Outside District	$75.00/mo.	$0.75/seat	Not Applicable

SEWER USE ORDINANCE

Sewer Use Ordinances are contained in Chapter 13.08 of both Cities Municipal Codes. All regulations pertaining to the use and charge for the sewer system are contained in this Chapter. The City of Chehalis sewer use ordinance is very thorough and covers the following topics:

- Connection policies.
- Construction standards for sewers and side sewers.
- Conditions on prohibition of specific discharges.
- Pretreatment standards.
- Administrative policy.
- Compliance and enforcement.

The sewer use ordinance serves the City well and no additions or amendments are anticipated or recommended. City of Napavine and LCSD No. 1 Sewer Use Ordinances and policies are not as comprehensive, but have serviced the respective utilities well in the past. Service area policy details and policy recommendations will be completed in the Facilities Plan.

SECTION IV

CONDITIONS IN THE PLANNING AREA

INTRODUCTION

The planning area is located in the western portion of Lewis County, Washington along the Interstate 5 corridor from the City of Chehalis to south of the City of Napavine. A vicinity map is provided as Figure IV-1.

Three entities, the City of Chehalis, the City of Napavine and LCSD No.1 are responsible for operation and maintenance of respective collection system components. Collection system policy, planning and financing is determined for each individual entity through the respective commissions and councils. All three entities share in funding improvements to the Wastewater Treatment Plant (WWTP) according to capacity ownership percentage. WWTP O&M costs are tracked separately and shared by each entity according to use.

The City of Chehalis is the primary agency responsible for planning, financing, operating and maintaining the Regional WWTP. The City of Chehalis coordinates Regional WWTP policy and planning decisions with Napavine and LCSD No.1 through the Chehalis Regional Sewer Operating Board (CRSOB). The CRSOB is comprised of one elected official from the each entity. Current representatives on the CRSOB are as follows:

- Mayor Robert Spahr (Chairman), City of Chehalis
- Jim Haslett, Councilman, City of Napavine
- Chuck Weiland, Commissioner, Lewis County Sewer District No.1

INSERT FIGURE IV-1 VICINITY MAP

SEWER INTERCEPTOR AGREEMENT

The rights and responsibilities for the CRSOB members regarding collection and treatment of sanitary sewage are contained in the Sewer Interceptor Agreement (SIA), which is included as Appendix B. The current SIA was established on June 22, 1994 and is an update to the original SIA established in June of 1976, and is valid through June of 2004. If the SIA is not updated on, or after that date, the SIA will continue on a year to year basis until updated.

The SIA establishes the City of Chehalis as the lead entity responsible for treatment and disposal for all sewage in the planning area. The City of Chehalis establishes rates for collection and treatment of sanitary sewage. Proposed rates must be presented to the CRSOB for review and comment prior to enactment by the Chehalis City Council. The City of Napavine and LCSD No.1 must prepare an annual report to the City of Chehalis with the number and classification of all sewer connections. The CRSOB is designated as the responsible entity for addressing any necessary dispute resolution related to SIA issues.

Interceptor line and WWTP capacity ownership is established by Exhibit A of the SIA and is shown as Figure IV-2. Equivalent Residential Units (ERUs) are used to establish a uniform basis of relative capacity allotted to each entity. Average ERU capacity established in the SIA for use in Exhibit A is 250 gallons per ERU per day. Average ERU capacity represents an estimate of 2.5 persons per average single-family residential household contributing 100 gallons per person per day. A factor of 2.5 is utilized to establish peak hydraulic flow per ERU (not including I/I). It should be noted that ERU values are system and time specific and may not correspond exactly to specific wastewater flows from each entity on a regional basis. However, if applied uniformly, the ERU value creates a basis for fair and reasonable cost sharing on a regional basis. The process to update the proportion of interceptor and treatment capacities for each entity will begin after completion of the GSP and Facility Plans. The financial analysis in Section VIII of this report presents several cost sharing options based on projected population of each entity.

INSERT FIGURE IV-2 INTERCEPTOR LINE AND WWTP CAPACITY OWNERSHIP

SEWER SERVICE AREA

Sewer service areas are shown in Figure IV-3. The existing service area is comprised of incorporated City limits, District boundaries and areas currently served by sewers within unincorporated areas.

The 2025 future service area shown in Figure IV-3 is approximately 8,700 acres. The future service area represents the Interim Urban Growth Management Areas (IUGA) for each City with minor adjustments to account for anticipated growth in LCSD No.1 outside of the City IUGAs. The IUGAs were established through County Ordinance on May 4, 1998 in accordance with the planning process under the Growth Management Act (GMA). The Cities and the County have developed comprehensive plans in conformance with GMA that establish Urban Growth Areas (UGAs) and future land use. The UGAs and any amendments to the UGAs will continue to represent the future service area (i.e., as the UGAs change, so will the future service area).

The City of Napavine updated their comprehensive plan in August of 1998 and is in compliance with GMA. The Chehalis Comprehensive Plan was adopted in July 1999. The Lewis County Comprehensible Plan is currently being appealed.

The ultimate service area identified in Figure IV-3 is approximately 12,200 acres and represents areas that may be served by the WWTP, but are not expected to contribute to WWTP flows until after 2025. Additional discussion regarding future service areas and GMA are included in Section V.

The current (1997) population within the planning area is estimated at 8,671 persons. Population projections for the 2025 service area are presented in Section V. Population projections for the ultimate service area are not presented in this report. The ultimate service
area is presented specifically for use in Section VI to determine future collection system flows.

INSERT FIGURE IV-3 SERVICE AREA MAP

ECONOMICS

Since the early 1970's, unemployment in Lewis County has been consistently higher than the statewide average (Washington State Employment Security). Employment growth in Lewis County has slightly exceeded population growth in recent years by 0.17% per year (Hovee). The continuing transition of the resource based economy along with more recent welfare to work programs, will assert additional pressure for job creation throughout the County. With the implementation of the GMA in Lewis County and lack of comprehensive utility service in unincorporated areas, the Chehalis-Centralia area will likely support a majority of the Lewis County job growth to adequately meet the needs of the anticipated population growth. Due to the relatively large amount of industrial land in Chehalis and Napavine UGAs compared to Centralia, industrial development in the County will likely occur in the planning area to meet County needs for family wage jobs.

LAND USE

Currently, the County, as well as the Cities of Chehalis and Napavine designate land use within their respective jurisdictions. As stated above, each jurisdiction must prepare comprehensive plans in conformance with GMA. The comprehensive plans for the Cities will designate future land use for their respective UGAs. Land use designations are shown in Figure IV-4 and includes only general land use classifications for the purpose of this report. Some of the designated land uses may change as final GMA plans are completed, but these changes are not anticipated to have a significant affect on projected sewer service levels.

Land use within the City of Chehalis corporate limits is regulated by the City of Chehalis Municipal Code Title 17 ("Zoning"). Within that title, Chehalis has twenty-one land use

INSERT FIGURE IV-4 LAND USE

designations. These "regular" zones include: single-family residential, (low and medium density), multiple-family (medium-density), multiple-family (high density), four forms of commercial, light and heavy industrial, and ten forms of essential public facilities. The City of Napavine has similar land use regulations, but fewer land use designations.

Land use outside the City limits is governed by Lewis County, which at this time employs no land use zoning. Current "county" land uses within the service area include commercial, industrial, agricultural and rural residential. There are also a few residential subdivisions with densities close to that of urban residential areas. Table IV-1 shows acreage within the 2025 and ultimate service area boundaries for general land use classifications. All of the commercial/industrial designation has been included in the industrial classification in this table.

TABLE IV-1
LAND USE ACREAGE

TYPE	2025 ACREAGE	ULTIMATE ACREAGE
Residential	3,000	5,600
Commercial	1,300	1,800
Industrial	2,000	2,200
Areas not suitable for development	2,400	2,600
Total	**8,700**	**12,200**

OTHER SERVICES

In the planning area, utilities are provided by a combination of city managed, state regulated, federally licensed and municipally franchised providers. In Chehalis, city managed utilities are sewer, water, solid waste and stormwater. The remaining non-city managed utilities are cable television (AT&T), electrical (Lewis County PUD), natural gas (Puget Sound Energy) and telephone and cellular (Qwest Communications). In Napavine, the city managed utilities are water and sewer. Napavine receives the same non-city managed utility services as Chehalis other than natural gas. LCSD No.1 manages sewer service only. A majority of water service in LCSD No. 1 is provided by the City of Chehalis. Electrical and gas utilities within the planning area are regulated by the Washington Utilities and Transportation Commission (WUTC) while the telephone and cellular telephone services are federally licensed. Cable television services are provided under municipal franchises through AT&T.

The 1990 GMA requires all comprehensive plans to contain a Utilities Element that includes the general location, proposed location and capacity of all existing and proposed utilities (RCW36.a.070- (4)). The utility element for Napavine, Chehalis, and LCSD No. 1 (through the County plan) has been completed.

TRANSPORTATION

Interstate 5 extends north/south throughout the service area. The planning area is served by five on/off ramps with one in the immediate vicinity of the WWTP. Railroad service is available to Seattle, Portland and Grays Harbor. Railroad service to Willapa Harbor was recently discontinued. The planning area is served by the Chehalis/Centralia Airport. The Federal Aviation Administration (FAA) publishes guidelines regarding minimum setbacks for facilities and water bodies from airports. The proximity of the airport can have negative ramifications for siting equalization basins or constructed wetlands in the vicinity of the WWTP. However, the airport is already limited by the fact that it is completely surrounded by wetlands and oxbow lakes. There are also height restrictions for structures built within the airport flight path.

PHYSICAL CONDITIONS

TOPOGRAPHY

The topography of the study area is characterized by the flood plain of the Chehalis and Newaukum Rivers and the adjacent low river terraces. Flood plain areas are flat or gently sloping. River terraces form very subdued stair-steps in the terrain which branch upward to the southeast and laterally away from the main stem of the Chehalis and Newaukum and toward the southwesterly fringes of the planning area, and are adjacent to the upper reaches of creeks flowing into the Chehalis and Newaukum Rivers. The gradients of the major stream tributaries increase greatly toward their headwaters, as is typical in Western Washington. Low-lying areas range in elevation from approximately 160-feet to 300-feet Mean Sea Level (MSL). Higher areas range in elevation from approximately 300-feet to 500-feet MSL.

SOILS

Soils within the drainage basin of the existing wastewater collection system are principally flood plain deposits or gravel, sand and silt. Another large portion is occupied by glaciofluvial sand and gravel in matrix of clay and silt, which forms a well-defined terrace along the Chehalis and Newaukum Rivers. Both these soils yield large supplies of groundwater.

The low, flat flood plain areas are primarily made up of recent alluvium soils or silt layers deposited by the River. A majority of these soils are poorly drained and are subject to unfavorable high water table causing swampy conditions in wet seasons as well as fall and winter flooding.

CLIMATE

Climate in the planning area is moderate with cool, dry summers and wet, moist and cloudy winters. Average daily temperatures vary from 45 degrees in January to 78 in July. Average annual precipitation is 47 inches. Average rainfall during the period of May through October is 25 percent of the total yearly precipitation. In contrast, average rainfall from July through September is 10 percent of the total. Freezing weather seldom continues for more than a few days before warmer, moist air from the ocean moves inland.

AIR QUALITY

The Environmental Protection Agency (EPA) and DOE set regulations for air quality. The EPA has established National Ambient Air Quality Standards (NAAQS) for six "criteria pollutants": carbon monoxide, particulate matter, nitrogen oxides, sulfur dioxide, ozone and lead. The Southwest Clean Air Authority (SCAA) is responsible for regulation and monitoring of air pollution in Lewis County.

The prevailing direction of wind around the planning area is south or southwesterly during the wet winter season and northwesterly during the dry summer season. The strongest winds are generally southeasterly to southwesterly and associated with more intense winter systems. During the winter, automobiles tend to produce more carbon monoxide, and home heating produces both particulate matter and carbon monoxide, especially when wood is used as a fuel. Conversely, breezes are generally stronger during spring and summer when less carbon monoxide and particulates are produced.

In the Chehalis area, the main sources of air pollutants are automobiles, wood stoves, road dust and industrial emissions (SCAA, 1994). Industrial air pollutant sources in Chehalis and its vicinity include Centralia Steam Plant, Coast Millwork Company, Kinnear of Washington, Lakeside Industries, Hardel Mutual Plywood and Northwest Hardwoods. All six operations are well below national EPA standards. The largest emission source in the State of Washington is a coal fired power plant operated by Pacific Corp to the northeast of the planning area. All of these point sources are regulated by 5-year operating permits issued and tracked by both SCCA and EPA.

SURFACE WATERS

The major surface water resources in the planning area are the Chehalis River and its tributary, the Newaukum River.

Newaukum River

The Newaukum River is a typical, medium-sized western river flowing generally northwest and entering the Chehalis River about three-quarters of a mile west of the City of Chehalis at Corps of Engineers Chehalis River Mile (RM) 75.4. About eleven river miles upstream from its mouth, the Newaukum forks into two branches; North Fork and South Fork. The Newaukum River drains approximately 155 square miles, with an average discharge of approximately 500 cubic feet per second (cfs). The City of Chehalis diverts approximately 5 cfs for municipal use from the North Fork of the river, approximately 17 miles from the confluence with the Chehalis River. Along the river there are various small diversions made for agricultural and domestic uses.

The TMDL Study reported that flow in the Newaukum River ranged from 27 to 72 cfs during the study period, and made up about one-half of the flow below the confluence of the Chehalis River. DO and pH were mostly within water quality standards during the study with one DO measurement just below 8.0 mg/l in an early morning sample. Temperatures exceeded 18°C in three of the six measurements taken.

Chehalis River

The main stem of the Chehalis River is over 100 river miles and covers a drainage area of approximately 1300 square miles. The Chehalis River is a relatively shallow and swift-moving stream. However, near RM 74, a section of the river called the "Centralia Reach" deepens, and stream velocities decrease substantially. Throughout most of this reach, the Chehalis River is confined to a deeply cut, meandering channel averaging about 50-feet wide. The Centralia Reach is characterized as having intermittent deep pools up to 30-feet deep. During low-flow periods, stream velocities as low as 2 to 3 miles per day are common (DOE, 1984). Below the mouth of the Skookumchuck River, near RM 67, the Chehalis River becomes wider and shallower. River velocities are much higher below RM 64.

WAC 173-522-020 specifies base flows for the Chehalis River basin. During the TMDL Study, flow was measured at twelve control stations on the main stem Chehalis River and its tributaries. Of the twelve control stations, only three (Cedar Creek, Salzer Creek and the Skookumchuck River) were referenced as having flows higher than base flows during August 1992.

The Chehalis River supports a diverse variety of aquatic life. Important salmon runs include spring, fall and summer chinook, coho and chum. Salmon are present within the Chehalis River on a year-round basis. The portion of the Chehalis River downstream of the WWTP is not a prime spawning area, but does serve as a transport zone for both spawning and downstream returning salmon. Riverbanks in this area are commonly lined with deciduous trees and/or brush. The Chehalis River Basin is not glacially fed, although snowmelt makes a minor contribution to flows in the upper Newaukum River watershed (CH2M Hill, 1998).

Water quality problems have been identified in the Chehalis River basin for at least 30 years. The TMDL Study lists general causes of water pollution which include municipal and industrial WWTP effluents, septic tank effluent, urban development and storm runoff, stream bank degradation, poor domestic livestock management, forest practices and pesticide usage (agriculture). Pollutant sources identified in the study area are discussed throughout the TMDL Study.

The Centralia Reach of the Chehalis River was characterized in the TMDL Study as having numerous stratified areas during the summer months in locations with deep pools. Temperatures at the surface in these stratified areas were very high during July and August. The deep waters of the stratified areas were cooler, but were mostly found to be anoxic, especially from RM 71.0 downstream. Some of the stratified areas showed evidence of water quality degradation from local pollutant inputs, in particular: at sites north of the Chehalis/Centralia Airport (RM 70.7) and below Salzer Creek (RM 69.1).

Other Tributaries

There are two tributary creeks, Dillenbaugh Creek and Salzer Creek that are suspected to have an adverse impact on water quality within the Centralia Reach. Salzer Creek was characterized in the TMDL Study as having the worst water quality of any tributary in the upper Chehalis River basin. Dillenbaugh Creek enters the Chehalis River at approximately RM 74.5 just upstream of the WWTP. Salzer Creek enters the Chehalis River at approximately RM 69.3. Salzer and Dillenbaugh Creeks contributed approximately 2% and 1% of the total low flow within the Chehalis River, respectively during the TMDL Study.

Temperatures in the tributaries were identified above the 18°C criterion on several occasions. The most likely cause of increased water temperature identified is loss of riparian canopy vegetation. Restoration of the riparian canopy on these tributaries would likely reduce water temperatures. Dillenbaugh Creek was reported to have extensive wetlands near the mouth that may produce low DO by natural processes. All the creeks with low DO in the TMDL Study have current livestock impacts.

FLOOD PLAINS

Flood levels for rivers and streams within the planning area are available in the November 1979 Flood Insurance Study published by the Federal Emergency Management Agency (FEMA).

The entire WWTP site is located in the Chehalis River flood plain in the proximity of River Mile 74.3. The westerly portion of the site is within the FEMA designated floodway which is shown in Figure IV-5. Flood stage levels for the WWTP site, as documented in the November 1979 Flood Insurance Study, are shown in Table IV-2. Flood stage levels for a 25-year event in Table IV-2 are interpolated from the FEMA data for the purpose of addressing DOE design standards.

INSERT FIGURE IV-5 EXISTING WWTP SITE DESIGNATED FLOODWAY

TABLE IV-2 1979 FLOOD STAGE LEVELS IN THE VICINITY OF THE WWTP	
Flood Event	Chehalis River Elevation (MSL) at River Mile 74.3 Approximate streambed elevation = 138-feet
500-year	181.0-feet
100-year	179.0-feet
50-year	178.5-feet
25-year	178.0-feet
10-year	177.0-feet

FEMA is in the process of updating floodway and flood plain maps. The updated 100-year flood levels in the vicinity of the WWTP may be as high as 179.5, which corresponds to the flood of record (February 1996). A floodplain map by Pacific

International Engineering (PIE) based on the February 1996 flood of record for the planning area is included in Appendix B.

Lewis County, through a contract with PIE, is currently developing flood-stage modeling and mitigation proposals to lower the Chehalis River flood stage. PIE completed a draft report in October of 1998, which outlines mitigation alternatives that will reduce flood stage levels in the Centralia/Chehalis area. Effects from the proposed mitigation efforts identified in the report may result in a 1 to 2-foot reduction in the flood stage at the current WWTP site. Design and construction of any WWTP improvements should consider the FEMA update, as well as, the information from the County flood mitigation study.

The City has recently completed a FEMA financed program to purchase property within the floodplain and floodway near the WWTP site. These properties could be used by the wastewater utility for nuisance abatement, floodway offsets and future facilities. However, these properties cannot be used for any permanent facilities which would prevent them from passing floodwaters (i.e.: buildings, diked basins. etc).

Figure IV-6 identifies the hazard mitigation program area and property currently under City ownership in the vicinity of the WWTP.

The original facilities at the WWTP were constructed in 1948, decades prior to development of the DOE design standards and the most recent flood stage levels. Ground elevations at the WWTP site range from a high of approximately 178-feet along the southeast dike to a low of approximately 173-feet along the frontage road to the north. Ground elevations for the majority of the plant are approximately 175 to 176-feet. Consequently, many of the original facilities did not meet the basic intent of the DOE guidelines. In recognition of this, all existing mechanical and electrical equipment at the plant has been raised above the current 100-year flood stage. Most of the plant functions can be adequately controlled during a 25-year flood event. However, the trickling filters and aeration basins are subject to over-topping depending on the severity of the flood event.

Past flooding at the WWTP site would typically begin as the stormwater drainage system backed up into the plant. Since the installation of pinch valves on the plant drainage system, flooding is delayed and reportedly starts on the northeast side, which is consistent with the relatively low ground elevations. The WWTP and/or Shoreline Drive have flooded at least seven times since December 1989, according to WWTP records. Flood impacts have ranged from creating minor access problems to structural damage of facilities.

During a recent flood event on December 30, 1996, flooding at the WWTP site damaged the concrete walls of the north aeration basin. The flood stage, as measured at monitoring station (MS) #4 (near the WWTP), peaked at 177.8-feet. This event was in excess of the estimated 25-year flood stage, but far below the 100-year flood stage and many other floods that have occurred.

INSERT FIGURE IV-6 HAZARD MITIGATION PROGRAM AREA

The highest recorded flood event in the past 70 years occurred on February 8, 1996 when the recorded flood stage at MS #4 was 180.2-feet. No noticeable damage occurred to the WWTP facilities, but this and previous flooding may have been a contributing factor to the damage during the subsequent event on December 30, 1996.

GROUNDWATER

The major geological formations of the planning area are the glacial outwash of the Pleistocene Age and the alluvial terrace deposits of late tertiary to Quaternary Age. The alluvial terrace deposits are the most predominant of the planning area. Alluvial terrace deposits generally occur as a yellow-gray to yellow-brown heterogeneous mixture of gravel and sand with lesser amounts of silt and clay. Lenses of sand or clay are common, as well as, lenses of till. The thickness of the alluvial terrace deposits exceed 150-feet and are thin towards the foothills. Extensive weathering of the upper 20 to 40-feet of the formation has reduced the permeability in some areas, whereas unweathered gravel in the lower parts produces yields of approximately 200 gallons per minute (GPM) in a few wells. Widely spread, thick clay and silt sections yield little water. Water levels are generally less than 400-feet below the surface.

Groundwater yields on the river terraces are usually small and often contain an objectionable amount of iron. In spite of these objections, this source has been extensively developed for domestic and agricultural purposes because of its accessibility.

A large portion of the Newaukum River Basin contains an artesian aquifer capable of providing moderate to large quantities of water of a reasonably high quality. Artesian water is obtained by tapping the down-folded tertiary rock. The sources of this water are on the hills to the north and south of the valley where these water-bearing strata are near the surface. Here rainwater flows down permeable strata, flushing out the saline water normally found at this level and providing water under pressure in the valleys.

The TMDL Study referenced several previous studies which estimated groundwater inflows from above Bunker Creek (RM 86.0) to Prather Road (RM 59.9). Average inflow rates ranged from 0.5 cubic feet per second (cfs)/mile at the upstream end of this area to 4.5 cfs/mile near the mouth of Lincoln Creek (RM 64.2 to 62.0). The TMDL Study concluded that groundwater inputs to the main stem may constitute up to one-third of the low flow reaching the Mellen Street Bridge (RM 67.5).

Future increases in irrigation needs must be met almost entirely from groundwater sources since much of the surface water and shallow groundwater is over appropriated by DOE estimates. Because of this, it is anticipated that water reclamation and reuse may become more prevalent in the future. Records of public water supplies within the vicinity of the WWTP were obtained from the Washington State Department of Health (DOH). From this information, there are no public water supply wells immediately downstream of the WWTP and only two public water supply wells within approximately 3 miles upstream of the WWTP along the Chehalis River.

WETLANDS AND SHORELINES

Wetlands

Because a majority of the planning area is located in a wide, flat valley with very minimal slope variation, the community is bordered on the west by several small to midsize wetlands. Wetlands support regular large concentrations of wintering migratory waterfowl, fish and other wetland species.

The approximate location of known wetlands has been inventoried and mapped by the United States Department of the Interior's National Wetlands Inventory. Wetland locations are available from Washington State Department of Fish and Wildlife's (WSDFW) Public Data Release Maps. The City of Chehalis and Napavine have adopted these wetland maps and use them for guideline locations when a development proposal is submitted. The City of Chehalis had made attempts in the past to verify wetlands and update their maps accordingly.

Shorelines

Streams within the study area that are subject to the Washington State Shoreline Management Act of 1971 include the Chehalis River, Newaukum River, Salzer Creek and Dillenbaugh Creek. In addition, the shoreline of the Chehalis River is designated as Shoreline of Statewide Significance. Activities within the shorelines of these waterways are guided by the regulations contained in the Chehalis Shoreline Master Program (SMP).

The SMP contains policies and regulations that specify permitted land uses within these shoreline areas and afford protection to these areas based on the designated shoreline environments. One policy of note reads as follows: "Sewage treatment, water reclamation and power plants should be located where they do not interfere with other public uses of the water and shoreline."

FISH AND WILDLIFE, THREATENED AND ENDANGERED SPECIES

The Chehalis River Basin contains approximately 3,353 miles of stream habitat, providing a complex and diverse ecosystem. WSDFW has been contacted to provide a list of state and federally listed and proposed threatened and endangered species, candidate species and species of concern that may be present within the area of the proposed sewer service area. The planning area has a regular concentration of bald eagles, which are a state and federal listed threatened species. Other listed species that have been identified to have habitats in the area include osprey, wild turkey and the Olympic mudminnow. Spawning and rearing areas within the basin support several economically viable species of anadromous fish including chinook salmon (*Oncorhynchus tshawytscha*), coho salmon (*O. kisutch*), chum salmon (*O. keta*), steelhead trout (*O. mykiss*), cutthroat trout (*O. clarki*) and Dolly Varden char (*Salvelinus malma*).

Early findings described in a report titled *"Chehalis River Basin Fishery Resources: Status, Trends and Restoration Goals"* (USFWS, 1993) and additional reports from USFWS and the Western Washington Treaty Indian Tribes show fish populations have declined as a result of pulp mill effluents, increased temperature and/or low DO, dams and diversions, domestic animal practices, forest practices, agriculture, urbanization and industrialization, gravel mining, sedimentation and excessive commercial fishing. However, with the exception of winter steelhead in the Skookumchuck and Newaukum Rivers, fish stocks in the Chehalis River system are considered healthy.

HISTORICAL AND CULTURAL RESOURCES

Archaeological Sites

Prehistoric use of the Chehalis River Valley by Native American people was high, due to the abundance of salmon and other resources in the area. A number of archaeological sites have been discovered in the study area during excavations for construction projects, although few systematic surveys have been undertaken. Investigations of known sites have yielded valued assemblages of artifacts dating back as far as the Olcott Phase (4,000-7,000 years before present). The general likelihood of archaeological resources being present is high throughout the study area. If any construction activities encounter archeological finds, construction will need to be suspended and the State Office of Archeology notified.

Prime and Unique Farmlands

Proposed land use for the planning area does not include farmland. However, there are areas within the planning area boundaries that have soil conditions that are designated prime farmland soils by the Natural Resource Conservation Services located mainly in the rich alluvial soils adjacent to the Chehalis and Newaukum Rivers. The soils in these areas are comprised primarily of Newberg fine sandy loam, with lesser amounts of Chehalis silty clay and Cloquato silt loam. There are no designated agricultural resource lands in the planning area by the City or County under the provisions of the GMA.

ADJACENT WASTEWATER FACILITIES

The City of Centralia WWTP is located approximately 2 miles north of the northern boundary of the planning area and approximately 3.5 miles north of the Chehalis Regional WWTP. The Centralia WWTP provides secondary treatment up to approximately 7.5 MGD as presented in the City's 1998 Facilities Plan (CH2M Hill, 1998). The 1998 Facilities Plan recommends replacement of the existing Centralia WWTP at one of three sites to the north of the existing WWTP. Options for utilizing the new Centralia WWTP as a regional facility for Centralia and Chehalis are discussed in Section VII of this report.

DESCRIPTION OF EXISTING WATER SYSTEMS

A majority of the water service in the planning area is provided by the City of Chehalis and City of Napavine municipal water systems. There are numerous small water systems and private wells which provide water service to a relatively small percentage of the population located between the two cities. The majority of the smaller systems are located in the proximity of LCSD No.1. Future service areas for the Chehalis and Napavine water systems correspond to each City's UGA.

CITY OF CHEHALIS WATER SYSTEM

The City of Chehalis water system currently provides service to approximately 3,160 service connections, of which, approximately 900 are outside city limits. The service connections outside the city limits are primarily along Jackson Highway and the North Fork Road. Figure IV-7 shows the major components of the City of Chehalis water system. The City of Chehalis currently has two sources of supply, one providing water from the North Fork of the Newaukum River and the other from the Chehalis River.

INSERT FIGURE IV-7 CHEHALIS DRINKING WATER

The North Fork of the Newaukum River Source

This supply system includes intake facilities and equipment consisting of a bar screen, traveling screen, turbidity monitoring and chlorination equipment, standby power and approximately 17.5 miles of raw water transmission line. The intake site is situated approximately 17 miles from the city, approximately 10 miles east of Jackson Highway, in Section 20, Township 14 North, Range 1 East, W.M. The watershed of the intake encompasses an area of about 18 square miles predominately owned by the Weyerhaeuser Company.

A majority of the 16-inch transmission line from the intake to the Henderson Park pump station line was replaced in 1977 with ductile iron pipe. The cast iron portion of
the line is believed to be in acceptable condition. Until recent years the intake operations were conducted jointly by the cities of Centralia and Chehalis and operational costs were shared by both cities. After provisions of the Federal Safe Drinking Water Act prohibited the City of Centralia from using "filtered water" from this source, they reluctantly curtailed their operations in 1993, and in 1994 were forced to abandon the Newaukum as an unfiltered supply source.

Now that Centralia is no longer using this source, it appears that Chehalis would be entitled to withdraw a much greater quantity of available water (2.8 MGD based on the City's initial 1912 right and 6.46 MGD based on the City's 1923 right). Even though this quantity of water appears to be sufficient to satisfy the 2015 peak day demand, the probable least mean monthly flow at the intake has been estimated in previous studies to be as low as 5.2 MGD.

Chehalis River Source

The Chehalis River pump station and intake were constructed on the east bank of the Chehalis River near Riverside Road at approximately Chehalis river mile (RM) 75 in 1961-62. The intake is a 10-foot square wooden crib with a layer of 6-inch rocks in the walls to act as a screen. A 48-inch diameter corrugated metal pipe extends from the intake crib approximately 50-feet to the pump station wet well located on the riverbank.

The wet well is a reinforced concrete structure, 19-feet in diameter. The station originally was equipped with one 100 horsepower (Hp) and one 150 Hp vertical turbine pump. In 1993, a third pumping unit (150 Hp) was added. An automatically cleaned traveling screen is housed in the pump station and screens the water before it enters the wet well.

The three pumps discharge into a common 18-inch steel transmission line that extends approximately 8,000-feet to the water treatment plant. The present capacity of the facility is 5.04 MGD (7.8 cubic feet per second), which is more than the projected 2015 peak day needs. The electrical service and controls were replaced within the past four years and even more recently controls were replaced and relocated above the record flood level. The existing 18-inch line from the pump station to the filter plant has a capacity of 15 cfs, which is the City's water right permit instantaneous limit.

Just like the North Fork source, the Chehalis River source faces vulnerabilities related to forest practices that take place in the upper watershed of the river and its tributaries. This source also faces potential problems related to agricultural and dairy activities that take place upstream of the intake.

It is anticipated that Chehalis will be required to use the Chehalis River intake to augment flows from the North Fork and make up differences in peak day demands beyond those that can be currently supplied (1.76 MGD in 2015). This source also provides a backup in the event of a failure or problem with the North Fork supply, in which case it would provide the entire water supply to the City.

Centralia – Chehalis Intertie

The Cities of Centralia and Chehalis have constructed an emergency intertie, connecting the two cities' water systems. The intertie is currently un-metered, but has two valves, one operated by each city. Operation requires cooperation and specific action by both cities. The purpose of this system is to provide each city with a source of water, although limited, from the other's water system, during emergency conditions.

WATER RIGHTS

Washington Water Law

The surface water code of the State of Washington, Chapter 90.03 RCW, was enacted in 1917. Before that, water rights could be established under the common law. These older water rights are often termed common law, or "vested" water rights. Common law water rights are of two types: riparian rights and appropriative rights. Riparian water rights must be used upon lands that are adjacent to the water body from which the water is withdrawn. The common law appropriation doctrine sanctions withdrawing water and using it at distant locations.

North Fork of the Newaukum River

The City of Chehalis initiated a common law appropriation of water from the Newaukum in 1912. Centralia later applied for water rights. After a series of disputes, a State Supreme Court decision in 1954 decreed that the City of Chehalis has the right to the first 2.8 MGD of flow in the river at the intake. The City of Centralia, which has ceased withdrawing water from the North Fork source, has (and may still have) a subsequent right to the next 4.8 MGD. The Court also ruled that the City of Chehalis had the right to all water in excess of 7.6 MGD. The City also has an additional certificate for 10 cfs (6.46 MGD) vested through a water right permit certificated in 1923. These water rights are sufficient to supply the projected 2015 peak day needs, however, the transmission main system cannot currently deliver the entire quantity to the water treatment plant.

Chehalis River

The City of Chehalis holds a water right permit to withdraw up to 15 cfs from the Chehalis River, dating back to 1957. The permit contains a 50 cfs minimum flow provision. The permit contained an initial completion date of May 1, 1962, which has been extended a number of times. In 1996, the City requested that the permit be certificated, based on projected 20-year use projections that included providing a proposed power generating facility with raw water. Since DOE saw the issue of supplying the raw water to the proposed power facility as an outstanding (but uncertain) factor that could significantly influence the 20-year projections, DOE elected to extend the permit for ten years, until May 1, 2006.

Water Treatment Plant

The treatment plant was constructed in 1960-61 and its components include a flash mixing chamber where coagulant is added and mixed, two slow mixing chambers (in series), a presettling basin and two (parallel) settling basins, two rapid sand filters (also in parallel) and a clearwell. The water surface elevation (maximum) at the treatment plant is 415.7-feet. Raw water from the North Fork and/or the Chehalis River may be fed into the plant. The plant provides coagulation, flocculation, sedimentation, filtration, disinfection, pH adjustment/control and fluoridation. Aluminum chloride hydroxide is the primary coagulant that is currently used at the plant. Lime is used to provide pH adjustment and the plant generally maintains a finished water pH of 7.2 to 7.4.

Post-treatment disinfection is accomplished with chlorine gas applied to provide a residual concentration ranging from 0.4 to 2.0 parts per million (ppm) in the water distribution system, with a distribution system average of approximately 1.0 ppm.

Three certified operators staff and operate the facility. They also conduct water quality monitoring, inspecting and testing throughout the water system. The plant operates 24 hours per day and is staffed at least eight hours per day during the workweek.

During the past six years, the plant has been upgraded to include a streaming current detector that provides extremely responsive coagulant feed rates that are automatically varied as the demand dictates and an emergency backup generator, which provides essential electric power during emergencies and outages. The water treatment plant currently provides finished water with a turbidity typically ranging from 0.03 to 0.09 nephelometric turbidity unit (NTUs).

Based on the maximum filter rate of 2.5 gpm per square foot of surface area (as established by DOH criteria), the water treatment plant's current capacity is 4.8 MGD or 3,360 gpm. Although the plant has a listed capacity of 4.84 MGD, the effective "operating" capacity is actually closer to 4.0 MGD. This difference is due to down time for filter backwashes and operational flow reductions that are required. Chemicals on hand include liquid aluminum chloride hydroxide, lime, fluoride, liquid chlorine, filter aid polymers and various laboratory chemicals and reagents.

Storage

The storage facilities for the main pressure zone (low-level system) consist of two reservoirs: a 5 MG reservoir located adjacent to the water treatment plant; and a 1 MG reservoir located south of the current city limits. The upper-level system is served by a 100,000-gallon reservoir. The Valley View pressure zone distribution system is served by two 67,000-gallon reservoirs (a total of 134,000 gallons).

The primary reservoirs (Main and Kennicott Reservoirs) serving the main zone have sufficient capacities to meet demands beyond the projected 2015 required levels. The other reservoirs, however, cannot meet current demand including fire flow needs, and improvements will be made as part of the City's Capital Improvement Plan (CIP). An additional 100,000-gallon reservoir will be constructed to augment the existing High Level Reservoir. In order to address the potential fire flow needs at the north end of the city, a 500,000-gallon reservoir will also be constructed.

Water Demand and Conservation

Water use in the Chehalis system is metered at several locations prior to treatment, post-treatment as the water enters the distribution system and through individual water meters. The total amount of raw water has been broken down into its components of plant operational use and loss, total entering the reservoir (distribution system), system operational losses and total demand. These water qualities were evaluated and this information was used to develop water consumption, use projections and peak need quantity forecasts in the City's 1997 Water System Plan (WSP). The 1997 WSP shows total residential water consumed divided by the number of residential connections results in an equivalent residential unit (ERU) of 183 gpd or approximately 65 gpd per capita at current housing densities.

Water use projections shown in the WSP assume water production per ERU can be reduced by 2.5 percent in 20 years by incorporating the programs presented in the City's Water Conservation Plan. Beyond use reductions that result from rate increases, the conservation plan assumes that strong public awareness and utilization of low-volume plumbing fixtures, and implementation of uniform water rates will result in long-term reduction beyond the already low demand per ERU.

CITY OF NAPAVINE WATER SYSTEM

The City of Napavine water system provides service to approximately 450 service connections, of which, approximately 10 are outside of the city limits. An elevated 100,000 gallon steel reservoir constructed in the early 1970's provides storage and system pressure. A 350,000-gallon at grade reservoir and booster pump station was recently constructed. The following lists the City’s wells and capacities:

Well No. 1: capped
Well No. 2: 80 gpm
Well No. 3: 35 gpm
Well No. 4: 110 gpm
Well No. 5: 90 gpm

Figure IV-8 shows the City of Napavine’s water system facilities.

Water quality has historically been excellent other than recent coliform problems. The system is completely metered. Leakage is not currently a problem as recent records indicate an average of more than 85% of water pumped is accounted for in metered consumption. The distribution system is generally adequate for domestic flows, however, most areas do not have adequate fire flows. The City is currently preparing a WSP update to address storage, hydraulic and capacity difficulties. Water use per capita reported in previous planning documents was 113 gpcd, which is projected to be approximately 300 gpd per ERU at current population densities.

LEWIS COUNTY SEWER DISTRICT NO. 1 WATER SERVICE

Water service within the LCSD No.1 is provided through a combination of private wells, small public water systems and extensions from the City of Chehalis water system. Further service from the City of Chehalis to this area is limited by both hydraulic capacity and GMA boundaries.

INSERT FIGURE IV-8 NAPAVINE DRINKING WATER

It is anticipated that future water service in this area will be provided by private and exempt wells since the majority of the basin is closed to additional groundwater withdrawals.

DESCRIPTION OF EXISTING WASTEWATER COLLECTION SYSTEM

The existing wastewater collection system consists of 16 pump stations and approximately 64 miles of mainline gravity sewer pipe ranging in size from 6-inches up to 27-inches in diameter. A map of the existing collection system is provided in Figures IV-9 and IV-10. The City of Chehalis, City of Napavine and LCSD No.1 each own and operate their respective portions of the collection system. The City of Napavine and LCSD No.1 wastewater collection systems were constructed in 1978 using concrete pipe with rubber gasket joints and PVC pipe. The City of Chehalis system began in 1907 using clay and concrete sewer pipe. The City of Chehalis has replaced about 80,400 feet of the old lines with new PVC pipe through I/I rehabilitation work. Currently, the entire collection system consists of about 97,700 feet of PVC and the remaining 233,700 feet is concrete, clay or other pipe materials.

There are 16 pump stations varying in size from the small North Kresky station, which is a 35 gpm submersible station, up to the largest, Prindle Street pump station, which has a peak capacity of approximately 7,500 gpm. Ten of these stations are wet wells with submersible pumps. The other six are wet well/dry well pump stations with pumps located in the dry wells. The largest station, Prindle, was constructed in 1948 and has been upgraded as recently as 1988.

The other three large wet/dry well pump stations are identical stations and were built in 1978. Those stations are Napavine, Rush Road, and Riverside pump stations.

A more comprehensive review and evaluation of the collection system and pump stations are provided in Section VI. That evaluation divides the collection system into twelve collection basins. Flows are estimated for each basin along with projected flows for the expanded service area.

INSERT FIGURE IV-9 EXISTING COLLECTION SYSTEM

INSERT FIGURE IV-10 EXISTING COLLECTION SYSTEM

DESCRIPTION OF EXISTING WASTEWATER TREATMENT SYSTEM

The WWTP was first constructed in 1949 and has undergone substantial upgrades in 1957, 1970, 1980 and 1995. Other minor improvements were implemented in 1988, 1993, and 1997. The existing WWTP site plan is shown in Figure IV-11. A schematic of the existing WWTP is provided in Figure IV-12. The following is a brief description of the major components of the WWTP.

1. All flow from the service area arrives at the plant in an 18-inch common force main from Riverside and Prindle pump stations and a small 6-inch force main and pump station serving Shoreline Drive. There is no gravity flow to the plant.

2. A Doppler meter, installed in February 1999, measures the influent flow where the force main enters the site.

3. The flow enters the top of the elevated headworks structure which consists of a grit chamber and rotating fine screen. The grit chamber removes sand and gravel and the screen removes plastic and rubber goods, rags and other larger debris. The screen (Hycor) apparatus compresses the screening and discharges directly into a trash bin for disposal. A bypass channel with manual bar screening is also provided.

4. The treatment process consists of primary clarification, trickling filter and secondary clarification. Two aeration basins are used for ammonia removal during the summer and flow equalization during the winter. There are two equal-sized primary clarifiers that are the spiraflow type. Each primary clarifier is 50-feet in diameter and has a sidewater depth of 9-feet. Both primary clarifiers are made of concrete and have a sloped floor and sludge collection rake arm assembly.

INSERT FIGURE IV-11

INSERT FIGURE IV-12

5. There are two unequally sized trickling filters. Filter No. 1 is 7-feet deep and has a diameter of 90-feet and Filter No. 2 is 6-feet deep and has a diameter of 66-feet. Both

use rock filter media.

6. After biological treatment in the trickling filters, the flow is pumped to two secondary clarifiers. The first secondary clarifier is the spiraflow type with a diameter of 65-feet and a sidewater depth of 10-feet. The newer clarifier is the center feed type with a flocculating center well and has a diameter of 65-feet and a sidewater depth of 18-feet. Both clarifiers are made of concrete and have a sloped floor and sludge collection hopper. Sludge is recirculated back to the aeration basins or wasted to the primary clarifiers.

7. During the summer, when the plant is operating in the nitrification mode, the flow leaving the trickling filters is sent to the north aeration basin prior to secondary clarification. The basin converts harmful ammonia into nitrate. The aeration basin has a volume of 0.95 MG with a sidewater depth of 9-feet. The basin is mixed and aerated with four two-speed fixed 15 Hp aerators. The second (south) aeration basin is not used in the summer. Flow leaving the extended aeration basin is pumped to the secondary clarifiers.

8. During the winter, when nitirification is not required, both aeration basins are drained and are used for influent equalization storage. Inflows in excess of about 7.5 MGD are routed to the equalization storage basins for treatment after influent flow decreases below plant capacity, which is usually a couple of days later.

9. After clarification, the wastewater flows by gravity to the chlorine contact basins for disinfection. There are three chlorine contact basins, but only two are currently used for disinfection. Chlorine Contact Basin 1 has been converted for use as a flow diversion structure. The chlorine is produced in the chlorine building by mixing chlorine gas with plant water. The gas is stored in 150-pound cylinders.

10. The disinfected effluent is dechlorinated with sulfur dioxide which is currently stored in 150-pound cylinders.

11. The dechlorinated effluent then flows by gravity to a single port outfall in the Chehalis River.

12. Primary solids and waste activated sludge (WAS) are pumped to the primary anaerobic digester, which has a capacity of 158,239 gallons. The primary anaerobic digester is mixed by recirculation pumps and heated to approximately 37°C. Digested sludge is then transferred to the secondary anaerobic digester, which is used for sludge storage and supernating. The secondary anaerobic digester has a capacity of 158,239 gallons and is neither mixed nor heated. Treated biosolids are pumped from the secondary anaerobic digester to the open sludge storage basin that has a volume of 342,000 gallons. Solids from the storage tank are pumped to sludge drying beds at approximately 8-10% solids. There are 14 sludge-drying beds that are all covered. However, four of the beds are used for storage of equipment. Total available drying bed area that is currently used is 20,000 square feet.

The City's operation and maintenance (O&M) manual provides descriptions of the various design parameters, sizes, locations, O&M and troubleshooting for the system. Design criteria for the plant is summarized in Table IV-3 and a list of plant equipment is summarized in Table IV-4.

TABLE IV-3
EXISTING WWTP DESIGN DATA

Rated Flow Capacity	
Secondary Treatment Capacity	7.5 MGD
Peak Hydraulic Capacity	13.0 MGD
Secondary Treatment Capacity w/Equalization Storage	9.3 MGD
Rated Loading Capacity	
ERUs	14,958
BOD_5	4,880 lbs/day
TSS	5,125 lbs/day
Influent Force Main	
Type, Diameter	18-inch DIP
Peak Pumping Capacity	15.4 MGD
Chlorine Contact Tanks	
No. 2 – 102,600 gallons (l:w) =	60:1
No. 3 – 89,900 gallons (l:w) =	

Total – 192,500 gallons	
Detention Time at 4 MGD	70 Minutes
Detention Time at 13 MGD (Peak Flow)	21 Minutes
Chlorination Equipment	
Number	1
Capacity	500 lbs/day
Chlorination Rate	15 mg/l
Control	Oxidation Reduction Potential (ORP)
Anaerobic Digester No.1	
Volume	158,230 gallons
Operating Temperature	37°C
Mixed	Yes
Sidewater Depth	22-Feet
Diameter	35-Feet
Anaerobic Digester No.2	
Volume	158,230 gallons
Operating Temperature	Ambient
Mixed	No
Sidewater Depth	22-Feet
Diameter	35-Feet
Sludge Storage Basin	
Volume	342,000 gallons
Sidewater Depth	10-feet
Dechlorination Equipment (Sulfur Dioxide)	
Number	1
Capacity	250 lbs/day
Control	Oxidation Reduction Potential (ORP) Paced
Plant Outfall	
Size	24-inch
Length	380-feet
Emergency Generator	
Size	90 kW
Fuel Type	Diesel
Sludge Drying Beds	
Type	Covered
Number	14
Size	20-Feet X 100-Feet
Total Area	28,000 Square Feet

TABLE IV-4
EXISTING WWTP PARAMETERS

Headworks	Primary Clarifiers
12 ft. X 12 ft. Grit Chamber 5.3 MGD Hycor Screen 7.7 MGD Parshall Flume 13+ MGD Bypass Bar Screen	No. of Tanks – 2 Sidewater Depth – 9ft. Diameter – 50 ft. Area/Tank – 1,963 sf. Weir length/tank – 342 ft. Volume/tank – 134,000 gal.

Trickling Filter No. 1 Rock Media Depth – 7 ft. Diameter – 90 ft. Area – 6,362 sf. (0.15 acres) 2 – 3.7 MGD Recirc. Pumps	**Trickling Filter No. 2** Rock Media Depth – 6 ft. Diameter – 66 ft. Area – 3,421 sf. (0.08 acres) 2 – 3.7 MGD Recirc. Pumps
Secondary Clarifier No. 1 Sidewater Depth – 10 ft. Diameter – 65 ft. Area – 3,320 sf. Weir Length – 477 ft. Volume – 270,000 gal.	**Secondary Clarifier No. 2** Sidewater Depth – 18 ft. Diameter – 65 ft. Area – 3,320 sf. Weir Length – 372 ft. Volume – 453,000 gal.
Filter Feed Pumps 2 at 2.1 MGD 1 at 4.3 MGD Total 8.5 MGD	**Aeration/Equalization Basins** No. of Basins – 2 L:W – 125 ft. X 125 ft. Sidewater Depth – 10 ft. Area/tank – 15,625 sf. Volume/tank – 955,000 gal. 2 – 1 MGD Dewatering Pumps
Secondary Clarifier Feed Pumps 2 at 5.5 MGD 1 at 2.0 MGD Total 13.0 MGD	**Aerators (each basin)** Type: Fixed Power: 15 Hp Number: 4 Hp/1,000 cf.: 0.5

CAPACITY OF EXISTING WWTP

A comprehensive capacity evaluation of the existing WWTP was prepared in 1993. This capacity evaluation took into account all of the plant upgrades and established firm plant capacities for flow, BOD_5 and TSS. The plant's firm capacity for secondary treatment is 7.5 MGD. Using the two equalization storage basins allows for a flow of 9.3 MGD to be treated to secondary standards. Flows through the plant in excess of 7.5 MGD receive only primary clarification and disinfection with chlorine. Peak hydraulic capacity is 13.0 MGD regardless of permit conditions. Table IV-5 shows the hydraulic capacity for major unit processes at the plant.

TABLE IV-5 EXISTING UNIT PROCESS HYDRAULIC CAPACITY		
Treatment Units	**Hydraulic Average (MGD)**	**Capacity Peak (MGD)**
Headworks Facilities	7.5	13.0
Equalization/Aeration Basins	N/A	13.0
Primary Clarifier Splitter Box	7.5	7.5
Primary Clarifiers	4.7	11.8
Trickling Filter Feed Pumps	7.5	7.5
Trickling Filter Distributor Arms	N/A	8.98
Secondary Clarifier Feed Pumps	7.5	13.0
Secondary Clarifier Splitter Box	7.5	13.0
Secondary Clarifiers	5.3	8.0
Chlorine Contact Tanks	5.0	15.1
Dechlorination Systems	7.5	7.5

Firm capacity of the plant under the current NPDES permit for BOD_5 and TSS is more difficult to quantify because of the variability in loading rates for each unit process shown in DOE’s "Criteria for Sewage Works Design" (Orange Book). The trickling filter loading rate for BOD_5 is the most critical for determining the plant's BOD_5 capacity. According to the Orange Book, the trickling filter loading rate should be between 25 and 300 lbs/day/1,000 cf. Therefore, with the existing trickling filters, the BOD_5 capacity is from 1,026 lbs/day up to 19,518 lbs/day. Based on a low loading rate of 75 lbs/day/1,000 cf the BOD_5 capacity is 4,880 lbs/day.
A more realistic rate of 150-lbs/day/1,000 cf yields a trickling filter capacity of 9,750 lbs/day. In addition, the primary clarifiers remove an average of 24% of influent BOD_5. Therefore, the rated BOD_5 capacity of the plant is 13,000 lbs/day based on 24% removal in the primary clarifiers and a loading rate of 150-lbs/day/1,000 cf for the two trickling filters. No allowance is assumed for BOD_5 capacity in the aeration basins.

The TSS capacity of the plant according to the 1993 capacity evaluation is 5,125 lbs/day. This is mostly limited by the solids process train. Ammonia removal was not required in the previous NPDES permit and subsequently was not evaluated in the 1993 report. The WWTP has demonstrated adequate ammonia removal capacity from June through October in 1996-98 when extended aeration has been applied. However, the ammonia removal capacity for high flow conditions is minimal and is limited by aerator capacity.

DOE performed a facility inspection of the WWTP in June 2000. Mr. Dave Knight (DOE) issued a letter to the City on September 15, 2000 regarding concerns he had about plant capacity and operations as a result of the inspection. See Appendix B for the inspection letter and subsequent correspondence from Gibbs & Olson to the City, which responds to Mr. Knight's concerns.

EXISTING PLANT CONDITIONS

Data on the plant flows and performance are collected by the plant operators and recorded on report forms (DMRs) which the City submits each month to the DOE. A detailed analysis of WWTP flow is presented in Section V of this report. A summary of the plant influent loading data for BOD_5 (mg/l), BOD_5 (lbs/day), TSS (mg/l), TSS (lbs/day), ammonia (mg/l) and ammonia (lbs/day) from the beginning of April 1995 through the end of March 1998 is presented in Table IV-6. The following figures show the influent parameters.

INSERT TABLE IV-6

MONTHLY WWTP INFLUENT DATA

Influent

Figure IV-13: This graph shows the monthly average influent BOD_5 concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the overall average BOD_5 concentration to be 165 mg/l. The average wet weather (November 1 though April 30) BOD_5 concentration is 129 mg/l while the average dry weather (May 1 through October 31) BOD_5 concentration is 200 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 246 mg/l and is used for design calculations.

Figure IV-14: This graph shows the monthly average influent BOD_5 mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). Table IV-6 shows the overall average BOD_5 loading to be 2,370 lbs/day. The average wet weather (November 1 though April 30) BOD_5 loading is 2,534 lbs/day while the average dry weather (May 1 through October 31) BOD_5 loading is 2,207 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 3,264 lbs/day and is used for design calculations. Existing BOD_5 removal capacity is greater than the existing design loading condition.

Figure IV-15: This graph shows the monthly average influent TSS concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the average overall TSS concentration to be 161 mg/l. The average wet weather (November 1 though April 30) TSS concentration is 143 mg/l while the average dry weather (May 1 through October 31) BOD_5 concentration is 178 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 241 mg/l and is used for design calculations.

INSERT FIGURE IV-13 INFLUENT MONTHLY BOD_5 CONCENTRATION

INSERT FIGURE IV-14 MONTHLY AVERAGE INFLUENT BOD LOADING

INSERT FIGURE IV-15 Monthly Average Influent TSS Concentration (mg/l)

INSERT IV-16 Monthly Average Influent TSS Loading

Figure IV-16: This graph shows the monthly average influent TSS mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). Table IV-6 shows the overall average TSS loading to be 2,458 lbs/day. The average wet weather (November 1 though April 30) TSS loading is 2,896 lbs/day while the average dry weather (May 1 through October 31) TSS loading is 2,020 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 3,971 lbs/day and is used for design calculations. Existing TSS removal capacity is greater than the existing design loading condition.

Figure IV-17: This graph shows the monthly average influent ammonia (NH_3-N) concentration to the WWTP. All values are expressed as milligrams per liter (mg/l). Table IV-6 shows the overall average ammonia concentration to the plant is 22.3 mg/l. The average wet weather concentration is 13.8 mg/l and the average dry weather concentration is 30.3 mg/l. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 41.0 mg/l and is used for design calculations.

Figure IV-18: This graph shows the monthly average influent ammonia (NH_3-N) mass loading to the WWTP. All values are expressed as pounds per day (lbs/day). The overall average ammonia loading to the plant is 303 lbs/day. The average wet weather ammonia loading is 274 lbs/day and the average dry weather loading is 331 lbs/day. For purposes of this report, the 90th percentile value (maximum monthly average) of the entire data set is 493 lbs/day and is used for design calculations. The existing WWTP does not have sufficient aeration capacity to provide 4.6 lbs of oxygen per pound of NH_3-N. However, the WWTP has demonstrated adequate ammonia removal in demonstration testing from 1996 to present.

Effluent

Table IV-7 shows average, minimum and maximum effluent flow for each month, as well as for base flows during the month of July through September. A detailed analysis of WWTP of current and future WWTP flow is presented in Section V of this report.

INSERT FIGURE IV –17 Monthly Average Influent Ammonia Concentration (mg/l)

INSERT FIGURE IV-18 Monthly Average Influent Ammonia Loading

INSERT EFFLUENT FLOW TABLE IV-7

Figure IV-19: This graph shows the daily WWTP effluent flow versus time. All flow values are expressed as million gallons per day (MGD). The overall average outflow of the plant is 2.23 MGD. The average dry weather flow is 1.34 MGD while the average wet weather flow is 3.12 MGD. Average dry weather base flow, determined using the months of July through September, is 1.15 MGD. The plant is subject to extreme swings in flow due to high I/I. The highest recorded flow during the period was 13.77 MGD on November 10, 1995. However, this flow does not directly correspond to a specific flooding or rainfall event. The validity of this flow data point is also questionable since the maximum pumping capacity of the existing influent pump stations is approximately 13.8 MGD. WWTP journal entries also do not validate the flow event. The next highest flow during the period is 12.02 MGD, which occurred during February 1996 (the most significant flood event during the period). This flow is substantiated by WWTP journal entries and is considered the existing peak day flow to the WWTP for this report.

INSERT FIGURE IV-19 DAILY WWTP EFFLUENT FLOW (MGD)

Effluent flow data from Table IV-7 shows that the ratio of wet weather flow measured in the months of November through April to dry weather flow measured in the months May through October flow as follows:

$$\frac{\text{Average Monthly Wet Weather Flow} = 3.12\text{ MGD}}{\text{Average Monthly Dry Weather Flow} = 1.34\text{ MGD}} = 2.3:1$$

$$\frac{\text{Peak Daily Wet Weather Flow} = 12.0\text{ MGD}}{\text{Peak Daily Dry Weather Flow} = 5.4\text{ MGD}} = 2.2:1$$

$$\frac{\text{Peak Daily Flow} = 12.0\text{ MGD}}{\text{Average Daily Flow} = 2.23\text{ MGD}} = 5.4:1$$

The 230 percent increase in average outflow and the 220 percent increase in peak flow during wet weather for 1995 – 1997 verifies the collection system still experiences significant I/I.

Table IV-8 (three pages) shows the monthly averages and the daily maximum and minimum effluent measurements for the following parameters:

- BOD_5 (mg/l)
- BOD_5 (lbs/day)
- BOD_5 (% Removed)
- TSS (mg/l)
- TSS (lbs/day)
- TSS (% Removed)
- Ammonia (mg/l)
- Ammonia (lbs/day)
- pH (su)
- Chlorine Residual (mg/l)
- Fecal Coliform (#/100ml)

The plant's performance is further illustrated in a series of figures in the following analysis of WWTP performance.

INSERT TABLE IV-8 PAGE 1 OF 3

INSERT TABLE IV-8 PAGE 2 OF 3

INSERT TABLE IV-8 PAGE 3 OF 3

WWTP PERFORMANCE

This subsection will discuss the plant's ability to meet the various NPDES permit conditions. The NPDES permit limits are complicated because of different dry weather and wet weather limits, as well as, interim and final limits. The various permit conditions and limits are discussed in Section III of this report. The Consent Decree is the basis for both interim and final permit limits. This analysis is based on calendar-based interim limits since that is how the NPDES permit is written. This report does not evaluate the plant's ability to meet final permit limits because the existing plant requires major upgrades in order to comply with the final limits. The existing plant may also be replaced with a new treatment system. Options for treatment systems that will comply with the final limits are presented in Section VII of this report.

Graphs that show the plant's performance for different effluent parameters along with permit conditions are presented herein.

Figure IV-20: This graph shows the monthly average concentration of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as milligrams per liter (mg/l). The data show that the WWTP has met the interim dry weather limit of 20 mg/l monthly average in 17 of the 18 months (94 percent of the time). During the same time period, the plant has met the monthly wet weather interim limit of 30 mg/l in 18 out of 18 months (100 percent of the time).

Figure IV-21: This graph shows the average weekly concentration of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as mg/l. The limits are 30 mg/l for dry weather and 45 mg/l for wet weather conditions. The WWTP has met the permit limit in 72 of the 72 weeks (100 percent of the time) when wet weather limits apply. During the same time period, the plant has met the dry weather limit in 71 out of 72 weeks (99 percent of the time). The overall average BOD_5 of the plant's effluent during wet weather conditions is 16 mg/l. The overall average BOD_5 concentration discharged during dry weather conditions is 9 mg/l.

INSERT FIGURE IV-20 Monthly Average EFFLUENT BOD_5 Concentration (mg/l)

INSERT FIGURE IV-21 Weekly Average EFFLUENT BOD_5 (mg/l)

Figure IV-22: This graph shows the monthly average pounds per day of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The WWTP has met the average monthly dry weather limit of 334 lbs/day in 17 of the 18 months that dry weather limits apply (94 percent of the time). During the same time period the plant has met wet weather limit of 1,000 lbs/day in 18 out of 18 months (100 percent of the time).

Figure IV-23: This graph shows the average weekly pounds per day of BOD_5 discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The data show that the plant has met the interim wet weather BOD_5 limit of 1,500 lbs/day in 72 out of the 72 weeks (100 percent of the time) when wet weather limits would apply. The plant has met the interim weekly dry weather limit of 500 lbs/day in 70 out of 72 weeks (97 percent of the time). The overall BOD_5 discharged from the plant during wet weather conditions is 418 lbs/day. The overall BOD_5 discharged from the plant during dry weather conditions is 114 lbs/day.

Figure IV-24: This graph shows the monthly average percent removal of BOD_5 at the WWTP for wet and dry weather conditions. The wet weather interim limit is 75% minimum removal and the dry weather limit is 85% minimum removal. The data show that the WWTP has met the wet weather removal limit in 12 of the 18 months (67 percent of the time) when wet weather limits would apply. The data also shows that the WWTP has met the interim dry weather removal limit in 17 of the 18 months (94 percent of the time) when dry weather limits would apply. The overall average BOD_5 percent removal during wet weather conditions is 82%. The overall average BOD_5 percent removal during dry weather conditions is 94%.

INSERT FIGURE IV-22 Monthly Average EFFLUENT BOD_5 (lbs/day)

INSERT FIGURE IV-23 Weekly Average EFFLUENT BOD_5 (lbs/day)

INSERT FIGURE IV - 24 BOD_5 percent removal

Figure IV-25: This graph shows the monthly average concentration of TSS discharged from the WWTP, along with the interim permit limits for wet and dry weather conditions. All values are expressed in mg/l. The wet weather interim limit is 30 mg/l and the dry weather interim limit is

25 mg/l. The data show that the WWTP has met the interim dry weather limit in 17 of the 18 months (94 percent of the time) when dry weather limits would apply. The plant has met the wet weather interim limit in 14 out of 18 months (78 percent of the time) when wet weather limits would apply.

Figure IV-26: This graph shows the average weekly concentration of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The limits are 37.5 mg/l for dry weather and 45 mg/l for wet weather conditions. The data show that the WWTP has met the dry weather limit in 70 of the 72 weeks (97 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim wet weather limit in 68 out of 72 weeks (94 percent of the time). The overall average TSS concentration discharged during wet weather conditions is 25 mg/l. The overall average TSS concentration discharged during dry weather conditions is 12 mg/l.

Figure IV-27: This graph shows the monthly average pounds per day of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. All values are expressed as pounds per day (lbs/day). The data show that the WWTP has met the interim dry weather limit of 417 lbs/day monthly average in 18 of the 18 months (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim wet weather limit of 1,000 lbs/day in 15 out of 18 months (83 percent of the time).

Figure IV-28: This graph shows the weekly average pounds per day of TSS discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim wet weather TSS limit of 1,500 lbs/day in 64 out of the 72 weeks (89 percent of the time) when wet weather limits would apply. The plant met the interim dry weather weekly average limit of 626 lbs/day in 67 out of 72 weeks (93 percent of the time). The overall TSS discharged from the plant during wet weather conditions is 668

INSERT FIGURE IV-25 Monthly TSS (mg/l)

INSERT FIGURE IV-26 Weekly TSS (mg/l)

INSERT FIGURE IV - 27 Monthly TSS (lbs/day)

INSERT FIGURE IV - 28 Weekly TSS (lbs/day)

lbs/day. The overall average TSS discharged from the plant during dry weather conditions is 155 lbs/day.

Figure IV-29: This graph shows the monthly average percent removal of TSS occurring at the WWTP for wet and dry weather conditions. The wet weather interim limit is 65% and the dry weather limit is 85% minimum removal. The data show that the WWTP has met the wet weather removal limit in 15 of the 18 months (83 percent of the time) when dry weather limits would apply. The data shows that the WWTP has met the dry weather removal limit in 17 of the 18 months (94 percent of the time) when wet weather limits would apply. The overall average TSS percent removal during wet weather conditions is 74%. The overall average TSS percent removal during dry weather conditions is 92%.

Figure IV-30: This graph shows the monthly and weekly geometric mean of the fecal coliform bacteria in the effluent. All values are expressed in number of colonies per 100 ml of effluent (#/100 ml). The interim monthly and weekly limits for wet and dry weather conditions are the same. The data show that the WWTP has met the monthly limit of 200/100 ml (geometric mean) in 36 of the 36 months (100 percent of the time). During the same time period, the plant has met the weekly limit of 400/100 ml in 141 out of 144 weeks (98 percent of the time).

Figure IV-31: This graph shows the daily effluent pH of the WWTP for both wet and dry weather conditions. All values are expressed in standard units of pH (SU). The daily interim limits for wet and dry weather conditions are the same. The interim permit requires effluent pH within the range of 6.0 to 9.0. The data show that the WWTP has had a pH within this range for 1,090 out a total of 1,095 days (99 percent of the time) where the pH was recorded. Of the 5 days where daily sample results were outside of the permitted range, all had a pH of less than 6.0. The average pH value of the effluent is 7.0.

Figure IV-32: This graph shows the monthly average and daily (5 days a week) concentration of residual chlorine in the effluent for both wet and dry weather conditions. All values are expressed as mg/l. The data show that the plant has met the wet and dry weather monthly average interim permit limits of 0.023 and 0.021 mg/l respectively, in 18 of the 18 months (100 % of the time). The plant has met the dry weather maximum daily limit of 0.023 mg/l in

INSERT FIGURE IV – 29 Percent Removal TSS (lbs/day)

INSERT FIGURE IV-30 EFFLUENT FECAL

INSERT FIGURE IV-31 EFFLUENT pH

INSERT FIGURE IV-32 Residual Chlorine

776 out of 780 samples (99 percent of the time) when dry weather limits would apply. The WWTP has met the wet weather maximum daily interim limit of 0.026 mg/l in 779 out of 780 samples (99 percent of the time) when wet weather limits would apply. The overall average chlorine residual discharged during dry weather conditions is 0.00 mg/l.

Figure IV-33: This graph shows the monthly average concentration of ammonia discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim dry weather limit of 18.6 mg/l monthly average in 18 of the 18 months (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim monthly wet weather limit of 12.9 mg/l in 16 out of 18 months (89 percent of the time).

Figure IV-34: This graph shows the daily concentration of ammonia discharged from the WWTP, along with the interim limits for wet and dry weather conditions. The data show that the WWTP has met the interim dry weather daily limit of 36.8 mg/l in 1,084 of the 1,084 daily samples (100 percent of the time) when dry weather limits would apply. During the same time period, the plant has met the interim daily wet weather limit of 31.6 mg/l in 1,084 out of 1,084 daily samples (100 percent of the time). The overall average ammonia concentration during dry weather conditions is 2.5 mg/l. The overall average ammonia concentration during wet weather conditions is 9.5 mg/l. It should be noted the WWTP is currently not capable of performing nitrification during wet weather conditions.

Summary

The existing plant meets most conditions of the NPDES permit with good reliability. Most of the violations are for TSS compliance during wet weather, which is caused by inadequate secondary clarifier capacity. The plant also experiences violations in the fall when the aeration basin is drained so that it can be used for equalization storage. Tables IV-9 and IV-10 show a performance summary of the existing plant.

FIGURE IV-33 Monthly Ammonia Concentrations mg/l

FIGURE IV-34 Daily Ammonia Concentration

TABLE IV-9 DRY WEATHER PERFORMANCE SUMMARY			
Water Quality Parameter	Permit Condition	Interim Permit Limit	% Compliance

BOD_5 Concentration	Monthly Average Weekly Average	20 mg/l 30 mg/l	94% 99%
BOD_5 Mass Loading	Monthly Average Weekly Average % Removal	334 lb/day 500 lb/day 85%	94% 97% 94%
TSS Concentration	Monthly Average Weekly Average	25 mg/l 37.5 mg/l	94% 97%
TSS Mass Loading	Monthly Average Weekly Average % Removal	417 lb/day 626 lb/day 85%	100% 93% 94%
Ammonia Concentration	Monthly Average Daily Maximum	18.6 mg/l 36.8 mg/l	100% 100%
Fecal Coliforms	Monthly Geometric Mean Weekly Geometric Mean	200/100 ml 400/100 ml	100% 100%
pH	Daily Maximum Daily Minimum	9 6	100% 95%
Chlorine Residual	Monthly Average Daily Maximum	0.021 mg/l 0.023 mg/l	100% 99%

TABLE IV-10
WET WEATHER PERFORMANCE SUMMARY

Water Quality Parameter	Permit Condition	Permit Limit	% Compliance
BOD_5 Concentration	Monthly Average Weekly Average	30 mg/l 45 mg/l	100% 100%
BOD_5 Mass Loading	Monthly Average Weekly Average % Removal	1,000 lb/day 1,500 lb/day 75%	100% 100% 67%
TSS Concentration	Monthly Average Weekly Average	30 mg/l 45 mg/l	78% 94%
TSS Mass Loading	Monthly Average Weekly Average % Removal	1,000 lb/day 1,500 lb/day 65%	83% 89% 83%
Ammonia Concentration	Monthly Average Daily Maximum	12.9 mg/l 31.6 mg/l	89% 100%
Fecal Coliforms	Monthly Geometric Mean Weekly Geometric Mean	200/100 ml 400/100 ml	100% 98%
pH	Daily Maximum Daily Minimum	9 6	100% 100%
Chlorine Residual	Monthly Average Daily Maximum	0.023 mg/l 0.026 mg/l	100% 99%

METALS PERFORMANCE

The draft water quality analysis for metals is presented in Section III of this report. The following graphs show the plant's performance for copper, silver and zinc.

Figure IV-35: This graph shows the effluent copper concentration of the WWTP. The interim limit for copper is the same for both wet and dry conditions and is based on a daily maximum. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (μg/l). The data show that the WWTP has met the interim limit of 53.5 μg/l daily maximum in 29 out of 29 months (100 percent of the time) where the copper concentration was recorded.

Figure IV-36: This graph shows the monthly effluent silver concentration of the WWTP. The interim limit for silver is the same for both wet and dry conditions and is based on a daily maximum and yearly average. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (μg/l). The data show that the WWTP has met the interim limit of 28.2 μg/l daily maximum in 29 out of 29 months (100 percent of the time) where the silver concentration was recorded. The plant has met the yearly average limit of 13.5 μg/l in 3 of 3 years. The overall effluent silver concentration is 3.08 μg/l and the 99th percentile value is 11.5 μg/l.

Figure IV-37: This graph shows the monthly effluent zinc concentration of the WWTP. The interim limit for zinc is the same for both wet and dry conditions and is based on a daily maximum. Except for the clean water sampling period, samples are only taken once a month. All values are expressed in micrograms per liter (μg/l). The data show that the WWTP has met the interim limit of 119.6 μg/l daily maximum in 28 out of 29 months (97 percent of the time) where the zinc concentration was recorded. The overall effluent zinc concentration is 75.6 μg/l and the 99th percentile value is 136 μg/l.

INSERT FIGURE IV-35 EFFLUENT COPPER

INSERT FIGURE IV-36 EFFLUENT SILVER

INSERT FIGURE IV-37 EFFLUENT ZINC

BIOSOLIDS (SLUDGE) TREATMENT AND DISPOSAL

The plant has both primary and secondary clarifiers. The primary clarifier sludge is wasted to the primary anaerobic digester for stabilization. Sludge is removed from the secondary clarifiers and pumped back to the primary clarifiers where it commingles with the primary sludge. The activated sludge is either returned through the treatment process (RAS) or wasted to the primary anaerobic digester (WAS). Sludge is sent to the digester at approximately 3 to 5% solids concentration. The solids are treated in two anaerobic digesters, which are operated in a series. The first anaerobic digester is heated and mixed. After a detention time of approximately 59 days, the sludge is transferred to the other anaerobic digester that is not heated or mixed. After another 59 days, the treated solids are sent to a sludge storage basin for thickening prior to being pumped to covered drying beds for dewatering. The thickened sludge has a solids concentration of 8 to 10%. The dried biosolids are trucked to eastern Washington where they are utilized for agricultural land application.

SEWER SYSTEM OPERATION AND MAINTENANCE COSTS

Each entity maintains separate budgets for operation and maintenance of their wastewater facilities. The budget for the regional WWTP is maintained by the City of Chehalis and costs are reimbursed to the City by Napavine and LCSD No. 1 for their proportional WWTP expenses. Using recent City expenditures, a budget for operation and maintenance of both the plant and the collection system were developed. Table IV-11 provide a line item breakdown of expenditures for the WWTP. The current cost to operate the WWTP is $1,100,680 annually. This does not include any costs associated with collection system improvements and maintenance of the collection system.

TABLE IV-11 WWTP Expenses (1998)*	
Inflation Rate Applied to Cost 3.00%	**Estimated WWTP Cost**
Salaries and Wages (S&W)	$ 407,000
Personal Benefits	$ 142,500
Office & Operating Supplies	$ 53,000
Professional Services	$ 8,000
Uniforms & Clothing	$ 6,000
Communications	$ 12,000
Travel	$ 300
Rentals and Leases	$ 1,000
Advertising	$ 100
Insurance	$ 5,000
Public Utility Service	$ 58,000
Small Tools & Minor Equipment	$ 1,000
Maintenance & Repairs	$ 36,000
Machinery & Equipment	$ 33,000
Fuel	$ 4,000
Miscellaneous	$ 11,000
Taxes	$ 12,000
Interfund Supplies	$ 200
Interfund Repairs & Maintenance	$ 1,000
Sewer System Reserve Fund	$ 43,000
Existing Debt Service	$ 110,900
TMDL Related Costs	$ 500,000
Total Expenditures	**$1,445,000.00**

* Does not include O&M costs for Chehalis, Napavine and LCSD No. 1 Collection System.

The municipal codes for the Cities of Chehalis and Napavine establish authority to charge for sewer service. LCSD No.1's authority to charge for service is established in Title 57 of the Resource Code of Washington (RCW). Rates and charges are established independently for each entity by the elected bodies through ordinances and resolutions. The rates and charges for each entity vary significantly as shown in Tables IV-12 through Table IV-17.

TABLE IV-12 CITY OF CHEHALIS CONNECTION CHARGES	
Type of Service	**Connection Charge**
All Customer Types	$2,991 (1999)
Existing Line Surcharge	$1,000 (if not previously contributed to line)
Airport Area Surcharge	$1,452 (1999)

TABLE IV-13 CITY OF NAPAVINE CONNECTION CHARGES		
Type of Service	**Connection Charge**	
Residential and Commercial	Inside City	Outside City
▪ ¾" meter	$3,500	$4,500 plus ERU Charge $750
Industrial and/or >2" meter	Determined by the Director and Council	
Multiple Units	$1,000 for each additional unit 2-20 $3,500 and $4,500 for the 21st, $1,000 for 22-40, etc.	

*Increase basic charge by $100 per year

TABLE IV-14 LEWIS COUNTY SEWER DISTRICT NO. 1 CONNECTION CHARGES (1999)			
Type of Service	**Connection Charge**		
Residential	ULID No.1	ULID No.2	Outside District
▪ Existing Stub or lot	$1,200	$3,750	Not Applicable
▪ New Lot*	$4,000	$12,000	$5,000
Multiple Units **	$1,000 each	$1,500	$1,500
Commercial & Industrial	Determined by the Commissioners		

* Not originally part of ULID No.1 or ULID No. 2.
** In addition to the basic charge for one residential.

TABLE IV-15 CITY OF CHEHALIS SERVICE CHARGES (1999-2001)*			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Single family	$25.02/($37.53)	$3.07/100 c.f.	$55.72/($68.23)
▪ Low Income/disabled	$17.89/($26.84)	$3.07/100 c.f.	$48.59/($57.54)
Commercial (per unit)	$25.02/mo.	$3.07/100 c.f.	$55.72
Industrial	$3,225 (1MG)	$0.26/lb of BOD $0.38/lb of TSS	Not Applicable

* Service charges increase by approximately 3% in 2002 and 2003

TABLE IV-16 CITY OF NAPAVINE SERVICE CHARGES			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Inside City	$26.00/mo.	$1.50/100 c.f. >300 c.f.	$36.50
▪ Outside City	$31.00/mo.	$1.75/100 c.f. >300 c.f.	$43.25
Schools	$25.00/mo.	$3.00/100 c.f. >300 c.f.	$46.00
Churches	$26.00/mo.	$1.50/100 c.f. >300 c.f.	$36.50
Commercial/Industrial			
▪ Inside City	$26.00/mo	$3.00/100 c.f. >300 c.f.	$47.00
▪ Outside City	$50.00/mo.	$3.75/100 c.f. >300 c.f.	$76.25

TABLE IV-17 LEWIS COUNTY SEWER DISTRICT NO. 1 SERVICE CHARGES			
Type of Service	**Service Charges**		
Residential	**Base Rate**	**Commodity Charge**	**Cost/1,000 c.f./mo.**
▪ Inside District	$18.00/mo.	Not Applicable	$12.00
▪ Outside District	$22.00/mo.	Not Applicable	$16.00
Trailer courts			
▪ Inside District	$3.00/mo./pad	$75.00/dump site/mo.	Not Applicable
▪ Outside District	$4.00/mo./pad	$90.00/dump site/mo.	Not Applicable
Nursing and Rest Homes			
▪ Inside District	$30.00/mo.	$5.00/bed/mo.	Not Applicable
▪ Outside District	$40.00/mo.	$6.00/bed/mo.	Not Applicable
Office, Daycare, etc.			
▪ Inside District	$40.00/mo.	Or $0.04/s.f. if greater	Not Applicable
▪ Outside District	$50.00/mo.	Or $0.05/s.f. if greater	Not Applicable
Restaurant, Café, etc.			
▪ Inside District	$60.00/mo.	$0.60/seat	Not Applicable
▪ Outside District	$75.00/mo.	$0.75/seat	Not Applicable

SEWER USE ORDINANCE

Sewer Use Ordinances are contained in Chapter 13.08 of both Cities Municipal Codes. All regulations pertaining to the use and charge for the sewer system are contained in this Chapter. The City of Chehalis sewer use ordinance is very thorough and covers the following topics:

- Connection policies.
- Construction standards for sewers and side sewers.
- Conditions on prohibition of specific discharges.
- Pretreatment standards.
- Administrative policy.
- Compliance and enforcement.

The sewer use ordinance serves the City well and no additions or amendments are anticipated or recommended. City of Napavine and LCSD No. 1 Sewer Use Ordinances and policies are not as comprehensive, but have serviced the respective utilities well in the past. Service area policy details and policy recommendations will be completed in the Facilities Plan.

SECTION V

EXISTING AND FUTURE WASTE LOADS

INTRODUCTION

The purpose of this section is to determine future population, wastewater flow and wastewater characteristics for the Chehalis Wastewater Treatment Plant (WWTP) service area. The population, flow and loading estimates are determined for the base year (1997) through the year 2025. The planning period goes beyond the traditional 20-year planning period to ensure that any new treatment facilities recommended in this plan will have a minimum 20-year capacity after the final construction date.

EXISTING AND FUTURE SERVICE AREA

The current and future service areas for the Chehalis WWTP are shown on Figure IV-3. The future service area is delineated for both the 2025 design year and for an "ultimate" buildout area. The WWTP currently serves the City of Chehalis, City of Napavine, LCSD No.1 and unincorporated areas of Lewis County. Lewis County, Chehalis, and Napavine have prepared comprehensive plans for compliance with the State of Washington Growth Management Act (GMA). Interim Urban Growth Areas (IUGAs) were established for Napavine, Chehalis and unincorporated areas in May 1998. To be consistent with GMA, the proposed 2025 service area represents the IUGAs for Napavine and Chehalis along with minor adjustments to account for LCSD No.1. The IUGA boundaries may change but should not be significant and will not affect the total population forecasts for each entity. Since the planning process is complete, the IUGA's have become Urban Growth Areas (UGAs). UGAs will be reviewed by each entity on a yearly basis and updated as needed. The 2025 and ultimate service area boundaries will continue to be updated in the future as needed to account for changes to the Chehalis and Napavine UGAs. The ultimate service area includes areas that may be served by the WWTP well into the future, but are not anticipated to contribute to the projected WWTP flow until after 2025. The ultimate service area will be used in the GSP for evaluation of long term siting issues. Since interceptor system improvements typically have a greater design life than WWTP improvements, the ultimate service area will also be considered in land use based flow projections in the evaluation of interceptor system components and future interceptor system capacities in Section VI.

POPULATION ESTIMATES

Population data and 20-year growth estimates for the two cities are obtained from the most recent update of the Washington State Office of Financial Management (OFM) population estimates and applicable comprehensive plans. Population data for LCSD No.1 is obtained by multiplying the number of residential sewer customers by the unincorporated county population density of 2.6 persons per household.

The Lewis County Comprehensive Plan identifies a 20-year uniform annual growth rate of 1.39% for all of Lewis County as amended in 1995 by the OFM. This uniform growth rate is applied in the County Comprehensive Plan to the unincorporated area population and each city is assigned a variable rate to account for the rest of the County's population growth. Annual growth rates assigned to the City of Chehalis and the City of Napavine within the County Comprehensive Plan are 1.1% and 3.9% respectively. This report utilizes the County uniform annual growth rate of 1.39% for population projections for Chehalis to be conservative and because a lot of the growth will be in the UGA. The City of Napavine Comprehensive Plan utilizes the annual growth rate of 3.9% for population estimates as identified in the County Comprehensive Plan.

From 1992 to 1997, the average annual increase in sewer connections was approximately 1.9% for Chehalis, 6.2% for Napavine and 18.1% for LCSD No.1. The recent growth rates for Chehalis and Napavine do not appear excessive over the short-term compared to their anticipated 20-year growth rates. Therefore, the long-term (20-year) growth rates of 1.39% and 3.90% will be used in this document for projecting future residential sewage demand in the City of Chehalis and the City of Napavine respectively. The Chehalis Planning Commission and City Council have agreed to use the 1.39% growth rate for sewer planning. The Napavine Planning

Commission also concurs with the 3.9% growth rate for sewer planning purposes within the City of Napavine.

The short-term growth rate for LCSD No.1 (18.1%) is substantially higher than the 1.39% unincorporated county 20-year growth rate. Conversations with LCSD No.1 staff indicate that much of the growth in the last 5-year period was due to a ULID and rapid development of several long plats. However, growth is expected to taper off to approximately 5 to 15 new connections per year over the next few years. The LCSD No.1 growth rate will likely decline further towards the end of the planning period unless district boundaries, land use densities or public water system facilities are significantly increased. A conservative 20-year growth rate of 2.00% (relative to other unincorporated areas) is used for LCSD No.1.

To be consistent with GMA, the GSP will use city generated growth estimates in lieu of county estimates where there are differences in growth rates or population estimates. The GSP will also use District estimates in lieu of county estimates due to the readily available anticipated growth trends for the relatively small and uniform service area. The individual 20-year residential population growth rates are applied to the base year population estimates from OFM to obtain the 2025 population estimate for each entity. The sum of the population estimates for each entity is the estimated total population to be served by the Chehalis WWTP for the planning period. A summary of the population estimates for the 2025 planning period is shown in Table V-1.

TABLE V-1
1997 and 2025 ESTIMATED RESIDENTIAL POPULATION

	Chehalis	Napavine	LCSD No.1	Total
1997 Residential Population	7,035	1,176	460(1)	8,671
Long term annual growth rate	1.39%	3.9%	2.0%	1.88%(2)
2025 Residential Population	10,354	3,433	801	14,588

(1) Based on 2.6 persons per household.
(2) Total overall annual growth rate based on the total population growth shown in the table.

COMMERCIAL/INDUSTRIAL USERS

The City of Chehalis completed a draft commercial and industrial user survey in September of 1998. The survey has been on-going since 1996 and resulted in a master list of 493 "Non-

Domestic Sewer Users (NDSUs). The master list contains pertinent information corresponding to the DOE standard industrial user survey form and includes all NDSUs from Chehalis, Napavine and Lewis County Sewer District No.1.

NDSUs are classified by DOE by the following four categories:

- SIUs – Significant Industrial Users;
- MIUs – Minor Industrial Users;
- DEQs – Domestic Equivalent Users; and
- NUs – Non-Users

The survey results identified 5 SIUs, 15 MIUs, 454 DEQs and 19 NUs. User classifications are updated continually through business license renewals. The City sent out surveys to all listed MIUs and SIUs in 1999 to ensure that all of the required information in their database is up to date.

A summary of the data for SIUs is shown in Table V-2. A copy of the SIU and MIU database tables required by DOE for industrial user survey compliance are included in Appendix C.

TABLE V-2
SIGNIFICANT INDUSTRIAL USERS (SIUs)

Company Name	Industrial Category and User Description	Waste streams to POTW (gpd)		DOE Identifier Number
		Process or Washwater	*Domestic Wastewater*	

Cummins Northwest, Inc. *	Diesel engine / truck repair.	N/A	420	24287712
Darigold **	Processing of milk products.	N/A	1,350	28466988 44275517
National Frozen Foods Corp. **	Processing of vegetables.	5,157	2,661	83558168 11511688 44582366
NC Machinery Co. *	Service of diesel equipment.	900	1,825	1182
Qualex, Inc. *	Large scale photoprocessing.	37,000	3,000	31446544
	Total	43,057	9,256	

* State Permit for Discharge to Publicly Owned Treatment Works (POTW) – All process and domestic wastes generated at these sites are discharged to a POTW.
** State General Discharge Permit – The quantity shown is discharged to POTW on a daily basis (only). Other waste generated at these sites are discharged under provisions of the general permit.

Cummins Northwest, NC machinery, and Qualex have pre-treatment facilities that are under state waste permits for discharge to the Chehalis WWTP.

Darigold and National Frozen Foods have state waste discharge permits. The amount of discharge by these companies to the WWTP shown in the table only represents a portion of their total discharge under their general permits. National Frozen Foods discharges a majority of their process waste streams to land application. Darigold discharges a majority of their process waste stream to an outfall in the vicinity of the Chehalis WWTP outfall after treatment in their own WWTP. During the summer months (since 1995), Darigold discharges approximately 250,000 gallons per day of treated effluent to the Chehalis WWTP to augment the Chehalis WWTP nitrification processes. The effluent discharge to the WWTP is not included in base year flow estimates or future flow projections since Darigold will land apply all process effluent in the future due to Consent Decree limitations.

Total waste discharged to the WWTP by SIUs shown in Table V-2 is 52,313 gallons per day. Total waste discharged by MIUs listed in the City's NDSU database is 15,054 gallons per day. Specific flow data for the 454 DEQs is not required to be included in the database. However, it is estimated that the DEQs account for approximately 390,000 gallons per day (approximately 860 gallons per day per DEQ) based on estimates of total commercial and industrial flow presented later in this section. NUs do not contribute to WWTP flow.

COMMERCIAL/INDUSTRIAL GROWTH RATES

The commercial sewer accounts for all of the entities are comprised of any customer that is not classified as single-family residential. The commercial accounts include, but are not limited to, uses such as multi-family, commercial and industrial. There are no industrial dischargers in Napavine or LCSD No. 1 and technically no industrial discharges to the WWTP as defined by the City of Chehalis sewage ordinance. The industrial customers and related sewage flows referenced in this document pertain to businesses that meet the definition of "industrial" within the Standard Industrial Classification (SIC) Code.

In this analysis, a uniform growth rate will be utilized for projecting future commercial sewage flow for the entire service area. More detailed land use based projections will be utilized in Section VI to evaluate and determine collection system capacities and components.

Growth rate estimates for commercial and industrial sewage flow are based on the projected employment growth and total residential growth within the planning area. Employment estimates for Lewis County and the City of Chehalis UGA are shown in the November 1997 Lewis County Economic Development Council (EDC) report titled *Lewis County Industrial Needs Analysis* by E.D. Hovee & Company referred to as the "Hovee Report". The Hovee Report evaluated several factors including land use, OFM population estimates and labor force participation to estimate future commercial and industrial work force within the Chehalis planning area. Total annual employment growth in all of Lewis County is projected at approximately 1.5% over the next 20 years. Recent historic trends show this growth is almost equal between the industrial and commercial employment sectors. Revised employment sector projections for the Chehalis UGA, within the Hovee Report and the draft Chehalis Comprehensive Plan, project higher overall employment growth and significant industrial employment growth relative to commercial employment within the Chehalis UGA. To account for the revised Chehalis UGA employment estimates and to be consistent with the cumulative population growth shown in Table V-1, a higher growth rate of 1.9% will be used for projecting commercial and industrial sewage flows in the GSP. This will also help ensure that adequate sewage capacity is available. The 1.9% commercial and industrial growth rate will be applied uniformly across the entire planning area for the purpose of this report.

The growth rates presented herein do not allow for a large number of “wet” industries to be located within the sewer service area. Proposals from “wet” industries with potentially large sewage flows that may impact the WWTP capacity must be addressed on a case-by-case basis. One such proposal that has been received by the City of Chehalis, is to build a power plant in the Chehalis industrial park area. Tractebel is currently planning on constructing a power plant that will produce an estimated wastewater flow of 80,000 GPD. Although this will use a significant amount of the capacity identified for commercial/industrial customers, no adjustment to the flow estimates will be made. However, it will affect the growth rate previously presented. The Tractebel plant is expected to be on line in 2002. The 1.9% growth rate is used from 1997 to 2002 where a lump sum of 80,000 GPD is applied. This brings the anticipated C/I flow in 2002 up to 0.581 MGD. From there, a uniform growth rate of 1.25% per year is used to arrive at the anticipated 2025 C/I flow of 0.77 MGD.

WWTP FLOW

Current and future WWTP flows are established by utilizing existing WWTP flow data and the population estimates discussed previously in this section. Daily WWTP flow data are available from 1988 to the present. For this report, data from April 1995 through March 1998 are utilized to establish base year (1997) flows. Where practical, specific 1997 flow data are utilized for comparisons to account for the aggressive I/I removal program that has been initiated over the past two decades with major I/I removal projects completed as recently as 1996 and smaller projects completed in 1997 and 1998. WWTP flow data are also analyzed based on river flow conditions as established in the Consent Decree discussed in Section III and summarized below.

FLOW-BASED PARAMETERS

The Consent Decree establishes final discharge limits for the WWTP, which are based on river flows. Since low treatment plant flows correlate with low river flows and high treatment plant flows correlate with high river flows, discharge limits are tied to river flow. This flow-based approach works better than the calendar-based approach to meet the goals of the Total Maximum Daily Load (TMDL) Study in the portion of the Chehalis River known as the "Centralia Reach". As discussed below, "dry weather" and "wet weather" periods are based on river flow, as opposed to using a calendar-based dry weather period of May 1 through October 31 and a calendar-based wet weather period of November 1 through April 30.

The dry weather period under the flow-based limits occurs the day after the 7-day moving average of river flow when the Centralia Reach drops below 1,000 cubic feet per second (cfs). The wet weather period begins the day after the moving 7-day river flow average exceeds 1,000 cfs, provided at least one day within the 7-day period is greater than 2,500 cfs.

Flow data from the Chehalis River at the Grand Mound monitoring station is utilized to determine the dry weather and wet weather periods for determining WWTP flow parameters. The Grand Mound data is adjusted to approximate conditions at the Centralia Reach by utilizing the formulas and conversion factors identified in the Consent Decree and in Section III of this report.

CURRENT WWTP FLOW

The WWTP currently meters influent and effluent flows. To account for data collection practices at the WWTP, the analysis of current flow data utilizes the higher of the influent or effluent flow for the “daily flow” to the plant. Since the summer of 1995, the City has accepted Darigold's treated effluent during dry weather because their effluent contains approximately 100 mg/l of alkalinity which helps the plant's nitrification process. DOE has raised concerns over Chehalis accepting Darigold’s effluent including dilution of the Chehalis influent and water quality concerns. Therefore, in 2001, the City stopped accepting Darigold’s effluent. The City is currently adding more lime in order to offset the loss of Darigold’s alkalinity. Daily records of Darigold, Inc. effluent contributions to the WWTP headworks are subtracted from the daily flow where applicable since Darigold will not contribute to future flows. Darigold has indicated that all of their wastewater effluent will be committed to their future land-based application. A schematic of WWTP flow meters is shown in Figure V-1. Flow data and summaries of data analysis are included in Appendix C.

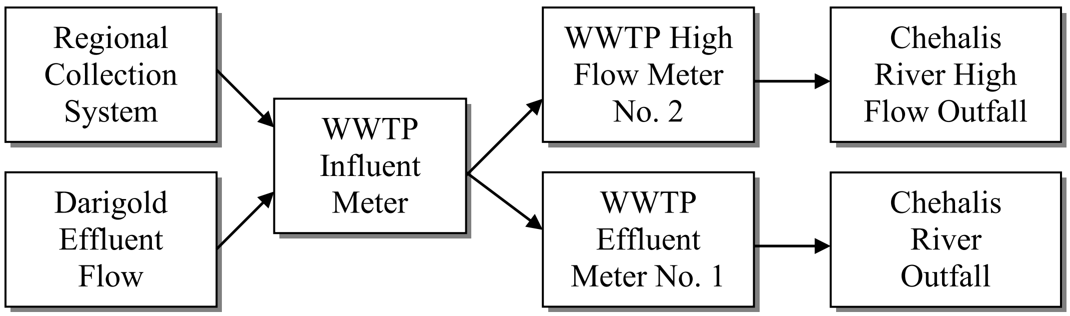


Figure V-1
WWTP Flow Meter Schematic

AVERAGE ANNUAL DAILY FLOW (AADF)

As discussed in Section IV, and shown in Table IV-6, the average annual daily flow to the WWTP based on the three year data set is 2.23 MGD.

AVERAGE DAY DRY WEATHER FLOW (ADWF)

Using an analysis of WWTP flows based on river flows (flow-based) results in an ADWF of 1.41 MGD. The ADWF for river flow from 200 cfs to 1,000 cfs is 1.43 MGD and the ADWF for river flow less than 200 cfs is 1.13 MGD. A base flow ADWF of 1.15 MGD is calculated by taking the average of the daily flow from the months of July through September throughout the period and subtracting Darigold's flow where applicable. Data from the months of July through September are used in this report for calculating base flow ADWF in order to get a true definition of the base sewage contribution from sewer customers with minimum impact from I/I. Historically, rainfall during the period of July through September is less than 10% percent of the total yearly rainfall in the area. In contrast, over 25% of annual rainfall occurs in the months of May through October according to the May 1987 *Soil Survey of Lewis County Area, Washington* by USDA SCS.

MAXIMUM MONTH DRY WEATHER FLOW (MDWF)

The maximum 30-day moving average flow for all data where dry weather limits apply is 1.90 MGD. The flow-based maximum dry weather monthly average flow is 2.48 MGD and occurred in an 11 day period in October 1997. This report will use the more conservative value of 2.48 MGD.

PEAK DRY WEATHER FLOW (PDWF)

The flow-based peak day dry weather flow during the analysis period was 5.40 MGD and occurred on October 4, 1997. The next highest dry weather flow of 4.91 MGD occurred on October 30, 1997. Both values correspond to significant rainfall events. This report will use the highest value of 5.40 MGD for PDWF.

AVERAGE DAY WET WEATHER FLOW (AWWF)

The AWWF is 2.86 MGD for all wet weather data under the flow-based criteria. This report will use the 2.86 MGD flow-based value.

MAXIMUM MONTH WET WEATHER FLOW (MWWF)

The flow-based MWWF using a monthly average of the wet weather flow days in each month is 4.88 MGD. This MWWF occurred during the entire 31 days in December of 1996. The next highest MWWF of 4.59 MGD occurred over 22 days in November of 1995. This report will use the highest MWWF value of 4.88 MGD.

PEAK DAILY FLOW (PDF)

The maximum daily flow recorded during the period is 13.77 MGD, which occurred on November 10, 1995. However, as discussed in Section IV of this report, this flow measurement is not reliable because WWTP records do not show any direct relationship to flooding or process conditions. A daily flow of 12.02 MGD occurred on February 7, 1996 and is utilized as the existing PDF for the WWTP. This flow does correspond to a specific flood event and is verified by other WWTP process records.

Peak Day I/I

The peak day I/I is calculated by subtracting the ADWF from the PDF and is 10.87 MGD (12.02-1.15).

EQUIVALENT POPULATION

The overall residential wastewater flow from Chehalis, Napavine and LCSD No.1 is estimated at 80 gallons per capita per day (gpcd). The estimate accounts for both DOE design guidelines and water use data discussed in Section IV. The 1997 base year equivalent population is the sum of the total 1997 estimated service area population and the equivalent population for commercial/industrial (C/I) customers as expressed by the relationship shown below.

Total Equivalent Population = Total Population + C/I Equivalent Population

C/I Equivalent Population = (ADWF - Residential Flow)/80 gpcd

Residential Flow = Total Population x 80 gpcd

Based on the population for the planning area of 8,671 (1997) and an ADWF of 1.15 MGD, the equivalent population for commercial and industrial wasteload (from the relationship shown above) is 5,704. The total base year equivalent population utilized to determine the per capita flow for the other WWTP flow parameters for the planning area is 14,375. Based on this evaluation, approximately 60% of the wasteload is residential and 40% is from commercial and industrial customers. A summary of the existing WWTP flow parameters is listed in Table V-3.

TABLE V-3
CURRENT WWTP FLOW PARAMETERS
Total Equivalent Population = 14,375

WWTP Flow Parameter	Flow (MGD)	Flow (GPCD)
Average Annual Daily Flow (AADF)	2.23	155
Average Dry Weather Flow (ADWF) (1)	1.15	80
Maximum Dry Weather Month Average Flow	2.48	173
Peak Day Dry Weather Flow	5.40	376
Average Wet Weather Flow (AWWF)	2.86	199
Maximum Wet Weather Month Average Flow	4.88	339

Peak Day Wet Weather Flow (PDF)	12.02	836
Peak I/I Flow (PDF – ADWF)	10.87	756

(1) Data from the months of July through September are used for the calculation in this analysis in order to get a true definition of the base wastewater contribution by sewer customers without the influence of I/I.

PROJECTED WWTP FLOW

Based on residential and commercial/industrial growth rates, the 2025 equivalent population for the planning area is 24,180. This represents an equivalent population gain of 9,805 for the planning period, or approximately 1.9% annually. Calculations for 2025 WWTP flow parameters are based on the per capita flow shown in Table V-3 and the 2025 equivalent population. WWTP flow parameters for 2025 are shown in Table V-4. Projections for peak, maximum month and average wet weather flow include an allowance for new I/I, which is estimated to be 25 gallons per population equivalent per day. The I/I from the existing population (i.e. the existing sewer system) is assumed to stay the same. The Consent Decree postpones any further mandatory I/I reduction efforts until after the final treatment limits take effect, but the City will have to maintain the collection system to prevent any further deterioration. This will require some pipeline upgrade or replacement as identified in Section VI of this report.

TABLE V-4
2025 WWTP FLOW PARAMETERS
Total Equivalent Population = 24,180

WWTP Flow Parameter	2025 Residential (MGD)	2025 Commercial/ Industrial (MGD)	Existing I/I (MGD)	New I/I (MGD)	2025 Total Flow (MGD)
Average Annual Daily Flow (AADF)	1.17	0.77	1.06	0.25	3.25
Average Dry Weather Flow (ADWF)	1.17	0.77	0	0	1.94
Maximum Dry Weather Month Average Flow	1.17	0.77	1.07	0.25	3.26
Peak Day Dry Weather Flow	1.17	0.77	3.99	0.25	6.18
Average Wet Weather Flow (AWWF)	1.17	0.77	1.70	0.25	3.89
Maximum Wet Weather Month Average Flow	1.17	0.77	3.72	0.25	5.91
Peak Day Wet Weather Flow (PDF)	1.17	0.77	10.86	0.25	13.05
Peak I/I Flow (PDF – ADWF)	--	--	10.86	0.25	11.11

AVERAGE ANNUAL DAILY FLOW (AADF)

The 2025 AADF to the WWTP is 3.25 MGD, which is the sum of the 1997 wastewater flow, plus the wastewater flow from the new equivalent population, plus the existing I/I, plus an allowance of 25 gpcd (30%) for new I/I which will be added to the projected flow for each new equivalent customer.

AVERAGE DAY DRY WEATHER FLOW (ADWF)

The 2025 ADWF is 1.94 MGD, which is the sum of the flow for the 2025 residential population and the 2025 equivalent C/I population.

MAXIMUM MONTH DRY WEATHER FLOW (MDWF)

The 2025 MDWF is 3.26 MGD, which is the sum of the 1997 MDWF (including existing I/I) plus wastewater flow from the new equivalent population.

PEAK DRY WEATHER FLOW (PDWF)

The 2025 PDWF is 6.18 MGD, which is the sum of the 1997 PDWF (including existing I/I) plus wastewater flow from the new equivalent population.

AVERAGE DAY WET WEATHER FLOW (AWWF)

The 2025 AWWF is 3.89 MGD, which is the sum of the 1997 AWWF, (including existing I/I) plus wastewater flow from the new equivalent population, and an allowance of 25 gpcd for new I/I, which will be added to the projected flow for each new equivalent customer.

MAXIMUM MONTH WET WEATHER FLOW (MWWF)

The 2025 MWWF is 5.91 MGD, which is the sum of the 1997 MWWF, (including existing I/I) plus wastewater flow from the new equivalent population and an allowance of 25 gpcd for new I/I, which will be added to the projected flow for each new equivalent customer.

PEAK DAILY FLOW (PDF)

The 2025 PDF is 13.05 MGD, which is the sum of the 1997 PDF and flow from the new equivalent population and an allowance of 25 gpcd for new I/I, which will be added to the projected flow for each new equivalent customer.

PEAK DAY I/I

The 2025 peak day I/I is 11.11 MGD, which is the sum of 1997 peak day I/I and I/I flow from the new equivalent population.

A summary of existing WWTP flows, 2025 flows and design flow values are shown in Table V-5.

TABLE V-5
EXISTING AND 2025 WWTP FLOW PARAMETERS

Flow Estimates	Existing (1997) Population 8,671	2025 Population 14,590	2025 Design Value
Total Equivalent Population	14,375	24,180	25,000
Residential Flow (gal/day) (80 gal/day/capita)	694,000	1,167,000	1.2 MGD
Commercial and Industrial Flow (gal/day) (80 gal/day/equivalent Population)	456,000	767,000	0.8 MGD
Average Dry Weather Flow - ADWF (gal/day)	1,150,000	1,934,000	2.0 MGD
Peak I&I Flow (gal/day)	10,870,000	11,115,000	11.0 MGD
Peak Daily Flow - PDF (gal/day)	12,020,000	13,049,000	13.0 MGD
Maximum Month Wet Weather Flow (gal/day)	4,900,000	5,909,000	6.0 MGD
Maximum Month Dry Weather Flow (gal/day)	2,480,000	3,264,000	3.5 MGD

Avg. Annual Flow (gal/day)	2,230,000	3,259,000	3.5 MGD

Future plant discharges are restricted by flow limits included in the Consent Decree. Future dry weather discharge to the Chehalis River below the Skookumchuck River is limited to a daily maximum of 3.0 MGD when the river flow is between 200 and 1,000 cfs (300 cfs as measured at the Grand Mound gage), and 2.5 MGD when the river flow is less than 200 cfs. Wet weather maximum day flow is limited to 13.0 MGD. The dry weather limits do not apply if the wastewater is used for a beneficial reuse. Since some of the anticipated future dry weather flows will exceed the discharge limits for a downstream discharge, equalization storage will be required. End-use options and equalization storage requirements will be discussed in Section VII of this report.

BOD_5, TSS AND AMMONIA LOADING TO WWTP

Daily, weekly and monthly wasteload data from April 1995 through March 1998 were used to evaluate existing wasteloads to the WWTP. Based on a detailed evaluation of the data, the following information summarizes the range of wasteloads that have been experienced at the plant. In the following discussion, the maximum monthly, weekly and daily loadings are based on a probability evaluation in which the maximum monthly value is represented by the 90th percentile of all data, the maximum weekly value is represented by the 95th percentile of all data and the maximum daily value is represented by the 99th percentile of all data. Future plant loading is estimated by applying the existing wasteloads per population equivalent to the estimated future population equivalents.

BOD_5 TO WWTP

The existing average BOD_5 influent concentration to the Chehalis WWTP is 165 mg/l and results in an existing average BOD_5 loading of 2,360 pounds per day, or approximately 0.164 pounds per equivalent population per day (including commercial and industrial equivalent populations). This report will use the 90th percentile value of the existing maximum monthly average daily BOD_5 influent loadings to define the existing BOD_5 loading. The future BOD_5 loading in the year 2025 will be estimated by increasing the 90th percentile value by the proportionate increase in equivalent population. The existing and future BOD_5 loadings for the year 2025 is shown below:

	1997 Existing BOD_5 (lbs/day)	2025 Future BOD_5 (lbs/day)
Average Annual BOD_5 at 0.17 lbs/capita/day (165 mg/l)	2,360	3,970
90th Percentile (Max. Month)	3,264	5,490
95th Percentile (Max. Week)	3,946	6,637
99th Percentile (Max. Day)	5,610	9,437

The overall WWTP will be designed using the 90th percentile value of 5,490 lbs/day, which represents the maximum month average loading in the year 2025. By using this value as the average, short-term fluctuations in the wasteload (either higher or lower) will be addressed through applicable design ranges of unit treatment processes.

SUSPENDED SOLIDS LOAD

The existing average TSS influent concentration to the Chehalis WWTP is 161 mg/l. The existing average TSS loading is 2,441 pounds per day or 0.17 pounds per equivalent population per day. This report will use the 90th percentile value of the TSS influent loadings to define the existing maximum monthly daily average TSS loading. The future TSS loading in the year 2025 will be estimated by increasing the 90th percentile value by the proportionate increase in equivalent population. The existing and future TSS loadings for the year 2025 are shown as follows:

	1997 Existing TSS (lbs/day)	2025 Future TSS (lbs/day)
Average Annual TSS at 0.17 lbs/capita/day (161 mg/l)	2,440	4,105
90th Percentile (Max. Month)	3,971	6,680
95th Percentile (Max. Week)	4,854	8,165
99th Percentile (Max. Day)	7,888	13,270

The overall WWTP will be designed using the 90th percentile value of 6,680 lbs/day, which represents the maximum month average loading in the year 2025. Again, by using this value as the average, short-term fluctuations in the wasteload will be addressed within the design range of each treatment process unit.

AMMONIA TO WWTP

The existing average ammonia influent concentration to the Chehalis WWTP is 22.3 mg/l, which results in an existing average ammonia loading is 302 pounds per day or 0.021 pounds per equivalent population per day. This report will use the 90th percentile value of the existing maximum monthly average daily ammonia influent loadings to define the existing ammonia loading. The future ammonia loading in the year 2025 will be estimated by increasing the 90th percentile value by the proportionate increase in equivalent population. The existing and future ammonia loadings for the year 2025 are shown below:

	1997 Existing Ammonia (lbs/day)	2025 Future Ammonia (lbs/day)
Average Annual Ammonia at 0.21 lbs/capita/day (22.5 mg/l)	302	510
90th Percentile (Max. Month)	493	830
95th Percentile (Max. Week)	587	990
99th Percentile (Max. Day)	854	1,440

The overall WWTP will be designed using the 90th percentile value of 830 lbs/day, which represents the maximum month average loading in the year 2025. Short-term fluctuations will be addressed in the design range of each unit process.

The WWTP has had numerous upgrades since it was first constructed. A wastewater treatment plant capacity evaluation established the capacity of all upgraded treatment process units as of 1993. That report serves as the basis for the rated capacity of the plant as stated in the NPDES permit. The predicted loading for the year 2025 and the current rated capacity of the plant is shown as follows.

	1997	This Report Predicted Loading (2025)	WWTP Capacity (current)
Population		14,375 Persons	14,458 Persons
Flow			
Max. Monthly Avg. Daily	4.9 MGD	6.0 MGD	4.0 MGD
Peak Daily	12.0 MGD	13.0 MGD	13.0 MGD
BOD_5 Maximum Monthly	3,264 lbs/day	5,490 lbs/day	4,880 lbs/day
TSS Maximum Monthly	3,971 lbs/day	6,680 lbs/day	5,125 lbs/day
NH_3 Maximum Monthly	493 lbs/day	830 lbs/day	0 (wet weather)

The above information indicates that the existing plant requires improvements to increase capacity for flow, BOD_5, TSS and NH_3 in the year 2025. Improvement options will be discussed in Section VII of this report.

SECTION VI

COLLECTION SYSTEM EVALUATION

INTRODUCTION

The City of Chehalis, City of Napavine and Lewis County Sewer District No. 1 collection system consists of more than 65 miles of collection system piping and has 16 pump stations. This collection system as mentioned previously, is owned and operated by the three individual entities. The purpose of this section is to describe the existing collection system and identify future sewer line extensions that will be needed to serve the future sewer service area. The section will also present an inventory of the entire collection system, evaluate the systems overall condition (through the inventory and flow monitoring), describe the results of a flow monitoring program, describe an Infiltration and Inflow (I/I) removal program and identify future sewer service extensions.

BASIN DESCRIPTIONS

The collection system was divided into twelve discrete flow basins, of which, ten were equipped with flow monitors (two basins consist of lines that are not possible to monitor). The monitors were used to conduct an I/I evaluation to determine I/I levels in the system, determine past I/I rehabilitation project effectiveness and to allocate base flow and I/I flow into the various basins. Figure VI-1 shows the collection system divided into the twelve basins. Characteristics of these basins are discussed below and a basin inventory is provided in Table IV-1. Maps of each basin are provided in Appendix D.

> **BASIN 1** is located on the north end of the City of Chehalis and is bounded by City Limits on the north side. This basin currently serves approximately 430 acres of relatively flat area. Land use within this basin is predominantly commercial with a small amount of industrial and residential land use in the State Street area. There are five pump stations within this basin.

INSERT FIGURE VI-1

INSERT TABLE VI-1

Collection system piping in the basin consists of approximately 44,000 feet of pipe ranging in diameter from 3-inch force mains up to a small portion of 12-inch gravity main. Pipe materials include PVC, concrete, asbestos cement pipe and a small portion of clay. There are about 30 inch-miles (pipe diameter in inches multiplied by feet of pipe divided by 5,280 feet per mile) of PVC pipe and 35 inch-miles of other pipe materials. The unit of inch-mile is an accepted way to present the amount of I/I in collection systems.

BASIN 2 is located in the central area of the City of Chehalis and includes the downtown core. Topography in Basin 2 is relatively flat in the main central area with a steep slope up along the northeast side. This Basin serves approximately 310 total acres, which includes residential, commercial and industrial land uses. Basins 3, 4 and 10 all drain into Basin 2.

Collection system piping includes about 62,000 feet of pipe, which is mostly 8-inch PVC. There is a main PVC gravity interceptor on the west side of the basin. That interceptor flows to the north and varies in size from 15 to 27 inches. This basin is a conglomerate of six of the nine sub-basins recently rehabilitated with PVC pipe. Of the 112 inch-miles of pipe in the basin, 106 inch-miles are PVC.

BASIN 3 is the smallest basin in the City of Chehalis and flows into Basin 2. It serves approximately 45 acres of predominantly residential land use. There is a small portion of commercial land use in the basin along Market Street. This Basin is at the base of a hill and slopes up to Dobson Park. Land slope gradually increases from a slight grade up to steep unbuildable slopes.

The collection system consists of 9,800 total feet of 6 and 8 –inch concrete pipe. This basin has a total of 13 inch-miles of concrete pipe.

BASIN 4 is located on the south end of the City of Chehalis and includes Henderson Park, WF West High School and Valley View area to the Northeast. Basin 4 serves about 220 acres of predominantly residential land use with some commercial land use along Market and 12th Streets. The high elevations in the basin are around the Valley View area on the northeast side. Flow direction in the basin is southwest to the interceptor on the western side of the basin.

There are approximately 30,000 feet of collection system piping in Basin 4 ranging in diameter from 6-inch up to 15-inch. Pipe materials are predominantly concrete with a minimal amount of 15-inch PVC pipe. There are a total of 48 inch-miles of sewer pipe and only 2 inch-miles are PVC pipe. The main interceptor, along the western side, is comprised of about 3,000 feet of 15-inch and 550 feet of 10-inch pipe. The east west interceptor, which travels in the alley between 13th and 14th and along Mills Avenue, is comprised of about 2,300 feet of 10-inch sewer pipe.

BASIN 5 starts along Interstate 5 at the Napavine City Limits, extends north to Parkland Dr. and includes the Port of Chehalis. The current area served by this basin is a narrow strip of land surrounding the Napavine/Lewis County Sewer District No. 1 interceptor and Maurin Road, which is about 365 acres. Land use in the basin is designated as commercial and industrial, with some residential along Bishop Road east of Rush Road. This basin is extremely flat with some elevation drop between Napavine and the Rush Road area.

The collection system consists of only interceptor sewers. The total length of pipe is 29,000 feet of 12-inch up to 18-inch. The Napavine/LCSD No. 1 interceptor is all rubber gasketed concrete sewer pipe that was constructed in 1978 and includes 51 inch-miles of pipe. The Maurin Road (Port of Chehalis) interceptor was constructed in 1995. The Maurin Road interceptor is all 12-inch PVC and is about 6,150 feet long accounting for 14 inch-miles of PVC pipe.

BASIN 6 includes the City of Chehalis Industrial Park and an area north along Jackson Highway from Kennicott Road to 21st Street. The basin serves approximately 420 acres of relatively flat land except for the steep slope on the northeastern side east of Market Street. Land use in the basin is predominantly industrial and commercial with a small amount of low density residential area on the east side of Market Street. Sewage flows to the west in the basin and drains into Basin 5.

This basin consists of approximately 29,125 feet of sewer mains, most of which are PVC. Most of these lines are 8-inch with about 7,000 feet of 12 and 15-inch. There are 51 total inch-miles of pipe in the basin.

BASIN 7 is the area currently served by Lewis County Sewer District No. 1 (LCSD). It was constructed in 1978 in response to failing septic systems in the area. The current service area is 170 acres of predominantly residential with very little commercial land use. The basin is relatively flat along Jackson Highway and has Logan Street Pump Station on the northern end. The basin flows to the west and discharges into Basin 5.

There is approximately 28,000 feet of sewer main in Basin 7 and is mostly 10-inch with some 8-inch diameter pipe. Most of the collection system was constructed in 1978. The 10-inch interceptor from the District is rubber gasketed concrete sewer pipe and the remaining sewer system within the District is PVC pipe. There is a total of 50 inch-miles of pipe in Basin 7.

BASIN 8 serves the City of Napavine, which is the southern most portion of the sewer system. There are about 385 acres in Basin 8. Land use in the basin includes residential, commercial and industrial and is governed by the City of Napavine. This basin has some topographical relief and has four pump stations throughout the basin.

The oldest portions of this basin were constructed in 1978 using gasketed concrete sewer pipe. There are about 33,000 feet of 8 and 10-inch gasketed concrete sewer pipe and about 7,200 feet of new 8-inch PVC pipe. The basin has 11 inch-miles of PVC pipe and a total of 63 inch-miles of pipe.

BASIN 9 is located in the central part of the City of Chehalis. Topography in the basin is flat and sewage flow is east to west. The basin covers about 42 acres that includes commercial, industrial and minimal residential land use designations.

Basin 9 was rehabilitated in 1991 and now consists of 10,000 feet of 8-inch and about 1,000 feet of 10-inch PVC sewer pipe. There is a total of 18 inch-miles of PVC pipe in this basin.

BASIN 10 is in the southern part of the City of Chehalis and serves the area south of WF West High School. This basin serves approximately 205 acres of residential and commercial land use. Topography in the basin slopes to the northeast and sewage drains into Basin 4 near WF West High School.

Collection system piping in Basin 10 consists of 8, 10 and 15-inch concrete pipe constructed prior to the 1970's. There is about 25,000 feet of 8-inch, 1,000 feet of 10-inch and 2,000 feet of 15-inch pipe for a total of about 28,000 feet and 44 inch-miles.

BASIN 11 includes the area on the western side of the freeway north of Basin 5 and a portion of land on the eastern side of I-5 in the Parkland Drive vicinity, including Green Hills School. The Basin is bordered by the Chehalis River and includes the surrounding floodplain on the eastern side of the river. Flows in this basin all drain into the Riverside pump station and are pumped directly to the WWTP. There is a total of 78 acres in Basin 11. Approximately 48 acres are designated for industrial and commercial land use and about 29 acres are designated for residential land use. The other land is floodplain or wetland.

Collection system piping in Basin 11 consists of the main Napavine/LCSD interceptor and small diameter collection lines. The 9,500 feet of interceptor is rubber gasketed concrete sewer pipe and varies from 18 to 21 inches in diameter. The 11,700 feet of collector mains are 8-inch diameter and are PVC and concrete. There is a total of 55-inch miles, with PVC accounting for 8 inch-miles.

BASIN 12 includes the area around the WWTP, land to the north along the west side of Interstate 5, and an area around the Prindle Street pump station. This basin covers 170 acres of commercial, industrial and residential land uses. It includes Wal-Mart, the regional airport and Prindle Street pump stations. Most of the flow in this basin drains to the Prindle pump station, with the exception of the area west of the WWTP which is pumped directly into the WWTP.

Collection system piping varies in size from 8-inch up to the 27-inch main discharging into the Prindle Street pump station. There is a total of 15,650 feet of gravity main, of which about 13,000 feet of 8-inch diameter. There is a total of 27 inch-miles of pipe, which includes 19 inch-miles of PVC pipe. The force mains are discussed with their associated pump station.

FLOW MONITORING PROGRAM (1998)

One of the key elements of this General Sewer Plan is to complete a flow monitoring program of the entire collection system. The purpose of the flow monitoring program is to locate those areas (basins) of the system that contribute the greatest amount of infiltration and inflow (I/I) to the WWTP and to verify the effectiveness of previous I/I reduction efforts. To accomplish this, flow monitors were installed in ten of the twelve basins previously identified. Two of the basins are actually fragments of various sewer lines that fall outside of the flow monitoring basins. Because of the cost to collect and process flow data, the flow monitoring program must be limited by both the number of monitors and the length of time the monitors are installed.

The collection system was divided into twelve basins. The boundaries of the basins were set using data developed from existing sanitary sewer maps, storm drainage maps, discussion with city staff, past monitoring and planning efforts, physical inspections and Gibbs & Olson's extensive knowledge of the sewer system. These basins are shown in Figure VI-1. The flow monitoring program also included the installation of a continuously recording tipping bucket rain gauge to allow flow in the system to be correlated against rainfall. The flow monitors and rain gauge were calibrated for their particular location and collected data at 5-minute intervals over the 3-month flow monitoring period.

Infiltration and inflow can be divided into various categories. The effects of these types of I/I can be seen in the various flow graphs presented later in this report. In general, inflow is considered to be stormwater that enters the sewer system during rainfall events. This inflow comes from such sources as roof drains, area drains, basement sump pumps, foundation drains, catch basins and storm system defects that are directly connected to the sewer system. The effect of inflow is a rapid rise in sewer system flows in response to rainfall, but then drop off almost immediately when the rain stops.

Infiltration is generally described as groundwater that enters the sewer system through defective pipe or manhole joints and old deteriorated sewer lines. These lines consist of both mainline sewers owned by the sewer utility, as well as, sidesewers on private property. Infiltration can be categorized as three types: 1) "immediate infiltration" - This is infiltration that causes the sewer system to experience a rapid increase in flows in response to a storm event, 2) "near term infiltration" - This is infiltration that enters the sewer system over a period of days or weeks after a storm event, and 3) "long term infiltration" - This is infiltration that enters the sewer system slowly and is influenced by rainfall from several previous months. Generally, the "long term infiltration" shows an annual cycle with significant increases during the November-March period followed by gradual reductions through spring, summer and fall.

In general, mainline and sidesewer pipe installed prior to 1960 are significant sources of all three “types” of infiltration. Private sidesewers installed in the 1960's and early 1970's also contribute large amounts of infiltration while rubber gasketed mainlines and sidesewers installed after the early 1970's usually contribute very little infiltration. Soil type and the actual level of groundwater within a basin also impact infiltration. Those basins with well drained sandy soil and/or a groundwater level below the pipe show lower levels of infiltration even if the basin contains older pipe than those basins with older pipe that have soils that tend to become saturated during the wet season and/or have a high groundwater table. Other factors such as illegal connections of roof, foundation, basement and area drains, topography and installation quality also impact the level of I/I from any given basin.

As mentioned above, significant sources of infiltration are most frequently experienced from sewer lines installed prior to about 1960. These older mainline sewer pipes and associated sidesewers on private property were most commonly installed using cement grouted joints or tar filled joints, both of which fail dramatically over time even if they were installed under ideal conditions. As such, the continued deterioration of the lines allow the system to act as an underdrain system for groundwater. Typical flow patterns from basins with pre-1960's pipe show a dramatic increase in flow during a rain storm that is caused by "immediate infiltration". This peak flow is then followed by several days, and in some cases two to three weeks of gradually decreasing flows caused by "near term infiltration". These basins also show a significant increase in "base flow" from summer to winter which is the result of the chronic infiltration of groundwater or "long term" infiltration that takes months to taper off.

In basins where significant development occurred during the 1960's, mainline sewer pipe is generally concrete with rubber gasketed joints. These joints show a low failure rate (usually between 1-4%) and a corresponding low infiltration rate. However, during the 1960's, the most common pipe used for sidesewers is what is referred to as "fiber pipe". This pipe is essentially tar impregnated paper, rolled and pressed under high pressure into a pipe. Typically, such pipe shows 85-100% failure rates and is a significant source of infiltration.

Most mainline sewers installed from the mid-1970's through the present consist of rubber gasketed joint Polyvinylchloride (PVC) pipe (i.e. plastic pipe). Although sewer lines, even new lines, cannot be made "water tight", PVC pipe shows low infiltration rates, when properly installed.

SELECTED FLOW MONITORING SITES

Ideally, a flow monitoring basin in a system the size of the Chehalis, Napavine, LCSD No. 1 system, and utilizing ten monitors, will consist of approximately 30,000 to 40,000 linear feet (L.F.) of gravity sewer main plus their associated side sewers. In reality, the size of a basin is dictated by how the land slopes, how the sewer system is installed, the type and age of pipe in the basin and, of course, the desire to control the cost of flow monitoring. The desirable characteristics of manholes selected for flow monitoring include:

- Straight through main sewer line, (i.e. no bends and no more than one pipe entering and only one pipe leaving the manhole).
- Good hydraulic characteristics, preferably laminar (non-turbulent) flow throughout the range of flows experienced in the manhole.
- Influent and effluent pipe of relatively flat grade.
- Good velocity, preferably in the 2-4 foot per second range.
- Sufficient depth at all times to cover the flow monitoring probe, preferably greater than 2 inches.
- Low traffic control requirements to increase safety and reduce field crew size for calibration/data collection requirements.

Minimizing the amount of pipe that is not monitored as well as minimizing the situation where flow data from one site is dependent on flow data from other basins are important considerations in developing a monitoring basin layout and monitoring program. Where basin flow data are dependent on each other, flows from one or more upstream monitoring sites may need to be subtracted from a given monitoring site to determine the net flow from the basin under consideration. Likewise, flows from two or more monitoring sites may need to be added in order to derive total flow from a single basin. Although it is desirable to avoid combining flow data from multiple basins or having pipe in the system that cannot be monitored, in reality such situations cannot be avoided if flow monitoring costs are to be controlled. Due to the configuration of the collection system and the limited number of monitors, approximately 77 in-miles of pipe (about 13%) within the system were not monitored.

Bypass or overflow locations require special attention. Limited manipulations and accurate records of system alterations (i.e. plugging lines so that I/I cannot flow from one basin into another without being monitored) must be kept in order for the system analyst to understand the flow conditions. For this monitoring program, no plugs were necessary. The Parkland Street bypass (which allows high flows to be diverted from the Prindle Street pump station to the Riverside Road pump station) was kept in its normal operating position throughout the flow monitoring period. However, two monitors had to be installed at this location to insure any bypass was measured. During the monitoring period, no flows were observed to overtop the slide gates at the Parkland street overflow structure.

This monitoring program consisted of the installation of ten monitors to measure flows in nine of the twelve basins. During the first three weeks of the program a monitor was installed at the location of monitor 9 in order to determine a base flow and establish the magnitude of I/I within this previously rehabilitated basin. For the balance of the monitoring period, this monitor was moved to the location of monitor 10, in order to divide the unrehabilitated basin 4 into two smaller basins. Throughout the monitoring period a rain gauge was also installed in the Chehalis Public Works Department equipment storage area. A schematic diagram of the monitoring sites showing the relationships among all monitors is shown in Figure VI-2. Table VI - 2 shows the location of the monitors.

FLOW MONITORING AND RAIN GAUGE EQUIPMENT

The monitors installed are manufactured by ADS Environmental Services, Inc., whose home office is in Huntsville, Alabama. The monitors were portable, battery operated units designed to be installed in manholes. Each unit measures and records depth and velocity of flow in user

INSERT FIGURE VI-2 CHEHALIS 1998 FLOW MONITORING SCHEMATIC DIAGRAM

TABLE VI-2 LOCATION OF MONITORS		
BASIN NUMBER	**MONITOR LOCATION**	**PIPE SIZE (INCHES)**
1	In R-R grade east of 616 Hawthorne	15
2	West shoulder of Quincy Avenue across street from 250 Quincy	27
3	In intersection of alley and 6th Ave. in front of 536 6TH Avenue	8
4A	In grassy area near trees West of intersection of Johnson and 13th Avenue	15
4B	In Recreation Park, two manholes downstream of Parkland Street overflow	15
5	West of R-R tracks & north of intersection of Kelley Road and Thomsen Street	18
6	In gravel parking area in front of 289 Interstate Avenue	15
7	In LCSD No. 1 Interceptor, one manhole upstream of Rush Road and Oechli Road intersection	10
8	In front of 1409 Rush Road, Napavine	10
9	Southeast corner of intersection of Prindle Street and State Street	10
10	WF West High School Athletic Field	15

defined increments of time. During the course of this program, data was collected at 5-minute intervals (288 data points per day per sensor). The monitor probes consist of a four-sensor depth probe, installed at the crown of the pipe, and a single sensor velocity probe installed in the flow of wastewater. The four individual sensors on the ultra sonic depth probe measure depth of flow four separate, independent times for each 5-minute interval. The "final" depth is usually calculated as the average of the four sensor readings. The velocity probe measures the velocity of flow at 5-minute intervals at the same time when the depth readings are taken.

Data collected by the rain gauge was in 0.01 inch increments and was date/time stamped in 5-minute intervals. The data were used to correlate rain with observed flows at each of the flow monitoring sites.

MONITORING PERIOD

The timing of flow monitoring projects in Western Washington generally extends for about 6 months from about mid-October through March or April. This period is of greatest interest because the groundwater is generally high, the soils are saturated, and the area experiences its heaviest rainfalls. It is important that the monitoring period be sufficiently long to insure the majority of the high flows are measured. However, because of the cost to collect and evaluate data, it is equally important that the monitoring period be as short as possible. Because of budget constraints, the monitoring period was reduced to a 3-month period, mid January through mid April 1998.

RAINFALL EVALUATION

During a flow monitoring period, it is hoped that the area under evaluation receives rainfall that is above normal for the time of year under study. Unfortunately, the selected monitoring period did not yield the amount of rainfall that results in unusually large flows from the basins. A review of 67 years of historical rainfall records as measured in the neighboring City of Centralia reveals the area received about average rainfall.

During the monitoring period, no 1-day storm occurred that resulted in rainfall exceeding 1-inch, a storm that is common in the area. It is unlikely that any unusual storm event (say 5, 10, 25, 50 or 100 year event) will occur during a short flow monitoring. Table VI-3 provides information showing the likelihood of a major storm occurring in any given year.

TABLE VI-3
PERCENT CHANCE OF A MAJOR STORM OCCURRING DURING A GIVEN YEAR

Storm Event	Percent Chance of a Storm Occurring in Any Given Year
5-year	20%
10-year	10%
25-year	4%
50-year	2%

In other words Table VI-3 shows that there is an 80%-98% chance that a major storm will not occur during a flow monitoring period. Because of the low chance of a major storm occurring and the resulting flows being evaluated directly, it is common to use actual flow data from lesser storms to project high flows that occur when heavy rainfall occurs.

FLOW DATA GATHERING, PROCESSING AND EVALUATION

The first six flow data collection visits were dedicated to calibrating each flow monitoring site by confirming depth and velocity readings with portable measuring devices. Based on the calibrated readings, a Manning's hydraulic coefficient was calculated for each site. Once calibrated, the monitor's data was collected and the monitor and site maintained during site visits which occurred at a minimum interval of nine days. In the field, the flow data was uploaded from the monitor into a portable computer and then transported to the office where it was uploaded into an office computer for processing. The data was first processed using a software program developed by ADS. The manufacturer's software uses the depth and velocity readings to calculate flows for each 5-minute increment. The flows can be calculated by using either the depth and velocity readings or by using only the depth and the calibrated Manning's coefficient for each site. These calculated flows can then be expressed in 5 minute, 15 minute, 30 minute, 1 hour, or 24 hour flow values. After flows were calculated using the manufacturer's software, a computer file was generated using Microsoft Excel. That file contained the date/time, depth, velocity, flow and rainfall for each monitor for each time period under consideration. This data was then used for further analysis and to generate various graphs and tables including the flow versus rainfall graphs and tables shown in this report.

To help insure that the data collected was accurate and to minimize the amount of editing in the office, several procedures were developed to identify problems early. These procedures included:

- The crew made site visits to each monitor at a minimum of every nine days.

- During each site visit, current depth and velocity data were compared to historic data collected from that site. If there was either an abrupt or subtle change which could not be accounted for, then manual depth and velocity readings were taken and compared to the instantaneous readings as measured by the monitor. If this comparison verified the monitor to be inaccurate, it was immediately removed, recalibrated and reinstalled. If it could not be repaired on-site, a backup monitor was installed. During the site visit, the depth and velocity probes were cleaned as required and the site was inspected for changed conditions such as silt or debris build-up, plugs or signs of surcharged conditions, and/or whether the probes were loose from their attachment device.
- Continuously updated velocity and depth graphs were generated for each monitor. These graphs were used in the field throughout the entire flow monitoring period. The graphs provided a valuable reference source to insure all probes were functioning properly and were not slowly drifting out of calibration.
  - Once collected, the data was again evaluated in the office and graphs generated that showed rainfall, depth and velocity over time. This information compared historic data with current data and was helpful in spotting problems with either the depth or velocity probes, the monitor itself or the site.

Figure VI-3 is provided to show a week (March 20-26) of typical raw data. This data is for Basin 10 and shows the actual data for each of the four depth sensors (U12, U13 U2 and U7), the velocity probe and rainfall. Since this is raw data, flow is not included on this graph. Several interesting points can be seen on the graph.

1) A typical diurnal curve, for a basin comprised of mostly residential structures, is observed on March 20, 1998.
2) The basin experiences a rapid increase in flow due to small amounts of rain, indicating at least some inflow of stormwater as observed at about noon on March 22, 1998.
3) Depth of flow on March 23, 1998 was above the depth sensor, indicating the pipe was flowing at a depth that was within 1-inch of the top of pipe and quite possibly surcharged.

INSERT FIGURE VI-3 TYPICAL RAW DATA

DATA SUMMARY

The data obtained from the flow monitoring program forms the basis for subsequent evaluations of the sewer system and ultimately the recommendations for implementing possible basin rehabilitation projects that the City may want or need to undertake. Several factors combined to rate the data collected as excellent. These factors include equipment selection, frequency of site visits, personnel attentiveness, good site calibration, editing techniques and software capabilities.

Figure VI-4 compares the total daily flow as measured in the flow monitoring basins to the daily effluent flow as measured at the WWTP for the entire monitoring period. As the graph shows, there is a high degree of correlation throughout the range of flows monitored. On average, the sum of daily total flow from the monitors is within 1.5% of the flow measured at the WWTP.

As previously mentioned Basin 11 and 12 were unmonitored and account for approximately 17% of the inch-miles of pipe in the collection system. These pipes were constructed recently and include the K-Mart, Wal-Mart, Airport and Riverside Road areas, along with the rehabilitated sewers of Green Hill School, the 21-inch LCSD No.1/Napavine interceptor north of 13th Avenue and the 5-inch line and pump station along Shoreline Drive. Comparison of total monitored flow with WWTP effluent shows good correlation. This analysis leads to the conclusion that the unmonitored portion of the system has little base flow and has limited impact from rainfall during storm events.

DESCRIPTION OF BASIN FLOW GRAPH

Because of the differences in quantity of flow measured at each site, it is necessary to plot the graphs showing each monitor's flow using an appropriate flow range on the graph. The reader is cautioned to observe this scale (the maximum range is 0.00 to 2.00 MGD; the minimum is 0.00 to 0.12 MGD). Because of the different scales, what may appear to be dramatic change in flow may actually be no more than a weekly variation in the waste flow. In reviewing the

INSERT FIGURE VI-4 TOTAL DAILY FLOW

flow graphs, it is important to remember that no conclusions can be made concerning I/I removal in any basin based solely on the graphs. A basin may show a substantial response to rain, however, a review of the inventory data may also show the basin is very large and as such, the cost to remove the observed I/I may be prohibitively expensive. Only after a thorough analysis of flows, inventory and cost data for all basins coupled with a prioritization of the basins and a cost comparison of I/I removal versus treatment cost associated with WWTP upgrades, can decisions be made relative to which basins are cost effective to rehabilitate. In the basins that are subject to I/I, the effects of the stormwater can be seen in the following flow graphs.

Figure VI-5 shows the flows and rain versus time as measured from Basin 1. This basin is comprised of industrial/commercial buildings along with residential homes. About 50% of the basin's pipes have been replaced with PVC. The flows observed during the five major rainstorms indicate the basin is affected by rain, resulting in about a 460% increase in flows from a winter base flow of 0.15 MGD to observed peaks of about 0.84 MGD. As can be seen on the graph, the high flows are generally delayed a day or two after the storm events. Analysis of this graph also shows the effect of near term infiltration as seen by the elevated flow values for several days after the storm.

Figure VI-6 graphs the flows and rainfall versus time from Basin 2. This mostly residential basin has undergone substantial rehabilitation using PVC pipe. Flows during the five major rainstorms indicate an approximate 80% increase in flow from a winter base of 0.54 MGD to about 1.0 MGD. A pattern of flows during and after rainstorms is unique in this basin when compared to other basins with I/I. In other basins, the amount of flow from each rainstorm varies linearly with the amount of rain (i.e. the more rain the more flow). In this basin, flows peaked at about 1.0 MGD in each of five separate rainfall events, regardless of the amount of rainfall. This relatively large basin has been rehabilitated (including main lines, manholes and sidesewers to the building), but still shows some signs of I/I. The primary source of the I/I is likely basement or under the house sump pumps, or roof drains that have been reconnected to the sewer system.

INSERT FIGURE VI-5

INSERT FIGURE VI-6

Figure VI-7 shows the flows and rain versus time for Basin 3. This basin is relatively small, comprised entirely of non-PVC pipe and is entirely residential. The basin's winter base flow is about 0.04 MGD. The flow increases by as much as 630% to about 0.27 MGD during heavy rains. Again, the rain precedes the high flows by about a day. The flows from this basin generally take 4-6 days to return to the base flow values indicating the majority of the increased flow is due to infiltration.

Figure VI-8 is a graph of the flow and rain versus time for Basin 4. This basin is comprised of mostly residential households, the majority of which are served by pipes that pre-date PVC. The winter base flow is estimated at 0.14 MGD. During heavy rains, the flow can increase about 500% to approximately 0.8 MGD. During review of the graph, the reader is cautioned as to the values reported during the January 21-23 storm. The net flow from this basin is calculated by subtracting flows from Basin 10 from the flows at Basin 4. During the early portion of the monitoring period, monitor 10 was not installed, therefore, the amount monitored during the January 21-23 storm is actually the amount attributable to both Basins 4 and 10. The flows from this basin generally peak a day after a major storm and gradually tapers off over a 7-10 day period indicating the majority of the increased flow is due to infiltration.

Figure VI-9 plots flow and rain versus time for Basin 5. The winter base flow of 0.050 MGD can double to about 0.1 MGD with rainfall. The bulk of this basin's inventory is comprised of the Chehalis/Napavine/LCSD No. 1 interceptor. The basin has relatively few sidesewers and the majority of the sidesewers are from commercial/industrial buildings in the Port of Chehalis. The recorded base flow from the basin is an indication of the land use type, with low flows generally occurring on Saturdays and Sundays. The flows from this basin generally take 1-3 days to respond to a rainstorm and then 2-4 days to return to pre-rainfall conditions which indicates the majority of the increase is from infiltration.

INSERT FIGURE VI-7

INSERT FIGURE VI-8

INSERT FIGURE VI-9

Figure VI-10 shows the flow and rain versus time from Basin 6. This basin provides sewer service to the Chehalis Industrial Park and newly constructed extensions in the Wallace-Jackson neighborhood. The winter base flow observed is calculated at 0.11 MGD and peak I/I was observed at about 0.19 MGD or an 80% increase in flow. Because this basin is largely Industrial/Commercial, weekly low flows occur on the weekends with flows increasing during the week. This weekly flow pattern can be observed on the graph. The basin shows very little impact from I/I. The increase and drop in flow during the major three storms may be as result of inflow.

Figure VI-11 graphs the flow and rain versus time from Basin 7 (LCSD No. 1). The winter base flow of about 0.04 MGD from this largely residential neighborhood increases by as much as 0.18 MGD to 0.22 MGD or about 400%. As can be seen from the graph, this system takes 1-2 days to respond to a rainfall event followed by 10-15 days before the high flows taper off and return to pre-rainfall conditions. Again, this demonstrates a system in which the majority of the I/I is infiltration.

Figure VI-12 shows the flow and rain versus time from Basin 8 (Napavine). The observed winter base flow is about 0.18 MGD. Following rainstorms the flows can increase to about 0.40 MGD or a 175% increase. This system takes 2-3 days before it reaches its peak flow following a storm event and then takes 12-17 days for the high flows to taper off and return to pre-rainfall conditions. The majority of I/I in this system is from infiltration.

Figure VI-13 shows the flow and rain versus time from Basin 9. This largely rehabilitated basin in the Chehalis downtown area is comprised mostly of commercial establishments and has winter base flows of about 0.04 MGD. The basin can experience increases in flow of about 100% during heavy rains. No conclusions can be drawn from the limited data.

INSERT FIGURE VI-10

INSERT FIGURE VI-11

INSERT FIGURE VI-12

INSERT FIGURE VI-13

Figure VI-14 shows the flow and rain versus time for Basin 10. This basin is comprised of mostly residential structures which are sewered by pipes that pre-date PVC material. The basin has a winter base flow of about 0.15 MGD. The high I/I flow was observed to peak at about 0.75

MGD or a 400% increase. An important note on this graph is the lack of flows recorded during the first 3 weeks of the monitoring period. During this time, flows from this basin were measured at monitor 4 which measured a value of about 2 MGD during the January 21-23 storm. Several factors combine to lead to the estimation that this basin would have generated about 1 MGD of flow during the January 21-23 rainstorm; 1) both basins sewer similar neighborhoods, 2) the basin inventory of the inch-mile of pipe are nearly equal, 3) the pipes are of the same age and 4) during smaller storms the basins yield about the same flows (about 0.8 MGD on February 21 and March 23). High I/I flows occur within 1-2 days following a rainstorm then take about 5-8 days to taper off to pre-rainfall flow. This indicates that the majority of I/I in this basin is from infiltration.

EXISTING AND PROJECTED I/I FLOWS

The preceding basin-by-basin description and flow data outlined major points to be observed in the flow graphs. Table VI-4 is provided to summarize the flows recorded in the basins during six large storm events during the monitoring period and to estimate flows that will occur from each basin during a storm similar to the event that occurred on February 7, 1996 and resulted in the maximum observed peak wet weather flow of 12 MGD at the WWTP. The table also shows the percentage increase of flow in each basin during the monitored rainstorms and an estimated dry weather (summer) base flow for each basin.

The summer base flows were calculated by distributing the average daily dry weather flow of 1.15 MGD among the basins at the same percentages as the winter base flows that were observed during the monitoring period.

The estimated I/I flows for each basin for February 7, 1996 are calculated by developing a “peak to observed I/I conversion factor” which is calculated by dividing the peak I/I flow

INSERT FIGURE VI-14

INSERT TABLE VI-4

recorded at the WWTP on February 7, 1996 (10.85 MGD) by the highest I/I recorded by the monitors on March 23, 1998 (3.06 MGD). That conversion factor is calculated as $\frac{10.85}{3.06} = 3.55$.

The I/I for the February 7, 1996 storm is then calculated for each basin by multiplying the observed I/I during March 23, 1998 by the conversion factor. The resulting peak I/I flows for each basin are shown in Table VI-4.

As can be seen in the basin flow graphs and Table VI-4, some basins have considerably more response to rainfall than others. These basins are largely comprised of older pipe. This older pipe was generally constructed of concrete or clay pipe, have inflow sources directly connected, have deteriorated, thereby causing cracks and breaks, and have been subjected to root intrusion. These basins, No.'s 1, 3, 4 and 10 show I/I projections 14 to 20 times greater than their base flows during the February 7, 1996 storm. That storm ranks as the highest 1-day storm of the past 11 years at 3.90 inches of rain. Also, for the same 11 years, that storm ranks highest when adding the previous 2 or 3 days of rain.

Analysis of observed flows during the monitoring period reveals these basins contribute the greatest percentage of I/I and that percentage tends to increase during increasing large storm events. Table VI-5 shows the rainfall observed for the specific storms and the percentage of total system I/I contributed by these older basins.

TABLE VI-5
I/I CONTRIBUTION

Observed Storm	Observed Rainfall	% of Total System I/I Contributed By Basin				
		Basin 1	Basin 3	Basin 4	Basin 10	Sum
1	2.39	18%	6%	22%	22%	68%
6	1.78	20%	7%	23%	21%	71%
3	1.29	12%	4%	24%	22%	63%
4	1.16	19%	9%	20%	18%	66%
5	1.05	15%	9%	18%	16%	57%
2	0.89	9%	2%	26%	20%	57%
AVERAGES		16%	6%	22%	20%	64%

As shown in Table IV-5, these four basins contribute about 64% of the total system I/I for all storms monitored and because this percentage increases with heavier rainstorms, it is likely these basins can contribute in excess of 70% of the total system I/I during even larger storms—those that create problems at the WWTP.

As shown in Table VI-1, these four basins are comprised primarily of pre-PVC pipe. The total inch-miles of pipe in these basins equals about 171 inch-miles of which 139 inch-miles are constructed of non-PVC materials. These 139 inch-miles amount to 22.6% of the total inch-miles in the system. When the amount of I/I contribution from these basins is considered along with the number of inch-miles in the system, one can say that approximately 70% of the system I/I is contributed by 23% of pipes in the system.

Table VI-1 of this report shows Basin 2 to consist of 61,903 linear feet of pipe of which 58,453 feet is constructed with PVC pipe. This amounts to about 94% of the pipe in the basin. The result of this flow monitoring study indicates the percentage of total system I/I contributed by this basin is about 24%. This means that 18% of the pipe contributes about 24% of the I/I. This is about a 50% reduction in the amount of the I/I projected from this same area of the City in the 1988 engineering report “Sewer System Rehabilitation”.

Based on basin characteristics, i.e.) age and type of pipe, observed and projected flows along with the number of inch-miles in the basins; Basins, 1, 3, 4 and 10 are candidates for I/I removal consideration.

HISTORY OF I/I REMOVAL WORK IN THE CITY OF CHEHALIS

The City of Chehalis has a long history of I/I removal from its sewer system. The history begins in 1977 when the City undertook rehabilitation work identified in its then current facility plan. All possible means to locate defective system components have been employed including cleaning, TV inspection, smoke testing and dye testing. Construction and rehabilitation work included manhole and pipe sealing and repair, insituform lining, pipe bursting, inpipe replacement, HDPE slip lining and total basin rehabilitation. Since 1979, the City has used city crews and equipment to locate and reconstruct isolated sections of pipe that were the most defective but not within specific basins slated for replacement. In each of the four years of 1981 through 1984, $108,000 was budgeted for this work. In 1985, this budget was increased to $200,000 yearly. In 1987, the City increased the amount funded for I/I work to $300,000 per year. Their I/I removal program included adjusting this funding level for inflation over time and to continue the program for 40-50 years. Although other cities have since been allowed to implement such a program, DOE in 1988 did not feel Chehalis' program was aggressive enough. Therefore, in 1988 the city entered into a two phase I/I removal program with an estimated goal of removing 44% of the I/I within five years as the first phase. In 1993 an I/I report was prepared that indicated the City had met their goal of 44% I/I removal at about $3 million under budget. The City requested EPA and DOE allow the City to use the $3 million for work on Phase 2. However, because the City had met their goal of 44% I/I removal, EPA made a determination that they would not amend the grant to fund any additional work. DOE did allow the City to use the remaining $1 million for additional work, which the City completed in 1995 and 1996.

In September 1988, the City of Chehalis' engineering report, "Sewer System Rehabilitation" outlined a basin-by-basin approach to I/I rehabilitation. Table VI-1 of that report is reprinted as Table VI-6 and lists among other attributes, the basins of the system, the estimated peak I/I by basin and the estimated cost to rehabilitate the basins.

The first column of Table VI-6 shows the basin numbers of this report as a cross-reference to the basin numbers used in the 1988 report. As can be seen in the table, the Estimated Table Peak I/I before rehabilitation (GPD) as estimated in 1988, totals 13,064,106 GPD for a 10-year storm of 3-inches in 24 hours. Table VI-4 shows the 1998 Total Estimated I/I for February 7, 1996 (with 3.90-inches of rainfall) to be 10,850,000 GPD. This means the City's I/I removal program has successfully removed about 2,800,000 gallons of I/I during major storm events.

The bulk of the I/I removal work has been done in the current Basin 2. Table VI-6 shows the 1988 estimated I/I contribution for Basin 2 to total 4,247,263 GPD. Table VI-4 shows the

INSERT TABLE VI-6 1998 SEWER REHABILITATION PROGRAM

1998 estimated I/I contribution to be 2,030,000 GPD. This analysis shows the reduction of I/I resulting from the rehabilitation of Basin 2 to remove 2,217,263 GPD or about a 50% removal of the 1988 levels of I/I. The results of future I/I reduction work through basin wide rehabilitation is projected to be a 50% reduction of I/I.

The rehabilitated basins costs and locations, when cross-referenced to this report's basin numbering system are highlighted in Table VI-6. The table shows the City of Chehalis has rehabilitated 70,231 linear feet of sewer main and all associated sidesewers. The total amount spent to replace 51,676 linear feet of sewer main in this basin was $4,457,438. The total also shows the construction costs for each completed project. Other costs, not included in this table are: design, surveying, legal, administration, construction management and inspection. These costs combined add about 30% to the project construction cost. Side sewer replacement from the property line to the home was done by the property owner at the property owner's expense.

I/I REMOVAL COSTS

This report projects a 50% reduction of I/I for any future basin-wide rehabilitation work based on results of past rehabilitation projects and the fact that construction techniques, materials and testing have remained constant since the rehabilitation program started ten years ago.

Table VI-7 lists all the basins in which basin-by-basin rehabilitation will be effective. The order that the work should proceed is based on the cost to rehabilitate the basin and the amount of I/I expected to be removed. This is expressed as a cost per gallon of I/I removed. Using a historical reduction of I/I of about 50%, the City should anticipate about the same performance in future projects. The cost are in 1998 dollars and are based on a total design and construction cost of $150 per linear foot of sewer pipe replaced as shown below:

1998 Construction Cost	$100/L.F.
Construction Contingency @ 10%	$10/L.F.
Administration, Design, Engineering and Legal @ 35%	<u>$38.50</u>
Total	$149

TABLE VI-7 CITY OF CHEHALIS GENERAL SEWER PLAN 1998 SEWER REHABILITATION PROGRAM					
1998 Basin No.	Length of Non-PVC Pipe (Feet)	Estimated Total Peak I/I (GPD)*	Estimated Total Peak I/I Removed (GPD)*	Total Estimated Rehabilitation Cost (@$150/LF)	Total Cost per Gallon I/I Removed ($/gallon of I/I Removed)
1	24,250	2,090,000	1,045,000	$3,637,500	$3.48
3	9,800	750,000	375,000	$1,470,000	$3.92
4	28,500	2,390,000	1,195,000	$4,275,000	$3.58
10	27,600	2,230,000	1,115,000	$4,140,000	$3.71

*Assumes 50% reduction in Peak Day I/I through replacement with PVC Pipe

FUTURE FLOWS AND COLLECTION SYSTEM NEEDS

Future flows identified in Section V and ultimate flows developed in this section will be allocated based upon the amount of land (within each of the basins previously described) for each land use designation. It assumed the following development potential for the various land uses:

High Density Residential – twenty homes per acre
Medium Density Residential – ten homes per acre
Low Density Residential – five homes per acre
Commercial – equal to Medium Density Residential
Industrial – 6 persons per acre and 105 gallons per person (per Port of Chehalis Comprehensive Plan)

The area of each land use was measured for each basin and ultimate flow projections were made. Those projections were then reduced to 14 percent of development potential to equal the projected year 2025 flow increase of 0.78 MGD. Flow allocations for the year 2050, assumed the area between current boundaries and 2025 boundaries will become 25 percent developed and the remaining flow will be from areas outside the 2025 boundaries. This resulted in 15 percent development of the area outside the 2025 boundaries by the year 2025 to result in an estimated 1.93 MGD increase in flow growth between current and the year 2050 flows. Table VI-8 presents flow projections if I/I rehabilitation is done. Based on a maximum daily WWTP flow of 13.0 MGD, this information indicates that additional I/I removal work will be required at least to 2050.

TABLE VI-8
FUTURE PEAK DAY FLOWS

	Year 2025				Year 2050			
Basin No.	Projected Growth (MGD)	Current Base (MGD)	I/I (MGD)	Total (MGD)	Projected Growth (MGD)	Current Base (MGD)	I/I (MGD)	Total (MGD)
1	0.03	0.12	2.24	2.39	0.13	0.12	2.27	2.52
2	0.00	0.43	1.70	2.13	0.00	0.43	1.70	2.13
3	0.00	0.03	0.80	0.84	0.00	0.03	0.81	0.84
4	0.02	0.11	2.55	2.67	0.04	0.11	2.55	2.70
5	0.15	0.04	0.19	0.38	0.44	0.04	0.27	0.75
6	0.03	0.09	0.26	0.38	0.15	0.09	0.30	0.54
7	0.16	0.03	0.30	0.49	0.31	0.03	0.34	0.69
8	0.34	0.14	0.66	1.14	0.75	0.14	0.79	1.68
9	0.00	0.03	0.00	0.03	0.00	0.03	0.00	0.03
10	0.02	0.12	2.37	2.51	0.03	0.12	2.38	2.53
11	0.03	0.00	0.01	0.04	0.06	0.00	0.02	0.08
12	0.01	0.00	0.00	0.01	0.01	0.00	0.00	0.02
Total	**0.79**	**1.15**	**11.09**	**13.02**	**1.93**	**1.15**	**11.43**	**14.51**

Note: Projected I/I includes 30 percent of additional base flow. Design flows use peaking factors determined during I/I monitoring study and peak both I/I and base to peak hour flow.

TABLE VI-9 FUTURE PEAK DAY FLOWS WITH I/I REHABILITATION								
	Year 2025				Year 2050			
Basin No.	Projected Growth (MGD)	Current Base (MGD)	I/I (MGD)	Total (MGD)	Projected Growth (MGD)	Current Base (MGD)	I/I (MGD)	Total (MGD)
1	0.03	0.12	1.34	1.50	0.13	0.12	1.37	1.62
2	0.00	0.43	1.70	2.13	0.00	0.43	1.70	2.13
3	0.00	0.03	0.40	0.43	0.00	0.03	0.40	0.44
4	0.02	0.11	1.28	1.40	0.04	0.11	1.28	1.43
5	0.15	0.04	0.19	0.38	0.44	0.04	0.27	0.75
6	0.03	0.09	0.26	0.38	0.15	0.09	0.30	0.54
7	0.16	0.03	0.30	0.49	0.31	0.03	0.34	0.69
8	0.34	0.14	0.66	1.14	0.75	0.14	0.79	1.68
9	0.00	0.03	0.00	0.03	0.00	0.03	0.00	0.03
10	0.02	0.12	1.19	1.33	0.03	0.12	1.19	1.34
11	0.03	0.00	0.01	0.04	0.06	0.00	0.02	0.08
12	0.01	0.00	0.00	0.01	0.01	0.00	0.00	0.02
Total	**0.79**	**1.15**	**7.33**	**9.27**	**1.93**	**1.15**	**7.68**	**10.75**

Notes: I/I for Basins 3, 4 and 10 assume removal of 50% through rehabilitation. gpd/in-mi for Basin 1 assumes PVC pipe has 15,000 gpd/in-mi. I/I for Basin 1 assumes 50% removal of non PVC I/I through replacement of non-PVC pipe. Projected I/I includes 30 percent of additional base flow. Design flows use peaking factors determined during I/I monitoring study and peak both I/I and base flow to peak hour flow.

FUTURE COLLECTION SYSTEM EXTENSIONS

The future service area identified in Section V will require extensions of the existing system to serve those areas. How these areas will enter the existing collection system is discussed below. Line sizing considers both the 2025 boundary and the ultimate boundary, since the useful life of the new pipes will extend well beyond the year 2025. The routing and location of these lines are estimated only and will be refined as the development in those areas occurs. At this time, we have assumed new lines will follow existing roads. Likewise, pipe diameter may be increased to meet minimum slopes required and/or reduce the need for pump stations. The extension identified below are also shown in the basin maps in Appendix D.

BASIN 1 has significant undeveloped land within the current service area of the basin in the upper reaches as well as growth beyond current boundaries to the east up Coal Creek. The increased service area to the east will add about 78 acres of residential land use designation. The main service route that will be needed to serve this are will be up Coal

Creek. This will most likely require an 8-inch line with a pump station near the entry to the existing system. The line may need to be larger to use a flatter slope but 8-inch will be sufficient for capacity concerns.

BASIN 2 is mostly developed, however not to full densities as allowed by the Land Use Plan. Expansion area for Basin 2 is beyond the 2025 growth boundaries and is up the hill along the northeast side. It is designated for low density residential land use. To serve this area, it is presumed short 8-inch main extensions will be made to existing lines.

BASIN 3 expansion can not occur since it is bordered on three sides by Basin 2 and the area to the northeast is Dobson Park, which is owned by the City. However there is undeveloped land that can be developed as well as redevelopment into higher density commercial use. None of this will require the construction of new collector main lines.

BASIN 4 expansion is out to the east, which is up on the hillside around the Valley View area. Land use in that area is designated low density residential. Development on the hillside will be slow due to steepness of the terrain. No major sewer main expansions will be necessary. As the area develops, short 8-inch main extensions will connect to existing sewer mains.

BASIN 5 has an expansion potential which is quite large and includes all of the area south of Maurin Road which is within City growth boundaries and the area west of I-5 that is south of Parkland Drive. This area is predominantly designated for industrial and commercial land use with the small pocket of residential on the south end of Bishop Road and the area East of Jackson Highway. Growth into these areas will require additional mains be constructed. Most will be simple 8-inch extensions, with the exception of service on the west side of the freeway. The west side will be served best by a new interceptor following along the freeway on the west side. Flows will only require this main to be 8-inches but due to the flat terrain, the diameter should be increased to 15-inches to eliminate the need for a pump station.

BASIN 6 growth will be a combination of infill at the Industrial Park and expansion up the hill to the east above the Kennicott area. Currently, the Industrial Park has several lots and acreage for industrial development. Commercial development is designated for the areas along Interstate 5 and along Jackson Highway. Land use to the east of Market Avenue is designated as low density residential. Expansion of the sewer system to serve this growth will require minor extensions along existing and new roads as the area develops. None of these lines will need to be greater than 8-inch in diameter to handle the anticipated flows. However, larger diameter mains may be required in the Industrial Park to avoid new pump stations and meet minimum slopes.

BASIN 7 has significant land around the basin that will most likely contribute sewage that will flow through this basin. This surrounding land is designated as Low Density Residential. Capacity of the existing 10-inch main is in excess of anticipated flows that will be generated in this basin within the next 50 years. Service to the areas north and east may be able to flow by gravity into the existing mains. Growth to the south will most likely require a pump station due the flatness of the area. Extensions will not require more than 8-inch diameter mains to handle flows but may be upsized to 10-inch to avoid construction of a pump station. Final determination of pipe size will be made once accurate survey data has been collected.

BASIN 8 is mostly developed but has space for infill. Growth for this basin will most likely be infill and stretching of the boundaries to the east. The designated land use to the east is divided between low density residential and commercial. Infill growth will not require additional sewer mains. Growth to the east however will require the extension of sewer mains and associated pump stations. Those mains can be 8-inch to handle the anticipated flows but may be upsized to reduce the need for pump stations. Pump station location and pipe size will be determined during the pre-design and design phases, when accurate survey data and development goals are established.

BASIN 9 is fully developed and is only expected to see minimal growth due to infill. No new sewer mains are required for this infill.

BASIN 10 growth will be due to infill and expansion to the northeast. The area to the northeast is steep hillside and is designated as residential land use. Extensions to serve this area will be 8-inch pipe. There are no roads in this expansion area, therefore the route of the sewer main extension will be determined as the land is developed.

BASIN 11 is bounded by the Chehalis River on the west and other basins on the east. New sewers to serve this infill will be required. Most should be 8-inch to meet capacity requirements but may be upsized during design to meet minimum slopes and reduce the need for additional lift stations.

BASIN 12 growth will be infill. The basin is not expected to see significant growth. Existing mains may be extended to serve the infill. Those mains will be 8-inch unless slope restrictions require larger pipe.

BASIN EVALUATION

The basin evaluation discussed below considers the collection systems ability to carry the current and anticipated peak day flows. The peak flows used in this report are based on the flow monitoring data collected in the winter of 1998 and allocated proportionally to obtain the peak day flow. These peak day flows are in excess of Criteria for Sewage Works Design requirement of 2.5 times average wet weather flow. Line sizing will consider 50-year growth projections since the life expectancy is more than 50 years for PVC sewer pipe. Current peak flows in the collection system were presented in Table VI-4. Future flows which considered no additional I/I rehabilitation work are presented in Table VI-8. A third flow projection, is presented in Table VI-9, which assumes I/I rehabilitation in Basins 1,3,4 and 10. Design flows presented in these tables are projected peak day flows allocated to the individual basins based on measured flows during the I/I monitoring period. The system is evaluated for current peak day flows and a maximum peak day flow of 14 MGD (projected 2050 peak day). As discussed below, many lines appear to surcharge during peak day flows. These lines are shown in Figure VI-15.

BASIN 1 peak day flows are estimated to be 2.35 MGD and are projected to increase to about 2.5 MGD by the year 2050. The 8-inch collector lines should be capable of carrying their portion of these flows. The 10-inch and 12-inch mains can handle in excess of 0.75 MGD and 1.08 MGD respectively. An evaluation of the basin characteristics shows the current line sizing and configuration can handle the peak day flows except for the last two 400 foot sections of 15-inch that have an estimated capacity of 1.62 MGD. These sections need to be upsized or replaced at a greater slope to carry 2.5 MGD. It appears this basin may be under surcharge conditions during current peak flow conditions.

Since this basin has extensive I/I, rehabilitation of the older non-PVC lines is recommended. This will require replacement of approximately 24,500 feet of 8-inch concrete sewer pipe, manholes and appurtenances. This rehabilitation will reduce anticipated peak day flows to 1.62 MGD, which can be handled by a 15-inch interceptor. Therefore, it is recommended to replace all the non-PVC pipe in this with PVC pipe the same size as existing pipe.

BASIN 2 peak day flows are estimated to be 2.13 MGD. The small growth area identified for the basin results in a negligible flow increase. Other flows into this basin come from Basins 3, 4 and 10. Collector lines have all been replaced in the City's I/I rehabilitation program. Evaluation of the basin interceptor under current peak day flow estimates, including influence from Basins 3, 4 and 10 shows the main interceptor from Green Hills to the basin outlet at Main Street may experience surcharge conditions. All other lines appear to be adequate to handle the current peak day flows. Projected future flows will not require additional pipe upsizing beyond what is needed to meet current projected peak day. However if I/I rehabilitation is continued as recommended, projected future flows will decrease in Basins 3, 4 and 10, and interceptor replacement will be limited to the 15-inch, 18-inch and 21-inch piping only. Each of these will only

INSERT FIGURE VI-15

require upsizing to 18-inch, 21-inch and 24-inch, respectively. One alternative is to limit flow from Basin 4 to 1.45 MGD. Basin 4 flow beyond the 1.45 MGD will then overflow into the Napavine/LCSD interceptor draining into Riverside Road Pump Station. Another alternative is to allow surcharging of the interceptor which will thereby increase line capacity.

It is recommended to maintain the existing collection system and interceptor and allow Basin 4 to surcharge. If flows exceed the increased capacity at surcharged conditions, the system can overflow into the Chehalis/Napavine/LCSD interceptor to avoid overflowing onto the ground. Therefore, flows will continue to surcharge during peak flows.

BASIN 3 peak day flows are estimated to be 0.82 MGD and are expected to grow to 0.83 MGD by the year 2050. Flow in the basin is basically two branches that combine one manhole upstream of the flow monitoring manhole. Therefore, the overall flow can be divided in two equal parts both of about 0.42 MGD. The existing 8-inch mains are adequate to handle these flows. Once the flow combines, the 8-inch main appears to be inadequate and may experience surcharge conditions. Replacement to alleviate surcharge conditions requires about 600 feet of new 10-inch main.

I/I monitoring identified this basin as having high I/I per inch-mile of pipe and is recommended for replacement. With the replacement, projected year 2050 peak day flows should be reduced to 0.31 MGD, which is less than the capacity of an 8-inch main at minimum slope. Therefore, it is recommended to replace the entire 9,800 feet of 8-inch concrete main with 8-inch PVC pipe.

BASIN 4 projected peak day flows are currently 2.64 MGD and are expected to grow to 2.69 MGD by the year 2050. This Basin is also influenced by flow from Basin 10, which flows into Basin 4 near the High School baseball field, and estimated peak day flow from Basin 10 is currently 2.51 MGD. In order to carry these estimated current peak day flows, the main north south interceptor would need to be upsized to a combination of 21-inch, 24-inch and 27-inch sewer mains. Currently the main lines in Basin 4 are undersized and surcharged during peak flows.

However, Basin 4 and 10 were identified as large contributors of I/I and are recommended for replacement. Once Basins 4 and 10 are rehabilitated, peak day flows should be reduced to 1.04 MGD and 0.99 MGD, respectively. With these projected flows, it appears the current interceptor sizing is adequate for peak day all the way down to the bypass, which drains into the Napavine/LCSD interceptor. At this point, the system will surcharge to carry the additional 0.4 MGD, with overflow into the Chehalis/Napavine/LCSD interceptor serving as a backup if surcharging cannot carry the peak flows. Therefore, the recommended improvement for Basin 4 is a complete replacement of non-PVC pipe with the same diameter pipe as existing.

BASIN 5 current peak day flows are estimated to be 0.19 MGD and are projected to increase to 0.76 MGD by the year 2050. This basin is around the southern portion of the Chehalis/Napavine/LCSD No. 1 interceptor and extends into the Port of Chehalis. Collection piping, other than the interceptor, is 12-inch PVC pipe that serves the Port of Chehalis. These 12-inch mains are adequate to handle the projected flows for the basin. Evaluation of the Chehalis/Napavine/LCSD No. 1 interceptor is presented later in this section. Therefore, there are not recommendations for Basin 5 other than extensions for growth, which were identified previously.

BASIN 6 peak day flows are estimated to be 0.34 MGD currently and are expected to increase to 0.54 MGD by the year 2050. Existing 8-inch sewer mains serving the northeastern end of the basin are adequate to serve both current and projected peak day flows. The main 12-inch and 15-inch interceptor serving the Industrial Park is capable of flows in excess of 1.0 MGD and the I/I monitoring showed the basin has less I/I than the rehabilitated Basin 2. Therefore, there are no recommended improvements for this basin.

BASIN 7 estimated current peak day flow is 0.28 MGD and is projected to increase to 0.69 MGD by the year 2050. The existing 10-inch sewer main at minimum slope is adequate to handle this projected peak day flow. Also it appears there are no 8-inch mains that will need upsizing to carrying the project peak day flow. Therefore, there are no recommended improvements for the Basin 7 collection piping.

BASIN 8 estimated current peak day flow is 0.71 MGD and is projected to increase to 1.68 MGD by the year 2050. Interceptor lines in this basin are all 10-inch that can carry 0.75 MGD at minimum slope, and are adequate to handle the current peak day flows. However, as growth occurs in the basin, these lines will need to be upsized to a combination of 12-inch and 15-inch lines. The 12-inch lines will replace the 10-inch lines leading into the West Washington Pump Station and the Napavine Pump Station, and the 15-inch main will be required from the northern end of the Napavine Pump Station force main to the basin outlet. During design, the 15-inch portion may be reduced to only 12-inches if the slope can be increased to 0.55 feet per 100 feet, which is greater than the minimum of 0.22 feet per 100 feet for a 12-inch pipe.

BASIN 9 peak day flow was estimated to be 0.03 MGD currently. I/I flows were minimal and assumed to be zero. There is no growth anticipated for this basin. The capacity of existing 8-inch sewer mains is 0.5 MGD at minimum slope. Therefore, further evaluation of this basin is not required because it can handle future flows and it has already been rehabilitated with PVC to reduce I/I.

BASIN 10 current estimated peak day flow is 2.51 MGD and is projected to increase to 2.54 MGD by the year 2050. The main interceptor in this basin is a 15-inch line from the Middle School to the High School baseball field. The capacity of this line is about 1.62 MGD. Other collector mains are 8-inch and are configured to serve a peak day flow of less than 0.5 MGD, which is the capacity of an 8-inch main. If the basin is rehabilitated to remove I/I, projected peak day flow for the year 2050 will be reduced to about 1.0 MGD. This projected peak is less than the capacity of the 15-inch interceptor. Therefore, the recommendation for this basin is to replace all non-PVC pipe. New pipe shall be the same diameter as existing. The main interceptor will continue to surcharge during peak flow conditions until I/I work is complete in this basin.

BASIN 11 was not monitored during the I/I monitoring period and therefore no flow has been allocated to this basin. Collector lines are all 8-inch in diameter, which can handle a minimum of 0.5 MGD. The interceptor in this basin is the Chehalis/Napavine/LCSD No. 1 interceptor, which is evaluated later in this section. I/I rehabilitation for the collector lines is not warranted at this time. Once the City completes replacement of non-PVC pipe in the priority basins, the non-PVC collector laterals in Basin 11 may be considered for replacement. However at this time, there are no recommended improvements for Basin 11.

BASIN 12 flows are minimal and have been assumed to be negligible during the flow monitoring period. Flows in this basin are less than 0.5 MGD, which is the minimum capacity of collector mains in this basin. Most of the basin consists of PVC mains. Non-PVC mains could be replaced but are not a priority. Therefore, no improvements are recommended for Basin 12, at this time.

CHEHALIS/NAPAVINE/LCSD NO. 1 INTERCEPTOR

This interceptor was constructed in 1978 along with Napavine, Rush Road and Riverside Pump Stations. It was designed to serve the southern portion of the City of Chehalis, Napavine and Lewis County Sewer District and is now nearing capacity. A model of this interceptor was developed using record drawings. This model shows the interceptor has a capacity of 1.08 MGD, assuming no surcharging. The estimated current peak day flows from the City of Napavine and LCSD No. 1 are 0.76 MGD and 0.31 MGD and account for all of the current 12-inch capacity. Using the projected growth in the areas served by this interceptor, additional capacity is needed.

Alternatives considered are replacement with a new larger diameter pipe, a parallel pipe alongside the existing, or a parallel pipe in a new alignment along the western side of Interstate 5. A pipeline schematic showing required pipe sizing for a new pipe and for a parallel pipe is provided in Figure VI-16. The advantages and disadvantages of the three options are discussed below.

The replacement pipe will not require additional right-of-way. It will replace older non-PVC pipe and reduce potential I/I along the interceptor route. Maintenance will be for only one pipe. Since there will only be one pipe, reliability will be minimal. Any future maintenance of the line will require bypass pumping. This is the most costly option because it will require larger pipe and bypass pumping during construction.

Construction of a parallel line alongside the existing interceptor will provide redundant piping that will allow one line to be taken out of service for maintenance. During low flow periods, flow can be routed through one line to maintain adequate velocities. Although the City has some existing right-of-way, additional right-of-way will probably be required.

Construction of a new parallel line on the western side of the freeway will provide service to properties on the western side of the freeway and can be designed to accept wastewater flow from Napavine. Construction along Hamilton Road will be simpler and more direct. Easements will be required on Hamilton Road. Private development along the route could help reduce the cost of this option, if adjacent landowners are willing to share in the construction cost.

The recommended interceptor upgrade is to construct a new 15-inch line along the west side of the freeway because of the potential cost sharing with new development along the route. This line will begin at the convergence of the LCSD No. 1 and Napavine interceptors and go west to the west side of the freeway. From that point, the line will be routed along the western edge of the freeway right-of-way all the way up to the 13th Street overpass. At the overpass, the line will tie into the existing 21-inch main. This will cause the 21-inch main to surcharge during projected year 2025 peak day flow conditions but should be capable of carrying the

INSERT FIGURE VI-16 INTERCEPTOR UPGRADE OPTIONS projected 3.77 MGD peak day flow without overflows. Alternate routes along the west side of the freeway can be considered if potential developments participate in the construction of the new parallel interceptor. The cost of the 19,000 foot of 15-inch parallel line is estimated at $180 per foot for a total of $3,420,000 and includes $120 per foot construction cost, 10 percent construction contingency, and 35 percent engineering, legal and administration cost. Right-of-way and wetland mitigation are not anticipated at this time and have not been included in this cost estimate.

Since preparing the draft of this report in 1998, Tractebel has decided to construct a power plant in the Chehalis industrial park. Due to the large sewage volumes that they will produce, a large portion of interceptor will be replaced as part of their project. Tractebel will replace the existing interceptor from 13th Street to just past LeBrie Road. The replacement line will be 24-inch diameter in the same alignment as the existing pipe. Tractebel will also construct a stub-out under I-5 so that the west side of the highway can be served. The stub out is in the vicinity of Hamilton Road. A new line up Bishop Road to the power plant site is also proposed. This project will have major benefits for the following reasons:

- It relieves a bottle neck that has been a problem for years.
- It replaces the existing line which may contribute I/I.
- It constructs a stub-out to serve the west side of the highway.

PUMP STATION DESCRIPTIONS AND EVALUATIONS

The City of Chehalis collection system includes ten sewage pump stations. The City of Napavine owns and operates five sewage pump stations and LCSD No. 1 owns one pump station, which is maintained by the City of Chehalis. All these stations are shown in Figure VI-17. The following evaluations of these stations considers existing pump station condition, compliance with Criteria for Sewage Works Design by DOE, recommendations to improve operation and capacity evaluation under existing and projected flows.

CONTROLS

All of Chehalis and LCSD No. 1 pump stations are equipped with the same telemetry system which sends a signal to a Supervisory Control and Data Acquisition (SCADA) system located at the WWTP. The Napavine pump stations do not have telemetry. All of the larger pump stations have controls that include level transducers, which operate the pump start and stop functions, and two mercury tilt float switches, which are used to override the level transducer for low water level and high water level. The low water level float switch shuts down all the pumps and signals the alarm and the high level signals the alarm. Controls are all located in stainless steel enclosures that house the motor starters, hour meters, circuit breakers, telemetry equipment and hand-off-auto (H-O-A) switches for each of the pumps. The stations are equipped with visual alarms, and instead of audible alarms, alarms are telemetered back to the WWTP. Riverside, Prindle, Napavine Fire Station and Rush Road Pump Stations have auxiliary generators on-site with automatic transfer switches. All other stations are equipped with auxiliary generator connections along with associated manual transfer switches for use with a portable generator. The City

Has recently purchased a 80kW, 460 volt, 3-phase diesel generator that is trailer mounted. This size of generator will be adequate for two 15 Hp motors.

- The South Kresky pump station serves about 20 acres of commercial and light industrial land. It is a 4-foot diameter manhole equipped with two submersible pumps. Current pump operation is about 4 hours per day average. Manhole access is good. The 6-inch diameter force main is 950 feet long.

  The two recently installed identical pumps are rated at 80 gpm. The wet well volume between pump start/stop levels is approximately 190 gallons. Existing flows are estimated to be about 8,000 gpd with a peak day of about 57,600 gpd (40 gpm). Projected growth in the area served by this pump station limited but assuming the same growth rate as the overall basin, the projected peak day flow into the pump station will be 44 gpm by the year 2025 and 48 gpm by the year 2050. Therefore, the existing pumps are adequate.

  INSERT FIGURE VI-17

- The North Kresky pump station is located near the northern border of Chehalis. The station serves the Grocery Outlet and an unoccupied meat cutting shop. It is also a 4-foot diameter manhole equipped with one submersible pump. The pump is not equipped with rails and it is not recommended to upgrade the pump with rails due to the pump station size. Criteria for Sewage Works Design recommends a second pump. However, wet well storage volume and low influent flow conditions do not warrant a redundant pump. Since it serves only two businesses, Chehalis can work with those businesses to limit flows into the pump station and use a portable trash pump when necessary. The station is equipped with a generator connection receptacle so that it can be run with a portable generator in an emergency. The 3-inch diameter force main is 325 feet long and does not need to be replaced.

The pump is a ½ Hp Barnes Model SE51, with a capacity of 35 gpm. The wet well operating volume is 125 gallons. Based upon hour meter readings, the pump operates about 0.2 hours per day on average and does not even operate some days. No increase in flow rate is anticipated for this pump station. Therefore, no improvements are recommended.

- The North National pump station serves the Chevrolet auto dealership and neighboring County Senior Center on the north end of Chehalis. It is a 4-foot diameter manhole with a single submersible pump, which is not equipped with rails. Criteria for Sewage Works Design recommends a second pump for redundancy. However, wet well storage volume and low influent flow conditions do not warrant a redundant pump. The 3-inch diameter force main is 950 feet long and is not in need of replacement.

  The pump is a 1½ Hp Barnes SE153 with a capacity of 85 gpm. The wet well operating volume is 125 gallons. Based upon hour meter readings, the pump operates about 0.4 hours per day on average. The service area of this pump station is already developed; therefore no increase in flow rate is anticipated. Once this pump fails and it needs to be replaced, Chehalis should consider installing two pumps with rails. The new pumps should be sized closer to actual flow conditions to increase run times and provide redundant pumping ability.

- The South National pump station serves approximately 325 acres of commercial/industrial area. It is a wet well/dry well station with two centrifugal dry pit pumps with submersible motors. The wet well is an older 6-foot diameter manhole with eccentric cones and a standard manhole lid. The dry pit is a prefabricated steel structure extending about 4-feet above grade. It has a small blower for air exchange that is always operating. The dry pit is also equipped with a sump pump and float switch and their associated controls. Check valves are equipped with flow indicators to ensure sewage flow upon pump startup. The potable water hookup is installed with a reduced pressure backflow preventor. The 6-inch diameter force main is 4,200 feet long and does not to be replaced to handle projected flows.

  There are two pumps that are 15 Hp Fairbanks Morse Model number 5430. The pumps are rated at 225 gpm. Wet well capacity, between the pump on and off levels, is 275 off level, if needed. On average, these pumps operate about 4 hours per day. The gallons. This volume can be increased up to about 1,000 gallons by raising the pump shut estimated current peak day flow is 410 gpm and projected future peak flow rate for the year 2025 is 420 gpm.

This station is capable of meeting peak day demands but is not equipped with redundant pumping during peak day demands. This station should be upgraded to include a third pump to meet reliability requirements. However to install the third pump, the station will need to be totally rebuilt. Therefore, this station will be on the list for replacement but is a lower priority and will be done following replacement and upgrade of higher priority pump stations.

- The State Street pump station serves a residential and industrial area in the north end of the City covering 68 acres. The station is wet well/dry well with two split case pumps. The wet well is a 4-foot diameter manhole with a standard manhole lid located in the street. The dry pit is approximately 10 feet by 10 feet. The dry pit has a metal walkway above the floor and has adequate lighting. It is in need of a blower system for dry pit ventilation. The 6-inch diameter force main is 350 feet long and not in need of replacement.

  The two split case pumps are 5 Hp Fairbanks Morse pumps Model D5432ND210. Each pump is capable of 300 gpm at 60 feet TDH. The estimated peak day flow is currently 290 gpm and is not expected to increase. Therefore, this pump station is capable of meeting peak day flows and is already equipped with a redundant pump. Dry pit ventilation can be installed permanently or a portable unit can be used when personnel enter the dry pit. Therefore no improvements are recommended for this pump station.

- The Wal-Mart pump station was constructed in 1995 to serve the commercial area west of I-5 Exit No. 79. The 6-foot diameter wet well is equipped with two submersible pumps with stainless steel rails. The check valves, located in a nearby valve vault, equipped with "no-flow" alarms that shut down the pumps and signal an alarm if flow is not produced after the pumps are signaled to start. Access lids are Bilco hatches in very good condition and provide good access. The associated force main is a 6-inch D.I. pipe, about 1,500 feet in length.

The two pumps are 7½ Hp Fairbanks Morse submersible pumps Model D5430MT, with a rated capacity of 175 gpm each. Currently the pumps operate 0.2 hours each day. Projected flows are limited and not expected to increase significantly. Therefore, this pump station will not require any upgrades.

- The Airport pump station was constructed in 1980 to serve the airport and surrounding area for a total area of 200 acres. The 6-foot diameter wet well is equipped with two submersible pumps with stainless steel rails. The check valves, located in a nearby valve vault, equipped with "no-flow" alarms that shut down the pumps and signal an alarm if flow is not produced after the pumps are signaled to start. Access lids are Bilco hatches in very good condition and provide good access. The associated force main is a 6-inch D.I. pipe, about 2,800 feet in length.

  The two pumps are 3 Hp Flygt Model C-3085, submersible pumps with a rated capacity of 160 gpm each. Current peak day flows are about 170,000 gpd and require the pumps to operate only 1 hour each day. Flows are not expected to increase at this pump station and therefore no upgrades or improvements are necessary at this pump station.

- Front Street pump station, located next to the train station, serves a small commercial area to the south of the pump station. It is a small 4-foot diameter wet well with two submersible pumps. The pumps are not on rails and check valves are on vertical pipes. There are no gate valves to isolate check valves for maintenance. Access is limited since the pump station located on a sidewalk with overhead limitations. The force main is a short 50 foot long 3-inch diameter pipe.

  The two submersible pumps are 1 ½ Hp Barnes Model 3SE1594L with a rated capacity of 85 gpm each. Current peak day flows are estimated to be 30,000 gpd (20 gpm) and require the pumps to operate about 6 hours per day on a peak day. On average the pumps operate about 20 minutes each day. Flows are not expected to increase at this pump station and therefore the station does not require any pumping upgrades.

- Riverside pump station serves all areas south of City of Chehalis city limits, including Napavine, LCSD No. 1, and the Port of Chehalis, and the area east of I-5 and south of Highway 6. The total area served is about 1,500 acres and includes industrial, commercial and residential customers. The station is a wet well / dry well setup that was constructed in 1980 along with the interceptor serving Napavine and LCSD No. 1. It was re-equipped with two 45 Hp pumps rated at 1,600 gpm each in 1987. The wet well is about 20 feet deep and has a capacity of 9,500 gallons. Both the wet well and the dry well are vented with separate vents. The dry well has recently been equipped with a 200 GPM flood pump that discharges back into the wet well. A 135 kW generator is located in a separate room and connected with automatic transfer switching. The force main is a combination of 10-inch and 18-inch diameter lines and runs 3,400 feet to the treatment plant headworks. The 10-inch line is about 2,500 feet long and combines with the 18-inch Prindle Street pump station force main about 900 feet prior to the WWTP.

  The revised capacity for this station after the 1987 upgrade is 1,600 gpm at 80 feet total dynamic head (TDH). Peak day flow into the Riverside pump station is estimated to be about 1.52 MGD (1,055 gpm) currently and expected to increase to 3.58 MGD (2,490 gpm) by the year 2050. The average run time for the pumps is about 13 hours per day, which is between 1.3 MGD and 2.5 MGD average day flow. This high flow is believed to be I/I in the gravity main in the low lying field east of I-5. Various reviews of this section of main have found problems with the manholes that have subsequently been repaired. However, a more thorough review through TV inspection and possible replacement should be considered. During peak storm events, both pumps run continuously. Based upon the estimated flows seen during flow monitoring, flows into this pump station could be reduced to 1.52 MGD on a peak day and will be about closer to 0.7 MGD on an average day. If the I/I is reduced, it appears the pump station could meet current peak day flows with one pump. However, when Riverside operates at the same time as Prindle pump station, head loss in the combined section of 18-inch increases and causes the Riverside pumps to pump operating, the Riverside pump station is producing about 1,700 gpm at about 104 feet of less flow. During peak flow conditions with both Riverside and all three Prindle pumps TDH. Therefore, this station does not have redundant pumping capacity during peak day flows.

To meet current conditions, this station will need to be reconstructed to add one identical pump or replace both pumps with two new pumps that can produce the 1,600 gpm at the higher head. This will provide sufficient pumping capacity to meet the projected year 2025 peak day capacity of 2.38 MGD (1,652 gpm). Other improvements recommended at this station are to install a flow meter, modify the telemetry system to allow control of this pump station from the WWTP and install a blower system in the dry pit that is activated with the lights. The pump station force main has a Doppler type flow meter on it that never worked due to electrical interference with the VFDs. The meter has been relocated to measure plant inflow. A new flow meter should be installed at this pump station away from the magnetic interference of the VFDs. The pump station controls need to be modified to allow pump control from the WWTP so that inflow to the plant can be regulated during peak flow events. The blower system shall ensure sufficient air exchanges to meet confined space requirements.

- Prindle Street pump station is the main pump station in the City of Chehalis collection system. It receives flows from the entire collection system with the exception of that portion which is pumped by the Riverside pump station and a small portion of gravity flow near the WWTP. This pump station was constructed in 1948 and has been upgraded several times to handle additional flows. It is a wet well/dry well set-up similar to the Riverside, Rush and Fire Station pump stations. Controls have been updated and are similar to all the other City of Chehalis controls with a pressure transducer, floats and telemetry back to the WWTP. It is equipped with three 50 Hp Aurora pumps rated at 2,500 gpm each. The wet well is about 23 feet deep and has approximately 16,000 gallons of operating capacity. The dry well has recently been equipped with a 200 GPM flood pump that discharges back to the wet well. Both wet well and dry well are vented with separate vents. A 150 kW generator is located in a separate room and connected with automatic transfer switching. There are two force mains leaving the Prindle Street pump station. One force main is a 14-inch line that was sliplined with a new 10-inch HDPE pipe approximately 1,800 feet long. The other is an 18-inch ductile iron force main about 2,000 feet long that follows a slightly different route from the pump station to the WWTP. The Riverside pump station force main combines with the 18-

inch Prindle Street pump station force main about 900 feet from the WWTP. When both pump stations are operating, this section of pipe experiences significant headloss.

The three pumps have a capacity of 3,700 gpm at 37-feet TDH, when operating individually. The combined flow rate with all three operating is 7,550 gpm at 62-feet TDH. The existing peak day flows are estimated at 10.5 MGD (7,300 gpd) and projected to decrease to 5.69 MGD (3,950 gpm) by the year 2025, if the I/I rehabilitation of Basins 1,3,4 and 10 is completed. The pumps operate about 8 hours per day on an average day, which is a maximum of 5.3 MGD. Variable speed control was added in the 1980 upgrade. The current pumping capacity of 7,550 gpm is adequate for the estimated peak day flows but does not provide a redundant pump at this rate. During peak storm events, all three pumps run continuously.

Since I/I rehabilitation is planned for a 40 year program, it is recommended this station be re-equipped with three new pumps that will meet the required flow rate of 7,550 gpm with one pump out of service. Pump replacement will require new inlet and outlet piping, as well as, upgrading the electrical system. In addition, the existing flow meter needs to be fixed so that the VFDs do not cause electrical interference with the flow meter, the telemetry needs to be upgraded to allow pump control to be from the SCADA system at the WWTP and a blower system should be installed to ensure sufficient air exchanges in the dry pit when personnel are present.

- Logan Hill pump station is owned by LCSD No. 1 and maintained by the City of Chehalis. The station serves the northern half of the LCSD No. 1 service area of about 56 acres. The station is a 6-foot diameter wet well with two submersible pumps that are equipped with rails. Check valves and gate valves are located in a separate vault. The wet well and valve vaults have appropriate Bilco hatches for access and security. This station was constructed in 1995 and includes an on-site emergency generator with manual transfer switch. The site has water service with appropriate backflow prevention of reduced pressure double check assembly and is fenced to discourage vandalism. The force main is 6 inches in diameter and 1,250 feet long.

The two pumps are 5 Hp Fairbanks Morse 5432ND210 submersible pumps with a rated capacity of 100 gpm each. Current peak day flows are estimated to be about 20,000 gpd and are not projected to increase. On an average day, the pumps operate 1 hour each for a total of 12,000 gpd. This station is up-to-date and in very good condition. It does not appear that this station will need any upgrades within the next 25 years.

- Napavine pump station number 1 is located at next to the Fire Hall at Jefferson and 2nd Avenue East. This pump station was constructed in 1978 and serves the majority of the City of Napavine. The station is inside a 30 x 20 foot masonry building and the site is fenced. It is a wet well/dry well setup inside a 16-foot inside diameter concrete caisson. The two pumps are 10 Hp Worthington SK25601375 with a 4-inch discharge. The discharge lines are equipped with check and gate valves, located on the vertical portion of the discharge piping, and join into a common 6-inch discharge pipe inside the dry pit.

The 25-foot deep dry pit has duplex sump pumps with float switches. The wet well and dry well have separate vent pipes. Pump controls use bubbler tube readings. The 6-inch ductile iron force main is approximately 1,000 feet long. The City has installed a quick connect on the force main for connecting the portable trash pump when the pump station is out of service. Backup power is supplied by the 50 kW diesel generator located in a separate room inside the building. The generator is connected with an automatic transfer switch.

The two pumps have a capacity of 250 gpm each. The existing peak day flows are estimated to be 0.32 MGD (225 gpm). Projected growth within the basin could raise peak day flows up to 0.39 MGD (270 gpm) by the year 2025 and up to 0.57 MGD (400 gpm) by the year 2050. Based upon these projected flows it appears that the pumps are capable of meeting peak day flows with one pump for approximately the next ten years. To meet the peak day flows beyond the next ten years, one new pump will need to be installed. The City of Napavine currently has two spare pumps that are suitable for this use.

Recommended improvements for this pump station are the replacement of the bubbler tube level controls with a pressure transducer level control, installation of a flow meter on the force main, a permanent blower system for the dry pit and installation of a third pump to meet future anticipated flows. The bubbler tube system has consistently created problems with operating pump No. 2, which causes the City of Napavine to rely on pump No. 1. This creates uneven wear between the two pumps. The City of Napavine is planning to upgrade level controls with a pressure transducer and float switch overrides in the near future. Flow metering is essential to obtain accurate data for system management. The blower system should ensure sufficient air exchanges and should be activated with the dry pit lights.

- Napavine pump station No. 2 is also referenced as the Rush Road pump station. This station was constructed at the same time as the Napavine and Riverside pump stations and also has the same design. The pumps have been upsized along with the discharge piping in the dry well. The station serves all of the City of Napavine. The 6-inch force main is about 6,300-feet long.

  Napavine upgraded the pump station in 1996 and installed two new pumps, which are 10 Hp Allis Chambers with an assumed capacity of 400 gpm each. The existing peak day flows are estimated to be 1.05 MGD (700 gpm) and projected to increase to 1.6 MGD (1,111 gpm) by the year 2025. Average run times of these pumps are about 5 hours per day or about 0.24 MGD. However according to the operator, both pumps operate continuously during peak flows. Without pump curves it is unknown what flow can be produced with both pumps. Assuming the pumps are 400 gpm pumps, Total Dynamic Head (TDH) would be 20-feet of static and about 120-feet of dynamic head. Assuming 80% efficiency, the horsepower required would be 17.7 Hp. Therefore, it is assumed that the 400 gpm rated capacity is with both pumps operating. To pump the current peak day of 700 gpm, headloss in the 6-inch force main would be in excess of 315-feet. This is well beyond realistic capabilities of most sewage pumps and it is recommended that a parallel force main be installed. The parallel force main shall be sized to meet the year 2025 peak day demands of 1,111 gpm and remain below about 100-feet of headloss using both force mains. This will require a parallel 10-inch force main.

Therefore, it is recommended this station also be upgraded to pump peak day flows with one pump out of service. It is recommended to replace the existing two pumps with three new pumps that would be capable of 1,111 gpm with one pump out of service. This will require new inlet and outlet piping headers and an upgrade to the electrical system. Additionally, a new 10-inch parallel force main will be constructed to operate in unison with the existing 6-inch force main. This pump station does not have any operational problems to be noted. However, the City is planning to replace the bubbler tube system with a transducer and float setup also proposed for the Napavine pump station No. 1. A flow meter should also be installed on the force main to accurately monitor the station's flow. The meter should be read on a daily basis as part of the daily pump station visits. A permanent blower system should be installed in the dry pit and activated with the dry pit lights.

- Pump Station No. 3 is located at 2nd Avenue and Washington Street. This station serves about 200 acres of residential area. It is a 8 foot diameter manhole equipped with 2 submersible pumps on rails. The pumps are controlled by float switches in the wet well. Electrical equipment is located on a pole and covered with a small shed roof. Electrical panels are showing signs of age. A visual alarm is provided, but no audible alarm or telemetry is provided. Check valves and gate valves are located in a valve vault. Access into both the wet well and valve vault is good. Site security is good with fencing and locking access hatches. The 4-inch ductile iron force main is 850-feet long.

  The two pumps were identified as 3.7 Hp Barnes Model 4SE3734L. Average pump run time is about 10 hours per day during winter conditions. The existing peak day flows are estimated at 0.24 MGD (167 gpm) and projected to increase to 0.27 MGD (190 gpm) by the year 2025. Based upon average winter pump run times, the pump station can not meet peak day flows with only one pump and therefore does not meet the reliability requirements. The pumps should be upsized to meet the projected year 2025 peak day flows of 190 gpm each.

- Pump Station No. 4 is located at 3rd Avenue and Grand Street. This station serves about 300 acres of residential area. This pump station is identical to Pump Station No. 3. It is a 8-foot diameter manhole equipped with 2 submersible pumps on rails. Pumps are controlled by float switches in the wet well. Electrical equipment is located on a pole and covered with a small shed roof. Electrical panels are showing signs of age. A visual alarm is provided, but no audible alarm or telemetry is provided. Check valves and gate valves are located in a valve vault. Access into both the wet well and valve vault is good. Site security is good with site fencing and latching access hatches. The force main is a 4-inch ductile iron line about 700-feet long.

  The two pumps were identified as 3.7 Hp Barnes Model 4SE3724L. Assuming an elevation difference of about 20-feet and 55 percent efficiency, these pumps are capable of about 180 gpm each. Average pump run time is about 4 hours per day during winter conditions, which is 43,200 gpd. The existing peak day flows are estimated at 50,000 gpd (35 gpm) and projected to increase to 60,000 gpd (42 gpm) by the year 2025. Based upon these estimates and current run times, it appears these pumps are adequate to meet projected peak day flows beyond the year 2025. The next time the pumps are removed or serviced, additional data on the pump motor and impeller should be noted so that pump curves can be obtained to verify these assumptions.

- Pump Station No. 5 is the newest pump station and is located in Napa Estates on Cheri Court. It was constructed in 1993 as part of the Napa Estates development and serves only a portion of the Napa Estates development, which consists of about 10 homes. It is a 4-foot diameter wet well with two submersible pumps on rails. The force main is a 4-inch line, about 200-feet long. The control panel is similar to the new controls on all the other Chehalis pump stations with Consolidated Electric control panel using a level transducer and emergency float switch system. Alarms signal a red alarm light located on the control panel. The station is not equipped with an audible alarm or portable generator hookup. However, the City of Napavine does have a portable trash pump for use in this station, as well as, the other lift stations, which reduces the need for a portable generator hookup.

The two pumps are 3 Hp Peabody Barnes Model SVG301 with a capacity of 80 gpm each. Average pump run time is about 1 hour per day for about 5,000 gpd. The existing peak day flows are estimated at 10,000 gpd and are not expected to increase. Both pumps are capable of meeting peak day flows well beyond the year 2025. Therefore, there are no recommended upgrades for this pump station.

RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

The review of the collection system shows areas of the system that are undersized to carry peak day flows by gravity to the Riverside and Prindle pump stations and causes the system to surcharge. As the system surcharges, the collection system capacity increases and has been shown to carry the peak day flows to the pump stations without overflows of raw sewage. Ideally the system should operate by gravity during all flow conditions, but that will require replacement of more than 25,000-feet of interceptors and could create higher flows at the treatment plant. Therefore, the recommended collection system improvement plan is to continue to allow surcharging of lines during peak day flows and continue the I/I reduction program to reduce these peak day flows. The I/I program will begin following the completion of the new WWTP and will continue on a basin by basin approach. The program is set for a 40-year program at about $400,000 per year. Prioritization of the basin rehabilitations will be based upon I/I measured in gallons per day per inch-mile of pipe and adjusted as problems are noted. The CIP presented in Table VI-10 prioritized I/I rehabilitation based upon the cost per gallon of I/I removed as presented in Table VI-7. In the mean time, the City of Chehalis will need to monitor peak flow conditions to ensure no overflows occur. As high surcharging occurs, spot improvements will need to be made to avoid overflows. In addition, major pump stations (Riverside and Prindle) will need to upsized to provide peak day flows with one pump out of service. The specific improvements to the collection system piping and pump stations are discussed below.

Basin 1 is recommended for I/I rehabilitation of all non-PVC pipe, which is about half the basin. There is approximately 24,250 feet of 8-inch and about 1,000 feet of 24-inch sewer main to be replaced. The estimated cost for the replacement is $150 per foot for a total of $3,637,500. This cost includes $100 per foot construction cost, 10 percent contingency and 35 percent engineering, legal and administrative costs.

Basin 3 was identified as having high I/I per inch mile of pipe and is recommended for complete replacement. The entire basin consists of 9,800-feet of 8-inch main. With the rehabilitation, 8-inch mains are adequate. Therefore, replacement shall be 9,800-feet of 8-inch mains. The estimated cost is $150 per foot for a total of $1,470,000.

Basin 4 is also recommended for I/I replacement. There is about 28,500 feet of main line sewers in Basin 4 with 3,000 feet of this being 10-inch and another 3,000 feet being 15-inch. During replacement, the last 1,500 feet of 15-inch should be considered for upsizing to 18-inch. All other mains will be the same size as they are now. The estimated cost, using $150 per foot, is $4,275,000.

The last Basin recommended for I/I replacement is Basin 10. Basin 10 consists of 27,600 feet of mainline sewers, with about 1,500 feet being 15-inch diameter mains. Replacement shall be the same size as current sizing. The estimated cost, using $150 per foot, is $4,140,000.

TABLE VI-10
CITY OF CHEHALIS
COLLECTION SYSTEM CAPITAL IMPROVEMENT PROGRAM

ITEM	COST (1998 DOLLARS)	IMPLEMENTATION YEAR (S)
Basin 1 (24,250 feet @ 150/ft) 2,500 feet per year for 10 years	$ 3,637,500	2007-2016
Basin 4 (28,500 feet @ $150/ft) 2,500 feet per year for 12 years	$ 4,275,000	2017-2028
Basin 10 (27,600 feet @ $150/ft) 2,500 feet per year for 11 years	$ 4,140,000	2029-2039

Basin 3 (9,800 feet @ $ 150/ft) 2,500 feet per year for 4 years	$ 1,470,000	2040-2043
Chehalis/Napavine/LCSD No. 1 Interceptor (19,000 feet @ $180/ft)	$ 3,420,000	2002
North National Pump Station	$ 25,000	2004
South National Pump Station	$ 96,000	2003
Front Street Pump Station	$ 14,000	2004
Riverside Pump Station	$ 660,000	2002
Prindle Pump Station	$ 1,620,000	2001
Napavine P.S. No. 1 (Fire Station)	$ 265,000	2020
Napavine P.S. No. 2 (Rush Road)	$ 1,935,000	2001
Napavine P.S. No. 3 (2nd Ave.)	$ 44,000	2005

Based on the above I/I removal project implementation schedule, the City should be able to meet the 85% TSS wet weather reduction limit in the year 2025. By that time, over 45,000 feet of faulty sewer pipe is scheduled to be replaced with new PVC pipe. This represents approximately 50% of the pipe scheduled for replacement in the entire City and rehabilitation of the two top priority basins. The goal of the I/I removal program is to be able to meet the 85% TSS wet weather reduction limit. Once this is achieved, the City will re-evaluate the program to determine if additional projects are required.

The Chehalis/Napavine/LCSD No. 1 interceptor is very near capacity and it is recommended to construct a new parallel 15-inch line along the west side of the freeway. This new interceptor line would begin where the LCSD No. 1 and Napavine interceptors connect and go west to the west side of the freeway. From that point, the line would be routed along the western edge of the freeway right of way all the way up to the 13th Street overpass. At the overpass, the line could tie into the existing 21-inch main. This would cause the 21-inch main to surcharge during peak flow conditions, but it should be capable of carrying the projected 3.77 MGD peak day flow. Alternate routes further west of the freeway could be considered if potential developments participate in the construction of the new parallel interceptor. The cost of the 19,000 foot parallel line would be about

$180 per foot for a total of $3,420,000 and includes $120 per foot construction cost, 10 percent construction contingency, and 35 percent engineering, legal and administration cost. Right-of-way and wetland mitigation are not anticipated at this time and have not been included in this cost estimate.

North National pump station upgrade will include two new submersible pumps with rails. The pumps will be smaller than the existing pumps to increase run times. The installation of a stainless steel rail system will also require the check and plug valves to be installed outside the wet well in a valve vault for better access. The total estimated cost for this upgrade is about $25,000. It is not a high priority and should be the last pump station upgraded.

The South National pump station is recommended to be upgraded to a submersible station and eliminate the confined space. The new station will require a new 6-foot diameter manhole, new pumps with rails and reuse of existing level transducers and controls. The new pumps will be rated at 400 gpm each. The estimated cost for this upgrade is about $55,000 for construction plus 30 percent construction contingency and 35 percent engineering, administration and legal for a total cost of $96,000. This is a high priority and should be done after Riverside and Prindle pump stations are upgraded.

Front Street pump station will be upgraded with plug flow valves to allow operation of the station while servicing the check valves. This will require new valves, piping connections and valve vault. The estimated construction cost is about $8,000 plus 30 percent construction contingency and 35 percent legal, engineering and administration for a total cost of about $14,000. The priority for this upgrade is low. It is not a health concern and is primarily for maintenance concerns.

Riverside pump station upgrade will be to replace both existing pumps with three new larger pumps capable of 1,800 gpm with two pumps which is the anticipated year 2025 peak day flow into the pump station. These pumps will need to be 40 Hp pumps rated at

900 gpm at 113 feet TDH. It is recommended the pump motors be the dry pit submersible type and include Variable Frequency Drives (VFDs) for low flow conditions. The wall between the wet well and dry well will need to be core drilled for an additional 8-inch suction line for the third pump. Discharge piping will need to be reconstructed to allow for three pumps and shall include a flow meter. Electrical upgrade may require a new service drop and a new panel board. Telemetry and pump controls to allow the pumps to be operated from the WWTP. With the larger pumps, the on-site generator will need to be upsized to handle three 40 Hp pumps. During construction, bypass pumping will be required to allow for core drilling and replacing piping and pumps. Estimated costs for these aspects are shown in Table VI-11 and total about $380,000. Installation of a blower system would be incidental to this cost estimate.

TABLE VI-11
COST ESTIMATE FOR RIVERSIDE PUMP STATION UPGRADE

ITEM	DESCRIPTION	UNIT PRICE
1	Pumps	$45,000
2	VFDs	$45,000
3	Submersible Motors	$2,000
4	Telemetry/ Controls	$25,000
5	Bypass Pumping	$10,000
6	Piping Modifications	$80,000
7	Electrical Upgrades	$50,000
8	New Generator	$60,000
9	Valve and Meter Vaults	$15,000
	SUBTOTAL	**$332,000**
	Mobilization @ 5% *Subtotal*	$16,600 *$348,600*
	Contingency @ 30% *Subtotal*	$104,580 *$453,180*
	Sales Tax @ 7.7% Subtotal	$34,895 *$488,075*
	Engineering, Admin. & Legal @ 35% **Total Capital Cost**	$170,826 **$658,901**

Prindle pump station also requires an upgrade. This upgrade will replace all three existing pumps with three new 100 Hp pumps to meet the year 2025 projected flow of 8,000 gpm. The new pumps will be rated at 4,000 gpm at 67 feet TDH each. It is recommended the pump motors be dry pit submersible type and include VFDs for low flow conditions. Installation will require core drilling three new 12-inch diameter suction lines into the wet well. These lines will feed each pump individually. Discharge piping will be combined together inside the dry well if space permits. Otherwise, discharge piping will combine outside the pump station and will require valve vaults. The flow meter will be relocated outside the pump station into a separate metering vault and the signal transmitted back to the pump station and ultimately to the WWTP. Electrical upgrades include a new service drop, new panel board, a blower system and a larger 250 kW generator. Bypass pumping will be required during construction. The estimated cost for this work is shown in Table VI-12 and totals about $1,065,000.

TABLE VI-12
COST ESTIMATE FOR PRINDLE PUMP STATION UPGRADE

ITEM	DESCRIPTION	UNIT PRICE
1	Pumps	$275,000
2	VFDs	$90,000
3	Submersible Motors	$30,000
4	Telemetry/ Controls	$50,000
5	Bypass Pumping	$40,000
6	Piping Modifications	$150,000
7	Electrical Upgrades	$30,000
8	New Generator	$100,000
9	Valve and Meter Vaults	$50,000
	SUBTOTAL	**$815,000**
	Mobilization @ 5% *Subtotal*	$40,750 *$855,750*
	Contingency @ 30% *Subtotal*	$256,725 *$1,112,475*
	Sales Tax @ 7.7% Subtotal	$85,661 *$1,198,136*
	Engineering, Admin. & Legal @ 35% **Total Capital Cost**	$419,347 **$1,617,483**

Napavine pump station No. 1, the Fire Station pump station, will be upgraded with new water level transducer and float system similar to the system used by the City of

Chehalis. The cost of this upgrade is about $2,500 for materials and about $2,500 to install by a local electrician. A new blower system will cost about $1,000 including installation. Also the pump station is recommended for a pump replacement in the next ten years to meet the project peak flow demands with one pump out of service. This will require new suction piping to allow the installation of three pumps. Pumps shall be 200 gpm pumps at about 50 feet of TDH, which will require 5 Hp motors. In conjunction with the pump replacement, a flow meter shall be installed on the force main. The estimated construction cost for this upgrade is about $150,000 plus 30 percent contingency and 35 percent legal, engineering and administration costs for a total of about $265,000. This upgrade should be constructed within the next ten years.

Rush Road pump station will need to be upgraded to meet the projected year 2025 peak day flows of 1,200 gpm with one pump out of service. This will require replacement of the two existing pumps with three new pumps and an additional 10-inch parallel force main. The pumps will need to be 20 Hp pumps rated at 600 gpm at 65 feet TDH each. The new 10-inch force main in combination with the existing 6-inch, will also handle the projected year 2050 peak day flow of 1,800 gpm. Headloss at 1,800 gpm will be about 90 feet of head and will require newer pumps rated at 900 gpm at 110 feet TDH, about 40 Hp when upgraded in the future. The wall between the wet well and dry well will need to be core drilled for an additional 6-inch suction line. All three suction lines should be combined together into a common suction header for the third pump. Discharge piping will need to be reconstructed to allow for three pumps and shall include a flow meter. Electrical upgrade may require a new service drop, a new panel board and a blower system. Pump controls will be new level transducers with pump controller. The existing generator should be capable of operating a minimum of two pumps but require that the pumps be equipped with slow start feature to reduce startup current. During construction, bypass pumping will be required while core drilling and replacing piping and pumps. Estimated costs for this upgrade is shown in Table VI-13 and total about $1,810,000.

TABLE VI-13
COST ESTIMATE FOR RUSH ROAD PUMP STATION UPGRADE

ITEM	DESCRIPTION	UNIT PRICE
1	Pumps	$55,000
2	VFDs	$30,000
3	Submersible Motors	$5,000
4	Telemetry/ Controls	$45,000
5	Bypass Pumping	$25,000
6	Piping Modifications	$60,000
7	Electrical Upgrades	$30,000
8	New Generator	$80,000
9	Valve and Meter Vaults	$15,000
10	10-inch Force Main (6,300 feet @ $90/ft)	567,000
	SUBTOTAL	**$912,000**
	Mobilization @ 5% *Subtotal*	$45,600 *$957,600*
	Contingency @ 30% *Subtotal*	$287,300 *$1,244,900*
	Sales Tax @ 7.7% Subtotal	$95,900 *$1,340,700*
	Engineering, Admin. & Legal @ 35% **Total Capital Cost**	$469,250 **$1,810,000**

The Napavine pump station No. 3 located at 2nd and Washington needs to be upsized to meet the projected year 2025 peak day demand with one pump out of service. Since this is a duplex pump station, each pump will need to be sized to meet the peak day flows of 190 gpm each. An audible alarm will need to be installed to meet DOE Criteria for Sewage Works Design. The estimated construction cost for installation of two new submersible pumps with rails and appurtenances is $25,000 plus 30 percent contingency and 35 percent legal, engineering and administration costs for a total of about $44,000. Prior to beginning design, the existing pump capacity should be determined by verification of pump impeller or through actual pump testing. Scheduling of the replacement shall be based upon the current pumps ability to meet projected peak day demands.

Napavine pump station No. 4, located at 3rd and Grand, needs an audible alarm. The cost of installation of an audible alarm is about $250 and can be installed by City of Napavine crews. Also the next time the pumps are serviced, pump model and impeller size should be noted.

Napavine pump station No. 5, located in Napa Estates, also does not have an audible alarm. The cost of installation of an audible alarm is about $250 and can be installed by City of Napavine.

SECTION VII

WASTEWATER TREATMENT SYSTEM ALTERNATIVES

INTRODUCTION

The purpose of this section is to develop conceptual alternatives that will comply with the TMDL, the Consent Decree and the new NPDES permit, which has been issued based on the Consent Decree. The new NPDES permit contains both interim and final effluent requirements which are significantly more stringent than prior NPDES permits. The following is a summary of key elements that direct consideration of various new WWTP alternatives:

- More stringent WWTP discharge limits have been set for BOD_5 and TSS during periods when river flows are less than 1,000 cfs in the Centralia Reach. The new limits are approximately 30% less than currently permitted.
- The outfall must be moved to below the Skookumchuck River during dry weather conditions. Discharge at the current outfall location will only be allowed during wet weather conditions unless the City and DOE reach an agreement on how the WWTP upgrade can be coupled with improving river water quality through river enhancement or discharge of high quality (Class A) effluent.
- New treatment plant facilities must be protected to three feet above the 100-year flood elevation.
- Dry weather discharge is limited to 2.5 or 3.0 MGD by the Consent Decree for river flows of less than 200 or less than 1,000 cfs, respectively. Equalization storage will be required for flows over these limits.
- The Consent Decree can be modified if the City and DOE agree that a proposed alternative will result in improved water quality.
- Ammonia removal (nitrification) is now required for dry weather conditions (river flows less than 1,000 cfs).
- Partial ammonia removal is required for wet weather conditions (river flows over 1,000 cfs).

- All sewage entering the WWTP must be treated to secondary standards, except that the City can apply for reduced percent removal requirements to account for additional inflow and infiltration removal.
- Effluent limits for silver, zinc and copper have been added to the new NPDES permit.

This section will present and evaluate alternatives for both treatment and end-uses of the effluent to meet the requirements of the TMDL and the new NPDES permit. The primary challenge of complying with the new permit is the limitation of no discharge at the current location during dry weather conditions. This means that during dry weather conditions, the discharge location must be five to seven miles downstream, or the effluent must be used for some form of beneficial reuse. Dry weather conditions are when the seven-day moving average of river flow is less than 1,000 cfs. Wet weather conditions apply when the seven-day moving average of river flow is over 1,000 cfs, provided at least one day within the seven is over 2,500 cfs.

It should also be noted that the dry weather limits can apply at any time during the year when the river flow drops below 1,000 cfs (seven day moving average). For instance, if the new flow-based NPDES permit had been in place during the 1976-77 time period, dry weather limits would have started to apply in the middle of May 1976, and would have stayed in effect until the beginning of March 1977, except for about a one week period right around Christmas. Although this is an abnormally long period of low river flows, it demonstrates that the consequences of the TMDL can be very severe in the worst case events.

There are numerous issues and alternatives that must be discussed and evaluated to arrive at a workable solution to the challenges of implementing the TMDL. First of all, potential end-uses of the treated wastewater must be evaluated to determine where the wastewater will be discharged. The end-uses include options for wastewater reuse. However, the most complex evaluation is centered around how to treat the wastewater. The existing Chehalis WWTP has the capability to produce a very clean effluent during normal conditions. But, the facility is almost 50 years old and is subject to secondary treatment process overload and frequent flooding problems during significant storm events.

This report will present and evaluate wastewater treatment options including use of the existing plant, constructing a new treatment plant at the existing site, as well as, constructing a new plant at a new site. To make the evaluation process clear and objective, this section of this report will be presented in the following manner:

I. Alternative Presentation and Screening Evaluation

- Present a description of the required upgrades to the existing plant so that it can produce an effluent which meets the permit conditions through the year 2025.
- Present a description of alternatives for end use of the treated effluent.
- Evaluate the end-use alternatives based on regulatory or operational constraints and eliminate the alternatives that cannot be feasibly implemented.

II. End Use Alternative Evaluation

- Evaluate the remaining alternatives based on the assumption that the existing plant will be upgraded and operated as an extended aeration plant up to the future maximum monthly average flow rate of 4.5 MGD and a complete mix activated sludge plant for flows in excess of 4.5 MGD.
- Determine preferred alternatives for end-use of the effluent.

III. Treatment Process Evaluation

- Present a description of three treatment system options to produce the required effluent for the recommended end uses, which are:
  1. Use the existing treatment plant but operate it as an extended aeration plant up to the future maximum monthly average flow rate of 4.5 MGD and a complete mix activated sludge plant for flows in excess of 4.5 MGD.
  2. Build a new Sequencing Batch Reactor (SBR) plant at the existing site.
  3. Build a new SBR plant at a new site.
- Evaluate the treatment plant options.
- Select a preferred alternative for the treatment plant process.

IV. Solids Train Process Evaluation

- Present a solids train needs assessment based on the selected treatment alternative.

- Present solids process train and alternatives.
- Evaluate solids train and biosolids utilization alternatives.
- Present a recommended solids train and biosolids utilization alternative.

V. Summary

- Present a summary of all recommended alternatives

All of the alternatives presented in this section are based on the estimated future flows and wasteloads presented in Section V of this report, as well as, the effluent requirements discussed in Section III of this report.

MODIFICATIONS REQUIRED FOR USE OF THE EXISTING PLANT

The existing plant will require numerous modifications to produce a quality secondary effluent with ammonia removal all year long. The following is a list of required upgrades to meet the effluent requirements, provide reliability and allow the entire plant to operate during flooding conditions. The modified plant must be capable of meeting the new NPDES permit conditions that are more stringent for both dry weather and wet weather conditions. The modified plant must also be capable of treating the anticipated flows and loadings for the year 2025 that are shown in Section V of this report. For purposes of this report, the improvements have been divided into three categories as follows:

MODIFICATIONS REQUIRED TO MEET NPDES PERMIT

- Secondary Treatment Process Modifications: Change the operation of the plant from a trickling filter plant to an extended aeration and activated sludge complete mix plant. The plant currently operates as an extended aeration plant during the summer and as a trickling filter plant the rest of the year.
- Increase the amount of oxygen supplied to the basins.
- Increase secondary clarifier capacity to improve TSS removal capabilities.
- Add a second influent screen.

- Construct an equalization storage basin since the existing aeration basins will now be used for treatment all year long.
- Construct a flood protection dike around the entire plant.
- Miscellaneous unit process upgrades: Upgrade the plant electrical system, instrumentation and control (I&C) system, yard piping and the lime addition facility.

CAPITAL IMPROVEMENTS NEEDED AT THE PLANT

- Rehabilitate the primary clarifiers.
- Upgrade the chlorine disinfection system and provide an emergency scrubber.
- Rehabilitate the plant wash down water system.
- Modifications to the solids process train (to be presented and discussed towards the end of this section).

OPERATIONAL ENHANCEMENTS

- Conversion of primary clarifiers to center feed.
- Remove the existing Parshall flume.
- Miscellaneous additional upgrades to buildings, I&C, etc.
- Continued removal of I/I in the collection system.

The following is a discussion of the upgrades listed above for NPDES Permit compliance and for Capital Improvements. The operational enhancements are addressed in the cost estimates for the preferred alternatives later in this section.

SECONDARY TREATMENT PROCESS MODIFICATIONS

The existing plant will be modified to operate as an extended aeration plant in low flow conditions and a complete mix conventional activated sludge plant during high flow conditions. The plant currently operates as a trickling filter plant during most of the year and as an extended aeration plant providing nitrification during the summer. The trickling filters serve as roughing filters to the aeration basin when the plant operates in the extended aeration mode. The plant was designed to operate as a trickling filter plant with the aeration basin provided to treat heavy industrial needs. In 1995, the City started using the aeration basins for nitrification to provide ammonia removal during summer months. If the existing WWTP is upgraded to meet the TMDL and new NPDES permit, then the two existing aeration basins will be used year-round as aeration basins. Both of the existing aeration basins would be used at all times, but the process will be different depending upon influent flows. New equalization storage basins will be built to provide needed capacity for both dry weather and wet weather conditions. The existing trickling filters would not be used in this treatment scenario but may be retained to handle high BOD_5 loading from potential future industries. If the trickling filters are used at a later date, they will serve only as roughing filters ahead of the aeration basins. The existing primary clarifiers would continue to be used and two new 65-foot diameter secondary clarifiers would be built to increase TSS removal capabilities.

In order to maximize nitrification and BOD_5 removal during low flow conditions, the plant would be operated in the extended aeration mode. This process is characterized by a hydraulic retention time (HRT) of 10-24 hours and a mixed liquor suspended solids (MLSS) concentration of 2,000 to 6,000 mg/l with continuous aeration to keep the dissolved oxygen (DO) above 2 mg/l. This process can be used for flows up to 4.5 MGD. One basin is adequate up to 1.5 MGD. However, high BOD_5 loading may require bringing the second basin on line, even during low flow. Flows between 1.5 and 4.5 MGD can be treated in the extended aeration mode using both basins.

As the inflow increases during wet weather conditions, the effluent limits for TSS, BOD_5 and ammonia are not as stringent as they are during the critical dry weather period. When this happens, the plant can operate as a complete mix activated sludge process. This process is characterized by a 3-5 hour HRT and a MLSS concentration of 3,000 to 5,000 mg/l. Inflows between 4.6 and 5.0 MGD will be treated in just one basin in the complete mix mode. When inflows are between 5.1 and 9.5 MGD, they will be treated using extended aeration in one basin and complete mix in the other basin. Flows in excess of 9.5 MGD will be treated in both basins using the complete mix process. The existing aeration basins will be able to produce an effluent that will meet NPDES permit requirements for BOD_5 and ammonia up to 13.0 MGD which is the projected 2025 daily maximum flow.

The DOE design criteria for the extended aeration process calls for an HRT of 10 - 24 hours and 3 - 5 hours for the complete mix process. Table VII-1 shows how the plant will be operated with varying inflows:

TABLE VII-1
MODE OF OPERATION VS. INFLOW

	Extended Aeration (HRT=10-24 Hrs)		Complete Mix (HRT=3-5 Hrs)	
Inflow	**Flow**	**HRT**	**Flow**	**HRT**
1	1	*23	0	--
1.5	1.5	*15	0	--
2	2	23.0	0	--
2.5	2.5	18.3	0	--
3	3	15.3	0	--
3.5	3.5	13.1	0	--
4	4	11.5	0	--
4.5	0	--	4.5	*5
5	0	--	5	*4.6
5.5	1	23	4.5	5.1
6	1.5	15	4.5	5.1
6.5	2	11.5	4.5	5.1
7	2	11.5	5	4.6
7.5	2	11.5	5.5	4.2
8	2	11.5	6	3.8
8.5	2	11.5	6.5	3.5
9	2	11.5	7	3.3
9.5	2	11.5	7.5	3.1
10	0	--	10	4.6
10.5	0	--	10.5	4.4
11	0	--	11	4.2
11.5	0	--	11.5	4.0
12	0	--	12	3.8
12.5	0	--	12.5	3.7
13	0	--	13	3.5

** Using 1 Basin Only except when BOD_5 loading to one basin exceeds 3,000 PPD*

The modified plant must be capable of handling the anticipated maximum month BOD_5 loading for the year 2025, which is 5,500 ppd. The primary clarifiers remove approximately 25% of the influent BOD_5 on average. After primary clarification, this leaves 4,125 PPD (5,500 - 25%) of BOD_5 that needs to be treated. The DOE design criteria calls for BOD_5 loading to be 10 -25 lbs/1,000 cf of aeration basin capacity for extended aeration and 20 - 120 lbs/1,000 cf for complete mix. The combined volume of both aeration basins is 255,000 cf. For extended aeration, the basins have a combined capacity of 2,553 to 6,383 PPD of BOD_5 based on allowable loading rates for pounds per basin volume. They are therefore adequate for BOD_5 loading with, or without, the primary clarifiers. For complete mix, the BOD_5 capacity is 5,107 to 30,640 PPD based on allowable loading rates for pounds per basin volume. The existing aeration basins are more than adequate for either mode of operation.

Switching the mode of operation of the plant will require complex flow splitting between the basins. The plant operations staff will also need to adjust sludge recirculation and wasting rates to maintain the optimal MLSS concentration.

The basins were originally constructed in 1970 with the intention of using them to treat high BOD_5 loading from Darigold, National Frozen Foods, Perry Brothers Meat and other industries. However, several companies went out of business and National Frozen Foods and Darigold no longer discharge a majority of their process wastewater to the Chehalis plant. By today's design standards, the basins are less than desirable because of the shallow depth (9.5 feet) and the sloped side walls. Ideal basins are at least 12 feet deep and have vertical walls to facilitate aeration within the entire basin.

The current aeration system for the basins consists of four 15 HP fixed dual speed aerators in each basin. The oxygen transfer rate is controlled by the speed of the aerators and a weir in the basin that raises or lowers the water level and thereby increases or decreases submergence and oxygen transfer to the water. The aerators were not sized with the intent of using them for nitrification and are therefore not adequate to handle the required oxygen production to treat the BOD_5 and fully nitrify ammonia under expected future conditions.

The required aeration system must be designed for future conditions based on the following assumptions:

- Influent BOD_5 loading is 5,500 ppd
- No reduction of BOD_5 in primaries or tricking filters (if used)
- Oxygen required for BOD_5 reduction is 1 lb. oxygen / lb. BOD_5 applied
- Influent ammonia loading is 830 PPD

- Influent Total Kjeldahl Nitrogen (TKN) is 50% higher than ammonia: Influent TKN is 1,245 ppd
- Oxygen required for nitrification is 4.6 lb. oxygen / lb. TKN applied

The required oxygen is 11,230 PPD (5,500 X 1 + 1,245 X 4.6). For this report a design value of 12,000 PPD will be used. This is the total amount of oxygen required in both basins. So, each basin will need to have equipment that can deliver 6,000 PPD of oxygen.

Several types of aeration equipment were evaluated for this report and are shown below:
- Replace the dual speed fixed aerators with larger units
- Install a fine bubble diffused air system and blower
- Install high speed floating aerators
- Install suspended fine bubble diffused air system (similar to Biolac by Parkson Corporation)

The existing dual speed fixed type aerators are mounted on four supports in each basin and have an access bridge and platform to each of them. This aeration option is to replace the units with larger units in the same mounts. The mixers will need to be replaced with single slow speed 50 HP units. Four mixers will be required in each basin which will produce 200 HP per basin and 0.42 HP/1,000 gallons. The existing fixed moorings will have to be modified to accept the larger units.

The most common way to aerate and mix an activated sludge basin is to use blowers to supply air through a grid of fine bubble diffusers installed in the bottom of the tank. However, the limited depth of the basins makes it very difficult for the fine bubble diffusers to create a rolling action that promotes mixing. Also, the sloped side walls preclude the use of fine bubble diffusers because the diffuser grid must be installed at the same elevation throughout the entire basin to assure an even distribution of air. Therefore, this option was eliminated from further consideration.

One of the few methods of aeration that will provide even air distribution in an awkward basin configuration is to use high speed floating aerators similar to the ones installed in sewage lagoons. This option will work but it was eliminated from further consideration because of the operations staff concerns. In 1980, the aeration basins were equipped with floating aerators and the City switched them out to the existing dual speed fixed aerators to get better access for maintenance.

Another option that will work and is acceptable to the operations staff is to use the type of suspended fine bubble diffusers that are the mainstay of the Biolac treatment plant design. The diffusers are suspended beneath a flexible air supply hose that oscillates to promote mixing. The suspended diffusers in this case, will be supplied air with three blowers (one redundant) each rated for 4,000 scfm at 5 psi and powered by a 150 HP motor. Due to the high pressure and discharge volume required, the blowers will need to be the positive displacement type. The blowers will be installed in the existing dissolved air flotation thickener (DAFT) building that is currently used for storage of spare parts. The existing bridge and platforms in the aeration basins will need to be removed since they will interfere with the operation of the moving aeration chains.

It is recommended that new slow speed fixed aerators/mixers be used with the modified treatment plant. The fixed aerators can supply the required oxygen and mixing for the least cost and are the most acceptable method with regard to operation and maintenance.

INCREASE SECONDARY CLARIFIER CAPACITY

The two existing secondary clarifiers do not adequately remove TSS, which results in numerous permit violations for effluent TSS concentration and percent removal during high flow conditions. One of the secondary clarifiers is the old spiraflow type and is not very efficient. It is also shallow and does not meet the depth requirements prescribed in the DOE design criteria. The other secondary clarifier was built in 1987 and is 18-feet deep with center feed and flocculating center well. It has excellent TSS removal capacity as long as the overflow rates are kept within design standards. The DOE design criteria calls for an overflow rate of 400 - 600 gpd/sf for the extended aeration process and 600 -

800 gpd/sf for all other activated sludge processes with a peak overflow rate of 1,200 gpd/sf. With two new clarifiers, the total area will be 13,268 sf. At the future maximum monthly average flow of 6 MGD, the overflow rate is 450 gpd/sf which is near the low end of the range for the extended aeration process and is more than adequate for the complete mix process. At the 13 MGD peak flow, the overflow rate is 980 gpd/sf which is less than the recommended peak overflow rate of 1,200 gpd/sf. During the summer months only one or two clarifiers will be used. As flow increases, additional clarifiers will be brought on line with the oldest one put into service last. A new flow splitter box will be built to proportion flow between the clarifiers. By providing two new clarifiers, there will be adequate TSS removal capability with one unit out of service.

Therefore, two additional 65-foot diameter secondary clarifiers are recommended. The clarifiers will be the same design as the one built in 1987 and will be 65-foot diameter and will have a side water depth of 18 feet. They will be the center feed type with flocculating center well and will be located just east of the recently built clarifier.

SECOND INFLUENT SCREEN

A screen is required to remove rags, plastic and rubber goods that clog equipment and wind up in the dried sludge that is used for soil amendment. The plant currently has one automatic screen at the headworks facility. It is a Hycor Helisieve Model 500 and is rated for 5.3 MGD. During high flows, the flow stream is split so that part of the flow goes through a manual coarse bar screen that does not provide consistent adequate screening. It is therefore recommended that a second similar unit be installed so that the combined capacity is 10.6 MGD. A second unit will also allow for redundancy for maintenance and repair. The manual bar screen will be maintained in this case, for infrequent periods when flow exceeds 10.6 MGD.

NEW EQUALIZATION STORAGE BASIN

The existing aeration basins have a dual purpose; in the summer they serve as aeration basins for nitrification and in the winter they serve as equalization storage basins to moderate high inflows caused by storms. If upgrading the existing WWTP is the selected alternative, then the basins would be used year-round as aeration basins for secondary treatment and cannot serve as equalization storage. Therefore, a new equalization storage basin will be constructed just east of the existing aeration basins. The size of the equalization storage basin is variable depending on the end use option. An analysis of equalization basin storage requirements is presented in Appendix E and specific storage requirements are discussed in detail under the description for each of the alternatives later in this section.

FLOOD PROTECTION

The existing plant site lies in the both the flood plain and the floodway of the Chehalis River and Dillenbaugh Creek. The plant has experienced numerous flood events in the past 20 years, and more recently seven times in the last five years. These floods have caused several problems including:

- Structural damage to the concrete basins (only one instance).
- Over-topping treatment process basins.
- Prevented plant personnel from getting to the plant.
- Prevented plant personnel from performing required sampling.
- Prevented plant personnel from effectively operating the treatment plant.
- Increased operational staff time to prepare for, and clean up after floods.

Operational and construction requirements within the flood plain and floodway were determined through conversations with City of Chehalis Community Development Division, the U.S. Army Corps of Engineers (USAC) and review of DOE design guidelines. The following criteria must be met at the existing WWTP site:

Existing Structures

Under local codes, there are currently no interim floodway or flood plain related requirements for maintenance or repair activities at the WWTP. However, DOE design standards require that all treatment facilities have operation components located above the 100-year flood/wave action or be protected from the 100-year flood/wave action. It also requires that the plant remain fully functional during a 25-year flood/wave action. From Section IV of this report, the anticipated revised 100-year flood level is 179.5-feet.

The anticipated wave height is approximately 1-foot. Therefore, to prevent overloading of the existing treatment basins and to prevent raw or partially treated sewage from entering the floodwaters, all existing basins must be protected to an elevation of 180.5-feet.

New Structures

Specific construction requirements for new facilities are dependent upon whether or not the proposed facilities are within the floodway or flood plain. Both the State and the City have regulations regarding development within the designated flood plain. State flood plain management regulations are found in RCW 86.16. These regulations are implemented through the City Land Use Codes. The Flood Plain Management portion of the City's Land Use Code contains regulations regarding development within the 100-year flood plain as defined by current FEMA maps. Under current Land Use Code, new development is limited within the flood plain. A structure can be built within the flood plain area, but must have a finished floor elevation (or equivalent protection) at least 1-foot above the 100-year flood elevation. In addition, the code requires that essential facilities within the flood plain be protected to a minimum of 3-feet above the 100-year flood elevation. Preliminary discussions with the City of Chehalis indicate that the WWTP and pump stations should be considered essential facilities for planning purposes. This means that the new facilities should be constructed with finished floors or top of walls at 182.5-feet under the anticipated 100-year flood level designation of 179.5 feet.

Construction within a floodway is much more restrictive than flood plain construction. Floodway construction requires approvals of a floodway revision consistent with the Lewis County Flood Hazard Management Plan, as well as approval of a 404 Permit by the United States Army Corps of Engineers (USAC). The 404 Permit approval entails an environmental assessment by the USAC Regulatory Branch. A floodway capacity revision may require physical expansion or rerouting of the floodway to offset any net decrease in the floodway resulting from the floodway construction. If a floodway revision is not practical, an engineering analysis must show that proposed floodway construction does not impact 100-year flood levels at adjacent properties.

Diking within the floodway to protect the WWTP will likely be provided to 3-feet above the 100-year flood stage or to above the 500-year flood stage, which ever is greater. The dike will be built to an elevation of 182.5-feet to ensure that the WWTP is protected from future flood levels. If the County flood mitigation project that is discussed in Section IV of this report is implemented, the dike may be built to an elevation as low as 180.5-feet.

Due to the anticipated negligible floodway obstruction from specific future plant improvements, it is anticipated that the required floodway offsets and negative declaration under NEPA can be obtained by the City in a reasonable and timely manner for individual basins or facilities. However, full diking of the WWTP site will require detailed hydraulic modeling and/or potential floodway revisions. If equalizing storage basins are constructed using diked walls within the floodway, they will require similar floodway offsets and mitigation.

The existing plant has numerous key facilities that are constructed below 180.5-feet and are shown below:

- Trickling filters
- Aeration basins
- Sludge drying beds
- Sludge storage basin

Figure VII-1 shows the existing WWTP site plan with the elevations of the top of wall or finished floor for the plant facilities called out. Several options for flood-proofing the plant were developed and evaluated including raising basin walls, constructing small ring walls (individual diking) around each unit process basin or group of basins that are below 180.5-feet, as well as constructing a complete dike around the entire plant site.

The concrete walls in the aeration basins, trickling filters and the aerobic digester would need to be raised by approximately 4-feet (to elevation 180.5-feet) to provide adequate protection from floods. This is not feasible due to structural and geotechnical concerns. This option was therefore eliminated from further consideration.

Concrete ring walls can be constructed around each basin or group of basins for flood protection with independent footings. The walls need to be constructed to an elevation of 182.5-feet because they are considered a new structure. Sheet pile walls were considered for this option, but cannot be used due to conflicts with yard piping in and around the basins. However, all of the main treatment basins, except for the primary and secondary clarifiers will require protection (see Figure VII-1). The total length of

INSERT FIGURE VII-1 EXISTING SITE PLAN

wall construction to protect each basin under this option is comparable to complete diking, but does not provide equivalent flood protection. Therefore, it is not feasible to implement this option relative to the benefits and cost of complete diking.

The remaining option to protect the plant from future floods is to construct a dike around the entire site. The dike would be constructed to an elevation of 182.5 feet and would surround the entire plant site including the expanded area to the east to make room for an equalization storage basin and two new secondary clarifiers. The proposed dike is shown in Figure VII-2. The dike would range from 5-1/2 to 8-1/2 feet above the existing ground level. The existing plant entrance will not be diked. Instead a flood door will be used to prevent floodwaters from entering the site during flood events. The flood door will be removable.

Several options were evaluated to access the site during a flood including:

- Using an inflatable boat to ferry personnel and supplies to and from the site.
- Raising Shoreline Drive above the flood level.
- Building a pedestrian bridge to the site.

An inflatable powerboat could be used to carry personnel and supplies to and from the site. The operations staff has voiced concern over safety issues with this option.

Raising Shoreline Drive to an elevation of 182.50 feet was also considered. The road would have to be raised by seven to eight feet. In addition, huge box culverts would need to be installed to pass the flood waters under the road. This option was eliminated from further consideration due to the high cost and difficulty in obtaining required construction permits.

A pedestrian bridge could also be constructed from Louisiana Avenue, which is above the 100-year flood level, to the plant site. The bridge would span the extensive wetland just east of the site. This option is feasible, but it would cost much more than an

INSERT FIGURE VII-2 PROPOSED DIKE

inflatable powerboat. The tentative recommendation is to use an inflatable powerboat for plant access during floods. This recommendation will need to be evaluated during the facility planning process after the other cost figures have been refined.

This option will also require floodway improvements to the east of the existing site to give the displaced floodwater a place to go. It is anticipated that although the proposal for a complete dike would be difficult to implement and permit, it can be done. The permitting process would be very long and expensive. It would also require coordination between numerous local, state and federal agencies. A storm water pump station would also be required inside the dike so that the rainwater that enters the site can be removed. In addition, all plant drains within the dike would need to be valved to prevent floodwater from entering the site through storm drains.

Three options of dike construction were evaluated and include:

- Earthen dike
- Steel sheet pile wall
- Vinyl sheet pile wall with soldier piles for extra support

The dike can be constructed using an earthen dike with 2:1 side slopes on the outside face and 3:1 side slopes on the inside face and a 12-foot wide top section. The native soils are not suitable for dike construction so fill material will need to be imported to construct the dike. The native soils are also subject to "piping" underneath the dike and can cause problems and potentially a dike failure (SCS). The earthen dike will be a total of 55 feet wide in some places which will take up a lot of valuable space at the already crowded site. In addition, there is no way to build such a wide dike where the plant site borders the River or Shoreline Drive. Therefore, the concept of an earthen dike was eliminated from further consideration.

The second option for constructing a complete dike is to use interlocking steel sheet piles to form a wall. This type of wall is only two feet wide and the piles can be driven right next to the top of bank on the west side of the plant. These “Z” piles would be driven in the ground to a depth of approximately 20 feet. The sheet pile wall would be self-supporting. A concrete cap would be constructed at the top of the wall to minimize leaking and to provide some aesthetic value. In some places, the dike wall would also serve as the side wall of the equalization storage basin (see Figure VII-2). In these areas, the sheets will be driven to a depth of about 30 feet. One advantage of this type of dike is that because the piles are driven into the ground twenty to thirty feet, they effectively cut off any “piping” action. Several yard piping and electrical conduit runs would need to be relocated under this option.

The third option for dike construction is to use a single row of vinyl sheet piles with steel H-piles installed at eight feet on center for extra support. The vinyl sheet piles are driven into the ground the same as steel sheet piles but they would only be driven to a depth of approximately 10 feet. The sheet piles would also serve to limit “piping” of water through the soil. The H-piles would be driven in to a depth of approximately 30 feet and provide much of the wall's strength. Treated wood walers would be installed between the vinyl sheet piles and the H-piles to provide support and for aesthetics. Either type of sheet pile wall can be used to construct a dike. Final selection of the sheet pile wall can be made later in the planning process since the walls are about the same cost and it is uncertain if this diking option will be implemented.

PLANT ELECTRICAL SYSTEM UPGRADE

The main upgrade required to the electrical system is to increase the capacity of the emergency generator. All of the options require additional horsepower for the aeration system that cannot be handled by the existing emergency generator. The required increase in generator capacity for each individual option will be presented as part of the option description.

The plant's electrical system was completely upgraded in 1980-81 at which time, additional spare conduit runs were installed for future use. An additional electrical system upgrade was performed in 1992. All switches and critical electrical components were raised to an elevation of 180.0 feet to protect the electrical system from floods.

PLANT INSTRUMENTATION AND CONTROL (I&C) UPGRADE

The plant's I&C system was completely replaced in the 1980 upgrade. However, most of the technology developments in the I&C area have occurred in the last 10 to 15 years. As early as the late 1980's, spare parts for some the I&C components were no longer available.

The City undertook a major supervisory control and data acquisition (SCADA) and upgrade in 1994 to install and integrate new conditions into the plant systems. The original work is complete, but many more improvements are needed. It is recommended that the I&C system and SCADA be modified to meet current City requirements.

PLANT YARD PIPING UPGRADE

The plant has undergone four major upgrades and numerous smaller improvements since it was constructed in 1948. It is almost impossible to put a new pipe or buried conduit in the ground anywhere on the site without hand digging to avoid other pipes and electrical conduits. There are several pipe runs that have been abandoned in place and even though there are record drawings available, it is very difficult to determine which pipes are still in service. The proposed plant upgrade would require many more pumps and pipes especially for the addition of two new secondary clarifiers and potentially an advanced treatment train for some of the reuse options. It is likely that these new pipes and conduits would have to be routed around the perimeter of the plant to avoid conflicts with existing pipe and electrical runs.

LIME ADDITION FACILITY

The plant currently accepts the effluent from Darigold's WWTP during the summer to enhance the nitrification process. The Darigold effluent is rich in alkalinity, which is required for proper nitrification. The plant also has a lime feed point set up between the trickling filters and the aeration basin to augment the alkalinity from Darigold and to raise the pH. However, in the future, Darigold will be land applying their effluent during the summer and will not be going to Chehalis' plant. It is therefore recommended that the existing lime addition facility be upgraded to a capacity of 500 ppd. This will require a change over from the existing bags to large totes due to the amount of lime that will need to be added on a daily basis. An on-site storage silo may also be required. A more permanent structure is also required to house the new storage and feed equipment. The application point in the process stream should stay the same.

REHABILITATE PRIMARY CLARIFIERS

The existing two primary clarifiers are 50 foot diameter each with a total area of 3,925 sf. The DOE design criteria calls for an overflow rate of 800-1,200 gpd/sf at average flow and 2,000 - 3,000 gpd/sf at peak flow. At 4 MGD, which is the future average wet weather flow rate, the overflow rate is 1,020 gpd/sf which meets the design criteria. The overflow rate at 6 MGD is 1,530 gpd/sf which is less than the allowable rate of 2,000 – 3,000 gpd/sf for peak flows. At the peak flow rate of 12 MGD, the overflow rate is 3,057 gpd/sf which is adequate. The primary clarifiers will be used as they are now with only one unit in service during the summer and both units in operation during the winter.

Both primary clarifiers are 50 years old, but were upgraded in 1980-81. The baffles and interior launderer are at the end of their design life and need to be replaced because of corrosion and cracking. The gear drives on both units are subject to frequent failure and deemed to be unreliable. The primary clarifiers should be converted to the center feed type in order to increase performance. However, they are not a critical process component in the proposed extended aeration treatment process. Therefore, replacement of the feed mechanism is recommended as an operational enhancement only.

DISINFECTION SYSTEM UPGRADE

The existing disinfection system consists of two chlorine contact chambers, two 500 PPD chlorinators, a 500 PPD sulfonator, and an oxygen reduction potential (ORP) controller. The chlorine and sulfur dioxide are stored in 150-pound cylinders. The chlorination system has a capacity of 13 MGD but the dechlorination system is limited to 7.5 MGD because there is no sulfur dioxide feed point to Chlorine Contact Chamber No. 3.

Ultraviolet (UV) light disinfection was considered for use on this project as an alternative to the continued use of the chlorination/dechlorination system due to concerns with chlorine and sulfur dioxide safety, strict discharge limits on residual chlorine, and the potential of future regulations addressing chlorination byproducts. The reclaimed water standards require that a chlorine residual of 0.5 mg/l be maintained at the furthest application point in the reclaimed water distribution system. UV can be used instead, but a regular and maintenance intensive flushing program will have to be initiated to prevent a slime layer from building up inside the reclaimed water pipeline.

If the treated effluent is pumped to a new outfall located downstream of the Skookumchuck River, chlorination is required to minimize slime from forming in the seven-mile long force main. The treated effluent will be high quality secondary effluent, not Class A reclaimed water, and will have organic material in it that will enhance a bacterial slime layer. This condition may cause NPDES permit violations when large pieces of the slime layer break off and are carried downstream to the effluent sample location near the downstream discharge location. For this option, a dry weather dechlorination facility is required near the downstream outfall location to avoid residual chlorine violations.

Some of the options will allow continued discharge at the current outfall location all year long. For these options, UV light should be used for disinfection.

Both the chlorinator and sulfinator are near the end of their design life and in need of replacement. The chlorinator should be sized for a peak demand of 500 PPD based on a flow of 6 mgd at a feed rate of 10 mg/l and the sulfinator should be sized for a peak demand of 250 PPD.

Hazardous Material Scrubber: Current EPA and fire code regulations require that a hazardous material scrubber be installed at chlorine and sulfur dioxide facilities that have on-site storage volumes that exceed a threshold requirement. The threshold requirement is 1,500 pounds for chlorine and 1,000 pounds for sulfur dioxide.

Future conditions should allow the continued use of 150 pound cylinders which will not require a scrubber as long as the active bottles are within the above weight limits.

It should also be noted that the reuse standards require a standby chlorinator and a continuous chlorine residual monitoring. The existing chlorinator would be retained for this purpose. The plant currently has two continuous chlorine residual monitors.

PLANT WASH DOWN WATER SYSTEM UPGRADE

The plant wash down water system is experiencing numerous failures. Repairs are relatively expensive and require surface restoration each time the line breaks. It is recommended that new piping be installed for the plant wash down water system to eliminate this problem.

TREATMENT AND END USE ALTERNATIVES

The following is a discussion of the alternatives for treatment and end-uses of the treated wastewater.

Alternative Presentation and Screening Evaluation

There are numerous options available to meet the effluent requirements imposed by the TMDL. The treatment and end use alternatives are presented below based on six major categories:

1. Regional WWTP Alternatives

   A. Join in a regional plant in Centralia and abandon the existing plant.

B. Pump raw sewage to a regional plant in Centralia for dry weather flows only; use the existing treatment system with partial ammonia removal for wet weather flows.

2. Dry weather discharge to the Chehalis River below the Skookumchuck River confluence (wet weather discharge at the existing outfall with partial ammonia removal).
3. Enhance the river to allow continued discharge at the current outfall location all year long.
4. Alternatives with wastewater reuse with no surface water discharge during dry weather conditions.
   - A. Class A Reclaimed Water (requires coagulation and filtration).
     - i. Groundwater recharge via infiltration.
     - ii. Poplar or Cottonwood tree irrigation with groundwater recharge in spring and fall.
   - B. Class B Reclaimed Water
     - i. Poplar or Cottonwood tree irrigation with storage pond for spring and fall; meter pond back to river in the fall.
     - ii. Natural wetland recharge.
     - iii. Constructed beneficial use wetland recharge.
5. Store all dry weather effluent flows and discharge to the river when wet weather limits apply.
6. Class A reclaimed water for streamflow augmentation of the Centralia Reach during dry weather conditions.

REGIONAL WWTP ALTERNATIVES

Two basic alternatives utilizing regional WWTP facilities to combine or coordinate service between the Chehalis and Centralia service areas have been considered. The basic alternatives include: 1) processing all raw sewage at a regional plant in Centralia on a year-round basis or 2) processing Chehalis raw sewage at the proposed Centralia WWTP only during low river flow conditions and using the existing Chehalis WWTP (upgraded) facilities during high river flow conditions.

ALTERNATIVE IA- YEAR-ROUND REGIONAL WWTP

This option entails abandoning the existing WWTP in Chehalis and pumping all raw sewage to a regional plant to be located in Centralia. Peak daily inflows will be as high as 13 MGD and minimum summer flows will be as low 1.0 MGD. Current and future design parameters are listed in Section V of this report. The following is a summary of the key features:

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Must be determined in a joint NPDES permit for both Centralia and Chehalis for wet weather and dry weather conditions (likely incorporating the sum of both allocations).
- The 2025 peak wet weather flow will be 13 MGD with an average wet weather flow of 2.9 MGD and a maximum monthly flow of 6.0 MGD. The 2025 maximum average daily dry weather flow rate is 3.5 MGD for river flows less than 200 CFS and 3.0 for river flows between 200 and 1,000 CFS. Flow conditions will require a minimum of 6.0 MG of equalization storage for use in either wet or dry weather conditions (See Appendix E).
- Reliability: Class II

The description of the recommended treatment plant for this regional option is detailed in the Centralia Facilities Plan that was prepared by CH2M Hill in 1998. The Facilities Plan includes the unit process equipment and basins that are required to treat the anticipated flow from Chehalis. The Centralia Facilities Plan recommends construction of a conventional complete mix activated sludge treatment plant consisting of headworks, primary clarifiers, aeration basins, secondary clarifiers, UV disinfection, post treatment aeration, effluent pumping and discharge to the Chehalis River. The aeration basins are set up with anoxic selectors to promote nitrification. The primary clarifiers are only needed if Chehalis joins in a regional plant.

Equalization Storage

Influent equalization storage in the amount of 6.0 MG will be provided to comply with the dry weather flow restrictions in the Consent Decree for the downstream discharge location. The amount of equalization storage required was calculated using an Excel spreadsheet model. See Appendix E for a discussion of the model and the required storage volume required for all of the options discussed in this section. The existing 2 MG of aeration basin capacity will not be used for equalization storage because the walls are not high enough to protect them from floods. It is not structurally possible to raise the aeration basin walls the required 4.5 feet.

The equalization storage basin will also serve to minimize the wet season capacity for a new raw sewage pump station. The equalization storage basin will be located where the existing aeration basins are now and will also take up some vacant area just east of the aeration basins. Figure VII-3 shows a schematic diagram of this option and Figure VII-4 shows the proposed site layout for this option.

INSERT FIGURE VII-3 OPTION 1A SCHEMATIC

INSERT FIGURE VII-4 OPTION 1A SITE PLAN

The equalization storage basin will be constructed with an earthen dike. The top of the dike will be at elevation 182.5 feet which is the 100 year flood level plus three feet of freeboard to account for local building requirements for essential facilities. The maximum water surface level in the equalization storage basin will be at elevation 181.0 feet. The dike will have 3:1 side slopes on the interior and 2:1 side slopes on the exterior and will be lined with 60 mil high-density polyethylene (HDPE).

The elevation at the bottom of the storage basin will be 165.0 feet resulting in a maximum side water depth of 16.0 feet. Floating aerator/mixers will be provided to keep the raw sewage mixed and aerated as needed. The aerators will have a total power input of 0.04 HP per 1,000 gallons of storage volume, which is 240 HP.

During the summer, inflows in excess of the allowable discharge downstream (2.5 or 3.0 MGD depending on the River flow condition) will be routed to the equalization storage basin. As inflows decrease, the storage basin will be drawn down and pumped to the regional plant. During the winter, the equalization storage basin can be used to store high storm flows, thereby decreasing the required size of the new raw sewage pump station and force main.

Existing Plant Facilities

Under this option all of the existing plant facilities would be abandoned and demolished. The plant site is currently in the floodplain and floodway of the Chehalis River and Dillenbaugh Creek. It is anticipated that removing the structures would help mitigate for construction of the new raw sewage pump station, force mains and the equalization storage basin.

Raw Sewage Pump Station

Raw wastewater pumping will be accomplished by constructing a new central pump station at the current Chehalis WWTP site. The peak capacity of the new pump station should be 12 MGD. Two 2.0 MGD variable frequency drive (VFD) and three 4 MGD fixed speed pumps will be required to provide a firm capacity of 12 MGD with one of the larger pumps out of service. The pump station will be capable of pumping a minimum flow rate of approximately 1.0 MGD at a two to one turn down ratio on the smaller pumps. The finished floor of the pump station would be at elevation 182.5 feet, which is three feet above the 100-year flood elevation. Standby power facilities would also be provided to ensure full operation of the facility during power outages. The raw sewage pump station will be built next to the new equalization storage basin using caisson construction methods. The caisson would be divided into a wet well and dry well. The station would be equipped with dry pit centrifugal sewage pumps powered by submersible motors.

Each of the small pumps would be rated for 1,400 gpm at a TDH of approximately 140 feet and will have a 75 HP motor. Each of the large capacity pumps would be rated for 2,800 gpm at a TDH of approximately 90 feet and would have a 100 HP motor. The bottom of the wet well should be at elevation 162.0 feet so that the equalization storage basin can be drawn down by gravity without cavitating the pumps.

Force Main

A dual force main is required to address the wide range of wastewater flows. Preliminary estimates indicate that 16-inch and 24-inch diameter force mains would attain minimum scouring velocity, allow for minimum sewage retention under low flow conditions, and still be able to handle peak flows. The preliminary force main route is shown on Figure VII-5. The proposed force main route leaves the plant and goes west on Shoreline Drive and Brace Street and then heads north on Georgia Street to Florida Avenue to NW Airport Road to Airport Road to the Centralia WWTP near Mellen Street in Centralia. At the existing Centralia WWTP site, the 16 and 24-inch force mains would continue in the same alignment proposed for the two force mains proposed by Centralia from their existing WWTP site to the proposed new WWTP site located in the Fords' Prairie area northwest of Centralia. The total length of the dual force main from the existing Chehalis plant to the proposed Centralia WWTP site alternative No. 1 is approximately 38,000 feet.

The entire alignment from Chehalis to the existing Centralia WWTP is located in either City or County owned roads and no easements would be required. The alignment from Centralia's existing WWTP site to the new regional plant would be the same as the Centralia force main route presented in the Centralia Facilities Plan. Some easements would be required from the City of Centralia, the Port of Centralia, the Department of Fish and Wildlife and private property owners. Since the original draft of this report was written, Centralia has decided to move the location of their proposed plant further downstream near the Thurston County border. Therefore, the forcemain for a potential regional plant would be even longer than to potential Site Alternative 1.

If this alternative is implemented, additional analysis and coordination with the City of Centralia will be necessary to determine if the Centralia pump station can be expanded to accommodate either dry and/or wet weather flows from Chehalis.

Operation

This option would require significant cooperation between the two cities. It is anticipated that a single operation staff, which would probably consist of personnel from both cities, would operate the regional WWTP and each entity would continue to maintain respective collection system facilities. If this alternative is selected, a benefit/cost analysis, a force main route evaluation (including the analysis of shared force mains) and an interlocal agreement will be required to formalize the details to fairly allocate costs between each of the cities.

INSERT FIGURE VII-5 FORCE MAIN ROUTE

ALTERNATIVE IB- DRY WEATHER REGIONAL PLANT

This alternative entails using "excess capacity" of the proposed new Centralia WWTP during dry weather conditions. The premise behind this option is that the new Centralia plant would be built with enough capacity to handle the high winter flows that are much larger than the dry weather flows, and there would be unused capacity during the dry season. Under this alternative, raw sewage would be pumped to the Centralia raw sewage pump station located at the existing Centralia WWTP site where it would commingle with the Centralia raw sewage and then be pumped to the new Centralia WWTP when the dry weather limits apply. The existing Chehalis Plant would be upgraded and continue to operate during wet weather conditions and discharge at the existing outfall location.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES permit for wet weather conditions and combined permit limits for dry weather conditions.
- Effluent limits for silver, zinc and copper to be established through further water quality testing and analysis.

- Maximum average daily flow rate is 2.5 MGD for river flows less than 200 CFS and 3.0 for river flows between 200 and 1,000 CFS which requires 6.0 MG of equalization storage.
- Reliability: Class II

Force Main

A single force main would be required to transport raw sewage to the proposed Centralia raw sewage pump station during the dry season. The force main would be 16 inches in diameter and would follow the same route as option 1A up to the existing Centralia WWTP. The Chehalis raw sewage would commingle with the Centralia raw sewage at the new Centralia raw sewage pump station and would be transported to the new Centralia WWTP in one of the two proposed force mains between the existing and new Centralia WWTP site.

Equalization Storage

As with the first option, 6.0 MG of equalization storage would be required to keep the Chehalis portion of the dry weather discharge below that prescribed in the Consent Decree. The equalization storage basin for this option would need to be constructed east of the existing aeration basins since they would be kept in service. In order to save space, the equalization storage basin would be constructed using a sheet pile wall instead of an earthen dike. Figure VII-6 shows a schematic diagram of this option and Figure VII-7 shows the proposed site layout for this option. The equalization storage basin has the same characteristics as the one in option 1A except that it will have sheet pile walls instead of earthen dikes.

Raw Sewage Pump Station

As with the first option, a new raw sewage pump station would need to be built at the existing WWTP site. Three 1.5-MGD solids handling centrifugal pumps would be provided to convey up to 3.0 MGD of raw sewage with one pump out of service.

Variable frequency drives (VFDs) would be provided to minimize the total number of pumps required to handle the moderate range of flows and to provide for flexibility. The pumps would be installed adjacent to the existing aeration basins. The finished floor elevation of the pump station would be 3 feet above the 100-year flood elevation and standby power facilities would be provided to ensure full operation of the new pumps during power outages. For wet weather operation, the existing plant would be upgraded as discussed previously.

INSERT FIGURE VII-6 OPTION 1B SCHEMATIC

INSERT FIGURE VII-7 OPTION 1B SITE PLAN

This option would be very difficult to implement because the Chehalis plant would be idle most of the summer, but would need to be ready to receive flow before the fall rains come. During the biological start up period, there would be nowhere to discharge the treated effluent except to the raw sewage pump station that goes to the new Centralia plant. It would also be very difficult to operate the Chehalis WWTP during dry weather periods that occur in long, cold stretches in the middle of winter. The cold temperatures would make it extremely difficult to re-establish the biomass after a cold weather shut down. Other disadvantages include draining all of the tanks for the summer, lack of convenient methods to exercise pumps in the summer, and it would be very labor intensive to maintain two plants.

ALTERNATIVE 2- MOVE THE DRY WEATHER OUTFALL LOCATION DOWNSTREAM OF THE SKOOKUMCHUCK RIVER

This option entails moving the dry weather discharge location of the plant to a point downstream of the Skookumchuck River. It is likely that a common outfall would be shared with the proposed Centralia plant to be located near the Galvin Road Bridge. A single 16-inch force main would transport treated effluent to the downstream outfall location. The force main route and length would be the same as the option for raw sewage pumping.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.

- Effluent limitations: Per the new NPDES permit for wet weather and dry weather conditions. Effluent limits for silver, zinc and copper are to be established through further water quality testing and analysis.
- Maximum average daily flow rate is 2.5 MGD for river flows less than 200 CFS and 3.0 for river flows between 200 and 1,000 CFS which requires 6.0 MG of equalization storage.
- Reliability: Class II

New Outfall

A new outfall would be required to discharge the treated effluent downstream of the Skookumchuck River during dry weather conditions. While preparing the Centralia Facilities Plan, CH2M Hill did an extensive evaluation of potential outfall sites below the Skookumchuck River. There is a large and long rock shelf near the confluence that is not suitable for a new outfall. The best site for a new outfall as determined by the City of Centralia is below the Galvin Road Bridge as shown in Figure VII-8. Constructing a new outfall is very expensive due to rigorous environmental restrictions. In some cases, it can be very difficult and expensive to go through the permitting process. It is recommended that the two cities work together and share the new outfall to minimize the burden on each city. Although the outfall would be common, there would be separate sample locations for each city's effluent for monitoring purposes. Chehalis would only use this outfall during dry weather conditions. If a separate outfall is needed, Chehalis would have to use it periodically during wet weather to ensure it is free of debris.

INSERT FIGURE VII-8 NEW OUTFALL LOCATION

Force Main

A single force main would be required to transport treated effluent to the new outfall location during dry weather conditions. The force main would be 16 inches in diameter and would follow the same route as option 1A to the site of the new Centralia plant. Options were considered to use one of Centralia's raw sewage force mains during dry weather, but eliminated due to Consent Decree flow based trigger delays.

Equalization Storage

This option would require the construction of a 6.0 MG equalization storage basin because the dry weather discharge is limited by the Consent Decree. As with option 1B, the equalization storage basin would be located just east of the aeration basins because the aeration basins would be kept in service. The equalization storage basin would be similar to option 1B which has a sheet pile wall instead of an earthen dike. Figure VII-9 shows a schematic diagram of this option and Figure VII-10 shows the proposed site layout for this option.

INSERT FIGURE VII-9 OPTION 2 SCHEMATIC

INSERT FIGURE VII-10 OPTION 2 SITE PLAN

Effluent Pump Station

A new effluent pump station would be required to pump the dry weather treated effluent to the new outfall. Three 1.5-MGD vertical turbine pumps would be provided to convey up to 3.0 MGD of treated effluent with one pump out of service. Each of the pumps would be rated for 1,050 GPM at 150 feet TDH and would be powered by a 50 HP motor. Daily discharge at the downstream outfall is limited to 2.5 MGD for river flows less than 200 CFS, and 3.0 MGD for river flows between 200 and 1,000 cfs. Variable frequency drives (VFDs) would be provided for flexibility. The pumps would be installed in the existing Chlorine Contact Chamber No. 2, which has adequate operational storage and contact time for dry weather diurnal variations. The pump motors would be 3 feet above the 100-year flood elevation and standby power facilities would be provided to ensure full operation of the new pumps (as well as the rest of the WWTP) during power outages.

The treated effluent would be disinfected using the existing chlorination system. Dechlorination prior to discharge into the Chehalis River downstream of the Skookumchuck River would be accomplished at the Centralia WWTP site using sulfur dioxide. The sulfur dioxide would be introduced into the effluent stream using a static mixer located near the outfall location. Just after dechlorination, the flow would go into a gravity transition manhole where the pressure head would be broken and the flow would go by gravity into the outfall diffuser. Figure VII-11 shows a schematic of the disinfection process.

A building would be required at the new Centralia WWTP site to house the sulfur dioxide storage and feed equipment, as well as, a sampler and refrigerator. The building would need to be approximately 200 square feet and equipped with emergency power from the Centralia WWTP. Since the treated effluent would not be exposed to the atmosphere, odor control is not provided. It is recommended that the outfall line

INSERT FIGURE VII-11 OPTION 2 DISINFECTION SCHEMATIC downstream of the sulfur dioxide injection point be made of HDPE to avoid potential corrosion concerns that may arise from periodic sulfide release.

It is anticipated that six 150-pound cylinders of sulfur dioxide would be required for approximately two to four weeks of operation. The total amount of stored chemical would be 900 pounds. The current storage threshold where more stringent regulations apply for storage and containment of potential leaks is 1,000 pounds. Only two bottles would be on line at any given time.

ALTERNATIVE 3- RIVER ENHANCEMENT AND DISCHARGE AT CURRENT LOCATION ALL YEAR LONG

This option entails continuing to discharge at the current outfall location year-round in conjunction with enhancing the dissolved oxygen levels within the Centralia Reach. River enhancement would be designed to achieve dissolved oxygen levels above the water quality standards for this stretch of the River at all times. The standards require that the DO level be 8.0 mg/l from October 1 to May 31 and 5.0 mg/l from June 1 to September 31 as discussed in Section III. It is therefore assumed that if there are no dissolved oxygen deficits, there would be no dry weather flow restrictions with this option. However, it is anticipated that full secondary treatment and ammonia discharge limits will be required.

<u>Design Criteria:</u>

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per a new revised NPDES permit for dry and wet weather conditions. It is assumed that mass limitations for the dry weather conditions may be the same or less stringent than established in the Consent Decree due to compliance with Class A water quality standards, but would apply to the current outfall location, not downstream of the Skookumchuck River. It is assumed that there would be no flow limitations under this option as long as mass limitations are met.
- Effluent limits for silver, zinc and copper to be established through further water quality testing and analysis.
- Maximum wet weather hydraulic capacity of the upgraded existing plant is 12.0 MGD, which requires 2.0 MG of equalization storage (for direct comparison with other options).

- Reliability: Class II

Equalization Storage

Under this option, equalization storage will be required for wet weather conditions to store the inflows in excess of 12.0 MGD. The required equalization basin volume is 2.0 MG. The equalization storage basin for this option would need to be constructed east of the existing aeration basins since the aeration basin would be kept in service during wet weather periods. Because of the small volume required, the equalization storage basin would be constructed using an earthen dike. Figure VII-12 shows a schematic diagram of this option and Figure VII-13 shows the proposed site layout for this option. The equalization storage basin has the same characteristics as the one in option 1A, except that it is smaller.

River Enhancement

The river enhancement goal is to maintain Class A water quality standards within the portion of the Centralia Reach that is impacted by the Chehalis WWTP discharge throughout the year. Additional study is needed to determine the limits of downstream influence on water quality by the WWTP. For practical implementation, compliance and enforcement of river enhancement goals should be based on conservative treatment techniques for specific flow events and calendar-based periods determined from the TMDL Study. Direct water quality sampling, however, should be utilized over the long term to measure success of the enhancement program and identify specific adjustments that are needed to improve operation of the system and further enhance water quality in the River.

INSERT FIGURE VII-12 OPTION 3 SCHEMATIC

INSERT FIGURE VII-13 OPTION 3 SITE PLAN

Analysis of the TMDL data was conducted to identify the relative DO deficit at each monitoring location from River Mile (RM) 75.1 at the old Riverside Bridge to RM 67.5

at the Mellen Street Bridge. Analysis of the data shows that the majority of water quality standard exceedances occurred during the late summer and early fall in many sections of the reach. During the period from June 1 through September 15, water quality standards were met at all sample locations to a depth of at least two meters over the entire reach and at most locations to a depth of four meters. All of the exceedances during the period occurred at lower depths in seven deep pools identified by DOE.

Based on the TMDL data, it appears to be feasible to increase DO to levels above specific water quality standards for the entire section of the reach. The general method to accomplish this goal is to raise the DO initially at about RM 75.1 to saturation levels and subsequently raise the DO to saturation levels at locations downstream as the DO falls to just above water quality standards. The DOE model was used to identify the initial oxygen input locations and the locations for subsequent inputs required downstream during different time periods and associated river flows. Results of this modeling show that oxygen inputs of approximately 800 lbs/day will be required at three locations along the Centralia Reach to achieve the water quality standards year-round.

The proposed aeration method is coarse bubble diffusion. With expected efficiency rates of 50%, each aeration facility will have to provide 1,600 lbs of oxygen per day. Due to estimated river depth, positive displacement blowers would be provided. The oxygen would be added to the river at the bottom of deep pools that are presently stratified. This would also serve to mix the river and destratify it where provided.

OUTFALL DIFFUSER

The existing outfall diffuser is a single port diffuser that is sometimes exposed during low river flows. Since this option calls for discharge at or near the current outfall site during dry weather conditions, it is recommended that the diffuser be replaced with a new, deeper diffuser. The current diffuser should only be replaced if it is used during dry weather conditions. A detailed field investigation should be conducted to determine the best location within the river to construct the new outfall.

ALTERNATIVES WITH WASTEWATER REUSE FOR DRY WEATHER FLOWS

The main thrust of the TMDL is to remove the point source discharge of the Chehalis WWTP to a location downstream of the Skookumchuck River where the Chehalis River has higher flows, a higher dissolved oxygen content and more assimilative capacity. Reuse options can eliminate any surface water discharge during dry weather conditions all together. So, instead of discharging to the River downstream, the treated effluent would be beneficially reused. The following options were developed as ways to prevent the necessity of any surface water discharge to the Chehalis River during dry weather conditions. The added benefit from these options is the independence from future (potentially more stringent) water quality standards.

ALTERNATIVE 4A - CLASS "A" RECLAIMED WATER (RW) TO GROUNDWATER RECHARGE VIA UNDERGROUND INFILTRATION GALLERIES AND/OR POPLAR IRRIGATION WITH GROUNDWATER RECHARGE IN THE SPRING AND FALL

This group of options entails producing Class A Reclaimed Water (RW) in the dry season that would be used for either groundwater recharge via underground infiltration galleries or poplar irrigation during the summer and groundwater recharge beneath the poplar stand during the spring and fall when the irrigation demand is not as high. Class A RW can also be used for irrigation of public golf courses, public parks, or other similar uses.

In order to produce Class A water suitable for reuse it is necessary to assure that the water would be safe for potential direct human contact. Treatment to achieve Class A RW standards typically requires coagulation/flocculation, filtration and very thorough disinfection to reduce pathogens to a safe level. Pathogen levels are estimated utilizing the concentration of coliform bacteria as the pathogen indicator. Disinfection must be adequate to assure that the total coliform bacteria levels are less than 2.2/100 ml for a seven-day average, with no samples exceeding 23/100 ml. Direct filtration (no flocculation step) is typically applied in wastewater tertiary treatment. The coagulation can be accomplished by adding polymer, alum or ferric salts to the secondary effluent prior to filtration. An in-line static mixer would be used to ensure thorough coagulant mixing with the secondary effluent. The coagulation step serves to agglomerate dissolved particles that are then filtered out in a sand filter. The reliability requirements are also more stringent for reuse because there is typically a greater chance to affect the health of the general public by direct contact with the RW.

Coagulation: Since the effluent from the biological treatment facilities is expected to be of a relatively high quality, coagulation is not expected to require large dosages of chemicals in order to aid in further clarification of the effluent. Based on experience at other sites and with other types of treatment, aluminum sulfate (alum) is the most economical coagulant available in this area. Alum dosages in the range of 20-30 mg/L are expected to be necessary to treat this biological effluent.

Since this alum dosage can consume up to 15 mg/L (as CaC03) of alkalinity, alkalinity control in the biological effluent will be a necessary part of the design and operation of the facilities. The extended aeration treatment will deplete alkalinity during nitrification and would require at least partial denitrification to recover some of the alkalinity. Otherwise deficiencies in alkalinity can limit adequate alum reaction.

Capability for alkalinity addition would be incorporated into the design to assure that nitrification, and effluent pH, is not jeopardized due to insufficient alkalinity availability. Potentially, up to 200 mg/L of alkalinity (as CaCO3) can be consumed during nitrification. Approximately 50% of this amount will be recovered during denitrification. From 15 to 30 mg/L of alkalinity is expected to be required for reaction with the alum used for chemical treatment. Alkalinity may be deficient after coagulation, which would require the inclusion of means for alkalinity addition (pH adjustment) in the design. This would most likely be done with lime addition facilities used for nitrification purposes.

For lower TSS levels, an in-line static mixer can be used to ensure complete mixing of coagulants prior to direct filtration.

The coagulation chemical addition facilities should be flow-paced to allow proper dosing of alum to all of the effluent from the biological treatment system and to maintain optimum coagulation.

Effluent filtration to achieve Class A water reuse standards would be accomplished with granular media filters. Filter systems come in numerous configurations including:

- Conventional gravity granular media filters.
- Conventional pressure granular media filters.
- Traveling bridge automatic backwash granular media filters.
- Continuous backwash filters.
- Rotating disc woven filters.

Each of these types of filters has been used for various water treatment applications and most of them have been utilized for advanced wastewater treatment, or effluent polishing. The effluent from the filter system is expected to meet the Class A Reuse requirements by reducing the biological treated (oxidized) effluent suspended solids to 5 mg/l or less and reducing turbidity to meet the definition for "Filtered Wastewater" of average 2 nephelometric turbidity unit (NTU) and not to exceed maximum of 5 NTU. Actual performance of the filters should reduce turbidity to below 0.5 NTU through optimizing coagulation processes. For comparison, drinking water filtration standards require 0.5 NTU 95% of the time. The filters would be operated continuously at a rate to match the biological treatment plant average flow rate, using the equalization storage basin to equalize the flows and dampen diurnal variations that can affect filter performance.

- Conventional gravity filters: These filters include settling processes and typically are configured in multiple units to allow taking individual units out of service for backwash on a regular basis. They would require a source of stored backwash water for the backwash. Backwash controls would be automated to conduct the backwashing sequence including rinsing of the filter following backwash. The backwash water would be discharged back into the treatment system for treatment and filtration again to produce a reclaimed effluent for reuse.

- Direct gravity pressure filters: Filters operation is similar to the conventional gravity granular media filters except settling and flocculation is not provided. Pressure filters require pressure head by pumping. Direct filters also require an external source of water for backwashing and again the backwash water would be discharged to the treatment facility for retreatment and polishing as reclaimed water.

- Traveling bridge automatic backwash filters: These filters are constructed in many small cells and are normally operated as direct gravity granular media filters. A traveling bridge backwashes each cell individually and utilizes effluent from the other cells for the backwash water. This alleviates the necessity for stored water for backwash of the filters on the treatment plant site. The backwash water is discharged to the treatment system, but the rate of production of the backwash water is much lower than the conventional gravity or pressure filters, consequently reducing the variable loading onto the primary and secondary treatment facilities. From a technological standpoint the traveling bridge automatic backwash granular media filter appears to be a superior application in this case compared to the conventional gravity filtration or direct pressure filtration due to the reduction in backwash water and backwash effluent.

- Continuous backwash filters: These filters are designed so that they operate continuously, while a pump, such as an air lift pump, continuously removes the dirty granular media from the bottom of the filter, discharges it into a media washing system and returns the clean media to the top of the filter system. The biological effluent following coagulation is fed into the filter from the bottom so that the suspended solids in the effluent are retained in the lower level granular media. The backwash water is provided by the filters on a continuous basis and the backwash water carrying the suspended solids is discharged back to the treatment system. Once again this system appears technologically superior according to the initial analysis due to the reduced requirements for storage of effluent for backwash water and the elimination of short-term variations in loading on the other facilities from the backwash water. These units have an advantage of operational and mechanical simplicity relative to traveling bridge filters.

The preliminary recommendation for filtration is a continuous backwashing sand filter (Dynasand). The filters would be sized for a flow rate of 3.5 MGD during the summer. The design loading rate is 4 -5 gpm/sf. Preliminary selection calls for 12 filter modules (6 cells with 2 modules per cell) with an area of 50 sf each, making for a total filter area of 600 sf. With one cell out of service to meet redundancy requirements, the remaining filter area is 500 sf.

An added advantage of the filters is that they can be used in the wet season to filter clarified effluent to further reduce TSS. The effluent would not be Class A because the throughput of the filters would be higher than the rate used to produce Class A reclaimed water. Wet season operation is based on a loading rate of 7 gpm/sf with all cells in operation at a flow rate of 6 MGD. Flows in excess of 6 MGD would not be filtered.

In addition to coagulation/flocculation and filtering, the options with a groundwater recharge component require nitrification and denitrification. This step is required to assure that when the reclaimed water reaches the groundwater table it has a nitrate concentration of less than 10 mg/l to meet drinking water standards. This can be accomplished by either providing a new anoxic basin between the primary clarifiers and the aeration basins or by using the Dynasand filters for denitrification with the addition of methanol as a carbon source. For this analysis it is assumed that denitrification would be accomplished by using methanol in conjunction with the sand filters. It should also be noted that additional nitrogen removal would be accomplished by soil aquifer treatment (SAT).

Soil Aquifer Treatment (SAT):

Soil aquifer treatment occurs when wastewater effluent that is applied directly to permeable soil at rates that are greater than the evaporative rate, moves through the vadose zone and ultimately to the groundwater aquifer. Considerable improvement in water quality may be obtained by movement of the wastewater through the soil, unsaturated zone, and aquifer (EPA Guidelines for Water Reuse, 1992). SAT is effective in removing the following constituents in the water: Suspended Solids (SS), nitrogen, ammonia, nitrate, phosphorus, BOD_5, COD, phosphate, fecal coliforms, viruses and heavy metals (see Table VII-2). SAT occurs throughout the entire soil profile until the reclaimed water reaches the groundwater. Removal processes are both

aerobic and anaerobic. SAT is being considered on this project for polishing reclaimed water for groundwater recharge via infiltration as it passes through the infiltration galleries and in the native soils beneath the infiltration galleries.

INSERT TABLE VII-2 SOIL AQUIFER TREATMENT

Reliability

The reliability requirements are stricter for reuse water options than for options that continue to discharge into the Chehalis River. This is due to the potential to impact public health through direct exposure to the reclaimed water or the potential to contaminate groundwater stores. In order to meet reliability requirements for the treatment systems, the following provisions would be made:

- Alarms - extensive alarm systems would be installed to alert and/or notify operators of loss of function of various systems, including power, aeration, coagulation, disinfection and pumping.

- Process monitors - provisions would be installed for continuous monitoring of the biological treatment process, effluent turbidity, chlorine residual (already in place), flows and other process parameters and alarms will be installed to notify operators of deviations from proper parameter ranges.

- Redundancy - the biological treatment process would be constructed with more than one basin to allow one to be out of service while retaining the capability of producing oxidized wastewater (nitrification, denitrification and/or biological phosphorus removal may be less efficient than normal during the operation with one treatment basin out of service); coagulant chemical feed system would be provided with standby feeder capability; chlorine and sulfur dioxide feed systems would have standby capability; chlorine contact tank would be constructed with parallel basins operation; effluent filters would be constructed with capability to take one unit out to allow one to be removed from service while retaining the capability for continued of service while maintaining capability for operation of the remaining units; pumping systems will be designed for full capacity with the largest pump out of service.

Standby power system upgrade. In order to provide Class I reliability for the wastewater treatment system, it is necessary to provide standby power sufficient to operate the plant at or near capacity for dry weather conditions under the most adverse conditions of process units being out of service. Consequently the existing standby power system would have to be upgraded to operate both the secondary and advanced treatment trains in the event of power outage. The current standby power system is rated at 90 kW. Since the future load is expected to be greater using the extended aeration system, it would be necessary to upgrade the standby power system. This would also provide adequate standby power for pumping the treated effluent to the receiving areas for the reclaimed water.

Equalization Storage

Equalization storage would be required for all of the reuse options because the peak daily flow during dry weather conditions is 6.2 MGD (2025) which is considerably higher than the average flow rate. Figure VII-14 shows a graph of required equalization storage volume for various capacities of reuse facilities. This analysis will be based on an equalization storage volume of 4.0 MGD that will require a reuse facility flow capacity of 3.5 MGD. Appendix E contains a discussion of the model that was used to create the sizing graph. As with all of the other options, the equalization storage would be used for raw sewage only.

Reclaimed Water Pump Station

A new reclaimed water pump station would be required to pump the reclaimed water to the end use facility. Three 1.75-MGD vertical turbine pumps would be provided to convey up to 3.5 MGD of reclaimed water with one pump out of service. Variable frequency drives (VFDs) would be provided for flexibility. The pumps would be installed in existing Chlorine Contact Chamber No. 2. The chlorine contact chamber would provide adequate contact time and operational storage for diurnal variations at lower flows.

INSERT FIGURE VII-14 EQUALIZATION VOLUME REQUIREMENTS FOR REUSE ALTERNATIVES

The equalizing basin would provide storage for flows greater than 3.5 MGD and for large diurnal variations. The pump motors would be 3 feet above the 100-year flood elevation and standby power facilities would be provided to ensure full operation of the new pumps (as well as the rest of the WWTP) during power outages.

Reclaimed Water Force Main

A reclaimed water force main would also be required to transport the reclaimed water to the end use facility. The force main size is dependent on distance to each disposal facility. The length of the force main would depend upon the end-use selected and the Facility Plan level siting analysis. An assumed force main size and length will be presented for each specific end use alternative.

Disinfection

All of the reuse options would use chlorination for disinfection. The reuse standards require a chlorine residual of 0.5 mg/l at the furthest application point in the reclaimed water system. This residual is required to prevent the buildup of a slime layer inside the pipe. The chlorine dosage required for fecal coliform inactivation and to maintain a 0.5 mg/l residual will have to be determined once the system is operational. This report will assume that an initial chlorine dosage of 10 mg/l would be used for reuse options. With all of the reuse options, there is no need for dechlorination. However, the dechlorination system would still be used for wet weather flows when the effluent is discharged at the existing river outfall. UV light is also suitable for some of the reuse options.

ALTERNATIVE 4Ai: CLASS A RW FOR GROUNDWATER RECHARGE VIA INFILTRATION

For this project to succeed there must be a beneficial consumptive use for the reclaimed water that satisfies the following requirements:

- Can accept 100 % of the reclaimed water during dry weather conditions;
- Is not weather dependent;
- Is close to the plant;
- Will not have a negative impact on groundwater or surface water quality; and
- Has regulatory and public acceptance.

Groundwater recharge is a beneficial, non-consumptive use that would fulfill these requirements. It will serve as a stopgap method of utilizing 100% of the reclaimed water during periods of low river flow, even during freezing weather. It can also easily be used in conjunction with other reuse options such as golf course and park irrigation or cooling water for a potential power plant.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES permit for wet weather conditions. Limits for silver, zinc and copper to be established through further water quality testing and analysis.
- Class A reclaimed water standards for groundwater recharge:
  -BOD_5 and TSS does not exceed 30 mg/l
  -Coagulated
  -Filtered to 2 NTU monthly average and 5 NTU daily maximum
  -Disinfected to achieve less than 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.
  -Denitrification such that nitrate concentration is less than 10 mg/l before the reclaimed water reaches the groundwater.
  -Meets drinking water quality standards shown in 43.20 RCW and 70.119A RCW which includes metals.
  -Maximum average daily flow rate is 2.8 MGD that requires 6.0 MG of equalization storage.

There are two alternatives to infiltrate reclaimed water into the ground. The first method is to use a rapid infiltration basin and the second is to use an underground infiltration gallery similar to an on-site sewage disposal drain field.

A rapid infiltration basin is an engineered basin that is used to infiltrate large quantities of water into the ground very quickly. The following excerpt from EPA Guidelines for Water Reuse describes rapid infiltration basins and how they work.

Infiltration basins are the most widely used method of groundwater recharge. Basins afford high loading rates and relatively low maintenance and land requirements. Basins consist of bermed, flat-bottomed areas of varying sizes. Long, narrow basins built on land contours have been effectively used. Basins are constructed on highly permeable soils to achieve high hydraulic rates are called rapid infiltration basins.

Rapid infiltration basins require permeable soil for high hydraulic loading rates, yet the soil must be fine enough to provide sufficient soil surfaces for biochemical and microbiological reactions, which provide additional treatment to the reclaimed water. Some of the best soils are in the sandy loam, loamy sand, and fine sand range.

When the reclaimed water is applied over to the spreading basin, the water percolates through the unsaturated zone to the saturated zone of the groundwater table. The hydraulic loading rate is preliminarily estimated by soil studies, but final evaluation is done by operating in situ test pits or ponds. Hydraulic loading rates for rapid infiltration basins vary from 65 to 500 ft./yr. (20 to 150 m)/yr., but are usually less than 300 ft./yr. (90 m)/yr. (Bouwer, 1988).

Though management techniques are site specific and vary accordingly, some common principles are practiced in most systems. A wetting and drying cycle with periodic cleaning of the bottom is used to prevent clogging by accumulated suspended solids, maintain a high rate of infiltration, maintain microbial populations to consume organic matter and help reduce levels of microbiological constituents in the reclaimed water, and promote nitrification and denitrification processes for nitrogen removal. The loading rates are usually higher when nitrogen removal is not a concern.

Spreading grounds can be managed to avoid nuisance conditions such as algae growth and insect breeding in the percolation ponds. Generally, a number of basins are rotated through filling, draining, and drying cycles. Cycle length is dependent on both soil conditions and the distance to the groundwater table and is determined on a case-by-case basis from field-testing. Algae can clog the bottom of basins and reduce infiltration rates. Algae further aggravate soil clogging by removing carbon dioxide, which raises the pH, causing precipitation of calcium carbonate. Reducing the detention time of the reclaimed water within the basins minimizes algae growth. Also, scarifying, rototilling or disking the soil following the drying cycle can help alleviate clogging potential, although scraping or "shaving" the bottom to remove the clogging layer is more effective than disking it.

Rapid infiltration basins are designed so that soil particle sizes are smallest at the surface and get larger with depth. This is done so that any clogging and creation of a surface skin (bio-mat) would occur at the surface where it can be maintained. Otherwise, the infiltration basin may clog well below the surface where it would be difficult, if not impossible, to restore infiltration capacity. A cross-section of a typical basin is shown in Figure VII-15. A sandy loam, loamy sand or fine sand works best for the upper strata (top 3 feet) of a rapid infiltration basin (Bouwer, 1985). The Chehalis area has only one soil type that is well suited for rapid infiltration. This soil type is Newberg and is located exclusively adjacent to the Chehalis and Newaukum rivers. Design water depth in a rapid infiltration basin is usually kept to a maximum of one foot to minimize the growth of algae and prevent soil compaction (Bouwer, 1985).

An underground infiltration gallery can also be used to recharge the groundwater. It would consist of a distribution pipe network in a gravel bed (see Figure VII-16). The infiltration gallery would be constructed using a one-foot layer of drain rock above a one-foot layer of washed sand. Filter fabric would not be used because it is prone to clogging over time.

INSERT FIGURE VII-15 RAPID INFILTRATION

BASIN TYPICAL CROSS SECTION

INSERT FIGURE VII-16 UNDERGROUND INFILTRATION GALLERY TYPICAL CROSS SECTION

The pressure distribution pipe network would consist of laterals spaced evenly at ten feet on center and designed for a residual head of ten feet at the end of the laterals.

Since the infiltration gallery is underground, there would not be any visual impact of the infiltration facility. As with rapid infiltration basins, infiltration galleries would have to be constructed in multiples so that each can be rested while another unit is active. The major advantage of underground infiltration galleries in this application is that they are protected from floods, even though they are right next to the river and will be flooded.

The use of rapid infiltration basins was ruled out because they would be very difficult to protect from floods. Each time a flood occurs, the basins would be clogged with silt and other debris that would have to be removed prior to using them. This would be done using a bulldozer or scraper and would be very time consuming and expensive. There is also no way to assure that the basins can be restored after a flooding event in time to be used in case the river flow dropped down and dry weather limits take effect. Therefore, the recommended method of groundwater recharge for this option is to use underground infiltration galleries that are under a foot of native soil and protected from floods.

The underground infiltration galleries would have to be located in areas of Newberg soil type (SCS No. 148) because they are the only soils in the Chehalis area that have adequate permeability to promote infiltration. The Newberg soil has a clean water permeability of 6 - 20 in/hr at depths from 17-inches to 60-inches. To calculate allowable RW loading rates, a conservative estimate of 10% of the low end of the clean water permeability is used. The design loading rate is 1.2-feet per day (6 in./hr * 10% * 24 hr/day ÷ 12 in/ft). Since there is only limited acreage in the vicinity of the treatment plant with this soil type (45 acres) it was necessary to increase the size of the equalization storage basin in order to reduce the amount of reclaimed water that must be discharged to the infiltration galleries. This option will assume a flow rate of 2.8 MGD must be handled and the required storage volume is 6.0 MG. A soils map of the area is included in Appendix E.

Based on the loading rate above, an area of 7.2 acres of infiltration galleries is required for a flow rate of 2.8 MGD. However, this amount must be tripled to allow for adequate drying times of the infiltration galleries to promote nitrification and denitrification. The design infiltration gallery area is 21.5 acres.

Potential sites for infiltration galleries are located just south of the treatment plant site adjacent to the Chehalis River. Multiple infiltration galleries would be constructed to allow adequate drying times between applications. To provide for flexibility it is recommended that 22 one acre infiltration galleries be constructed. Each one acre basin will receive 400,000 GPD at a flow rate of 2.8 MGD. The submain will be 6" PVC and the distribution laterals will be 2" PVC with holes drilled at 4" on center. The laterals would be spaced at 10' on center. The drain rock would be 1-1/2 inch washed rock similar to that used in septic tank drain field construction.

Equalization Storage

Under this option, 6.0 MG of equalization storage would be required to keep the infiltration gallery size to a minimum since there is limited suitable soil type near the WWTP site. The equalization storage basin for this option would need to be constructed east of the existing aeration basins since they will be kept in service. Figure VII-17 shows a schematic diagram of this option and Figure VII-18 shows the proposed site layout for this option. The equalization storage basin has the same characteristics as the one in option 1A except that it would have sheet pile walls instead of earthen dikes. The location of the sand filter that is required for Class A reuse is also shown on this figure.

INSERT FIGURE VII-17 OPTION 4Ai SCHEMATIC

INSERT FIGURE VII-18 OPTION 4Ai SITE PLAN

This option would also be able to operate during dry weather conditions during the winter if necessary. When there is an extended freeze in the area, river flows drop because there is no runoff source anymore. Under this condition, the ground surface can stay frozen for several weeks at a time. However, since the infiltration distribution pipes are at least one foot below the ground surface, they would not freeze. This is a very important consideration since each viable option must have a place to discharge the reclaimed water when dry weather limits apply, regardless of weather conditions.

Reclaimed Water Pump Station

This option would require a 2.8 MGD reclaimed water pump station to convey the water to the infiltration galleries. The pump station would consist of three 1.4 MGD vertical turbine pumps that would be installed in Chlorine Contact Tank No. 2. Three pumps are provided for redundancy. All pumps would be equipped with VFDs.

Reclaimed Water Force Main

The reclaimed water would be sent to the infiltration galleries in a 12" force main.

ALTERNATIVE 4Aii: CLASS A RW FOR POPLAR IRRIGATION WITH GROUNDWATER RECHARGE IN SPRING AND FALL

Class A RW can also be used for poplar irrigation in conjunction with groundwater recharge. The Class A RW would be applied to a poplar tree stand on all days when dry weather limits apply. In the spring and fall the application rate can be higher than the hydraulic agronomic rate required which would result in groundwater recharge beneath the poplar stand. However, the reclaimed water will be applied at the agronomic rate for nitrogen uptake to assure protection of the groundwater. This option uses Class A RW to allow for groundwater recharge, even though poplar irrigation by itself only requires Class D RW. The premise behind this option is to treat the effluent to Class A reuse standards so that the reclaimed water can be applied to the poplar trees even when they are dormant or when it is raining. Since the reclaimed water is applied at greater than the hydraulic agronomic rate, it would percolate into the ground as groundwater recharge. Staff at DOH Division of Drinking Water, as well as the USEPA Ecosystems Office encourages development of this type of innovative solution. DOE also encourages reuse as a way to comply with the TMDL requirements.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES permit for wet weather conditions. Limits for silver, zinc and copper to be established through further water quality testing and analysis.
- Class A reclaimed water standards for groundwater recharge:

  -BOD_5 and TSS does not exceed 30 mg/l

  -Coagulated

  -Filtered to 2 NTU monthly average and 5 NTU daily maximum

  -Disinfected to less than 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.

  -Denitrification such that nitrate concentration is less than 10 mg/l before the reclaimed water reaches the groundwater.

  -Meets drinking water quality standards shown in 43.20 RCW and 70.119A RCW, which includes metals.

  -Maximum average daily flow rate is 3.5 MGD that requires 4.0 MG of equalization storage.

The following is a listing of the three predominant soil types in the areas (See Appendix E) along with their listed infiltration rates:

SCS Soil Type 48, Chehalis, silty clay loam, 0.6-2 in/hr

SCS Soil Type 148, Newberg, fine sandy loam, 2-6 in/hr

SCS Soil Type 172, Reed, silty clay loam, 0.6-2 in/hr

The following assumptions are used to estimate the amount of land required for poplar irrigation based on nitrogen uptake by the trees:

1. Nitrogen uptake rate = 250 lbs/ac/year of nitrogen as N

2. Existing effluent flow during dry weather conditions is estimated based on the 90th percentile of the total dry weather flow for the last seven years and is 235 MG. The design effluent flow for the year 2025 is 375 MG which is based on the proportional increase in equivalent population.
3. Effluent total nitrogen as N = 10 mg/l which requires both nitrification and denitrification.

The land area required for poplar irrigation is 125 acres net (375 MG * 10 mg/l *8.34 lbs/gal ÷ 250 lbs/acre/yr N). The land area should be at least 150 acres to leave room for roads and perimeter setbacks. There is an adequate supply of appropriate farmland within two miles of the WWTP. Figure VII-19 is an aerial photograph of the area that shows where the desirable soils are located based on the SCS Lewis County Soil Survey. Most of the best soils are located across the Chehalis River on both sides of State Route 6 and next to the Newaukum River. The City plans to purchase property for the tree farm and plant trees prior to building the new plant so that the trees will be established and have a higher nitrogen demand when the City must comply with the TMDL in 2008. Expected time to harvest is 7 – 10 years.

Even though this option does not specifically require poplars to be grown, they will be provided for the following reasons:

- They take up large amounts of nitrogen
- They take up BOD, phosphorus and metals
- They provide shade canopy
- They are a valuable resource

The water demand for the poplar trees is estimated to be 0.25 acre-inch/day over a 150 day growing season which equates to 1 million gallons per acre per growing season.

INSERT FIGURE VII-19 POTENTIAL POPLAR TREE FARM SITES

Bud break is typically the first of April and the leaves stay on the trees into November. With trees planted on 125 acres of land, a total of 125 MG of reclaimed water would be taken up by the trees. That leaves approximately 250 MG (375 – 125) of reclaimed water that would recharge the groundwater for future conditions.

Equalization Storage

Under this option, equalization storage is not specifically required but is provided in order to keep the filter capacity and reclaimed water pump station at a maximum capacity of 3.5 MGD. The equalization storage basin would also be used for inflows in excess of 12.0 MGD during the winter. The equalization storage basin for this option would be 4.0 MG and would need to be constructed east of the existing aeration basins
since they would be kept in service. The equalization storage basin would be constructed using an earthen dike. Figure VII-20 shows a schematic diagram of this option and Figure VII-21 shows the proposed site layout for this option. The location of
the sand filter that is required for Class A reuse is also shown on this figure. The equalization storage basin has the same characteristics as the one in option 1A.

This option may be difficult to implement because dry weather limits apply during cold spells in the deep of winter when river flow drops below 1,000 cfs. The poplars would be dormant and the ground beneath the poplars may be frozen which precludes groundwater recharge via infiltration. Historical temperature data from a reporting station at Centralia show that there are up to 15 days where the daytime high temperature is less than 32 degrees. Therefore, an alternative end use would need to be found for this option for up to 15 days during freezing weather conditions when dry weather limits apply. Underground sprinklers could be used, but they would be considerably more expensive and would get in the way during tree harvesting operations.

INSERT FIGURE VII-20 OPTION 4Aii SCHEMATIC

INSERT FIGURE VII-21 OPTION 4Aii SITE PLAN

One option to handle this situation is to provide storage for the reclaimed water during the freezing weather. It is estimated that 30 MG of storage would be required (2-MGD average dry weather flow x 15 days). This would be a 9.2-acre pond at a depth of ten feet. After the ground thaws out, the stored reclaimed water can then be used for groundwater recharge. A pump station at the tree farm would be required for this purpose.

Another option for handling this situation is to discharge the Class A reclaimed water directly to the Chehalis River at the current outfall location or near the Mellen Street Bridge when the dry weather limits apply during the periods of a deep freeze. When the water temperature in the Chehalis River is low, it can hold more dissolved oxygen. At a water temperature of 5°C, the saturated DO level is 12.8 mg/l which is considerably higher than the 8.0 mg/l water quality standard.

The TMDL did not evaluate cold weather conditions for high quality effluent discharge. At present, the Consent Decree specifically states that no discharge would be allowed at the current outfall location during dry weather conditions. A specific exclusion would have to be authorized by DOE to allow the reclaimed water to be discharged into the river during these cold spells. The assimilative capacity of the River should be more than adequate with the colder water temperature and lack of stratification in the deep pools.

The last option to have a discharge location for the reclaimed water during freezing conditions is to berm the poplar plantation. Under this scenario, a berm would be built around the entire plantation, which would allow ponding of the reclaimed water if the ground would not percolate. In order to store 30 mg of RW in a 150-acre plantation, the berm would need to be 0.6-feet tall.

Initial hydrogeological research indicates that a large portion of the soils in the vicinity of the WWTP site may have a clay layer beneath them. The clay layer would impede infiltration and would force the reclaimed water into a lateral direction. If the clay layer was shallow enough, it could lead to saturation of the upper soil zone resulting in standing water during reclaimed water application in excess of hydraulic agronomic rates. The presence of a clay layer beneath the top soil is much more likely for the Chehalis and Reed soil types than it is for the Newberg soils which have considerably more permeability.

Without site specific test pits or borings being dug, there is no way to tell how far down the clay layer is; or if it is there at all. If this option is to be implemented, additional site-specific soils information will need to be obtained.

This option will be based on constructing a perimeter berm to temporarily store the RW during freezing conditions.

Reclaimed Water Pump Station

This option would require a 3.5 MGD reclaimed water pump station to convey the water to the poplar tree stand. The pump station would consist of three 1.75 MGD vertical turbine pumps that would be installed in Chlorine Contact Tank No. 2. Three pumps are provided for redundancy. All pumps would be equipped with VFDs.

Reclaimed Water Force Main

The reclaimed water would be sent to the poplar tree stand in a 12-inch force main.

ALTERNATIVE 4B - CLASS B RECLAIMED WATER (RW) FOR POPLAR OR PASTURE LAND IRRIGATION WITH A STORAGE POND; OR DISCHARGE TO NATURAL OR CONSTRUCTED BENEFICIAL USE WETLANDS

This group of options entails producing Class B RW in the dry season that would be used for poplar or pastureland irrigation in conjunction with a storage pond, or discharge to natural or constructed beneficial use wetlands.

Class B reuse water has the same requirements as Class A, except that advanced filtration is not required. Class B reuse water is essentially a high quality secondary effluent that has undergone thorough disinfection. It should be noted that with Class B reclaimed water no body contact with the water is allowed. Also, for recharge of both natural and constructed beneficial use wetlands, nitrogen and phosphorus removal is required.

ALTERNATIVE 4Bi - CLASS B RW TO POPLAR IRRIGATION WITH STORAGE POND FOR SPRING AND FALL

This option entails producing Class B RW in the dry season that would be used for poplar irrigation during the summer and for filling a storage pond during the spring and fall when the trees do not need water. This option is similar to Alternative 4Aii, except that when the trees do not need the water, it would be diverted to a storage pond instead of being used for groundwater recharge. This option requires that storage be provided for the reclaimed water when the trees are dormant and do not need water but the river flows are down and dry weather limits apply. When the river flows have come back up and the wet weather limits apply, the pond contents would be metered back to the river. The reclaimed water that is stored in the spring can be used for irrigation instead of being pumped back to the river. The Class B RW should also undergo nutrient removal for nitrogen and phosphorus to avoid algae problems in the pond.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES permit for wet weather conditions. Limits for silver, zinc and copper to be established through future water quality testing and analysis.
- Class B reclaimed water standards for non-food crop irrigation and restricted recreational impoundment:
  -BOD_5 and TSS concentration does not exceed 30 mg/l

-Disinfected to 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.

-Nutrient removal for phosphorus and nitrogen

-Maximum average daily flow rate is 3.5 MGD that requires 4.0 MG of equalization storage.

The following assumptions are used to estimate the amount of land required for poplar irrigation based on nitrogen uptake by the trees:

1. Nitrogen uptake rate=250 lbs/ac/year of Nitrogen as N
2. Existing effluent flow during dry weather conditions is estimated based on the 90^{th} percentile of the total dry weather flow for the last seven years and is 235 MG. The design effluent flow for the year 2025 is 375 MG which is based on the proportional increase in equivalent population.
3. Effluent total nitrogen as N= 10 mg/l which requires both nitrification and denitrification.

The required land area for poplar irrigation is 125 acres net (375 MG * 10 mg/l * 8.34 lbs/gal ÷ 250 lbs/acre/year N). The land area should be at least 150 acres to leave room for roads and perimeter setbacks.

Since the pond level would fluctuate significantly during the year, it would not be suitable for a fish pond or as a park water feature. The following assumptions were used to estimate the size of the storage pond:

1. The poplars would not need water when precipitation exceeds evapotranspiration or when they are dormant starting October 1st.
2. Storage would need to be provided for all days when precipitation exceeds evapotranspiration and for all days when freezing weather precludes irrigation.

3. The pond level would be allowed to decrease through evaporation during the late summer to leave room for RW in the early fall when the trees approach dormancy, but dry weather limits still apply.
4. The required storage pond volume is governed by the drought period of 1976-77 when dry weather limits continued to apply from October 1, 1976 to February 28, 1977 except for one week at Christmas. The storage pond volume is estimated by adding up the number of days that dry weather limits would have applied from October 1 through February 28 except for seven days at Christmas
and multiplying that number times the average daily dry weather future flow. The calculation is shown below:

   Storage volume = (151 days – 7 days) x 1.9 MGD = 274 MG. This makes for a pond 84 acres at a depth of ten feet. In addition, the pond needs to be sized to include room for rainwater that would fall during that period. Figure VII-22 shows the schematic diagram for this option and Figure VII-23 shows the site plan for this option. This option was eliminated from further consideration because of the vast size of the pond required for storage.

INSERT FIGURE VII-22 OPTION 4Bi SCHEMATIC DIAGRAM

INSERT FIGURE VII-23 OPTION 4Bi SITE PLAN

ALTERNATIVE 4Bii - CLASS B RW FOR NATURAL WETLAND RECHARGE

This option entails producing Class B RW in the dry season that would be used to recharge natural wetlands during the summer. Nutrient removal (nitrogen and phosphorus) is also required to avoid algae blooms and to minimize the impact on the natural wetland habitat. With Class B RW, no direct human contact with the wetland water would be allowed. The reuse standards have very strict limits for nitrogen and phosphorus which are 3 mg/l TKN and 1 mg/l P.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.

Effluent limitations: Per the new NPDES permit for wet weather conditions. Limits for silver, zinc and copper to be established through further water quality testing and analysis.

- Class B reclaimed water standards for wetland recharge without body contact:

  -BOD_5 concentration does not exceed 20 mg/l and loading is less than 5 Kg/ha/d on an annual average basis.

  -TSS concentration does not exceed 20 mg/l and loading is less than 9 Kg/ha/d on an average annual basis.

  -Disinfected to 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.

  -TKN concentration is less than 3 mg/l and loading is less than 1.2 Kg/ha/d on an annual average basis.

  -Total phosphorus concentration is less than 1 mg/l and loading is less than 0.2 Kg/ha/d on an annual average basis.

  -Ammonia concentration is less than the chronic toxicity standards in WAC 173-201A-040(3).

  -Metals concentrations are less than the surface water quality standards in WAC 173-201A.

  -Maximum average daily flow rate is 3.5 MGD that requires 4.0 MG of equalization storage.

The following assumptions are used to estimate the amount of land required for natural wetland recharge:

1. Loading rate = 2 cm/day (0.8 in/day) annual average based on a Class II wetland
2. Existing effluent flow during dry weather conditions is estimated based on the 90th percentile of the total dry weather flow for the last seven years and is 235 MG. The design effluent flow for the year 2025 is 375 MG that is based on the proportional increase in equivalent population.
3. Length of application period is based on the 90th percentile of the number of days per year that dry weather limits would apply based on the last fifty years of data and is 231 days.
4. Allowable loading during dry weather = 231 days * 0.8 in/day ÷ 12 in/ft = 15.4 feet/year

The required land area for natural wetland recharge is 75 acres net (375,000,000 gal ÷ 7.48 gal/cf ÷ 15.4 ft/year / 43,560 sf/ac). The wetland area should be at least 100 acres to leave room for perimeter setbacks. There is an adequate supply of appropriate natural wetlands within three miles of the WWTP. There are some wetlands within a mile of the plant, but not 100 acres. Figure VII-24 shows a wetland map of Chehalis. Since this option does not have a surface water discharge to the Chehalis River during the dry season, there will not be any dry weather flow limitation. However, equalization storage is still required to keep the reclaimed water pump station sized at 3.5 MGD and to handle winter storm flows in excess of 12.0 MGD. The equalization storage basin would have a volume of 4.0 MG and would be constructed just east of the aeration basins using an earthen dike. Figure VII-25 shows the schematic diagram for this option and FigureVII-26 shows the site plan for this option.

INSERT FIGURE VII-24 WETLAND MAP

INSERT FIGURE VII-25 4Bii SCHEMATIC DIAGRAM

INSERT FIGURE VII-26 4Bii SITE PLAN

Reclaimed Water Pump Station

This option would require a 3.5 MGD reclaimed water pump station to convey the water to the natural wetland. The pump station would consist of three 1.75 MGD vertical turbine pumps that would be installed in Chlorine Contact Tank No. 2. Three pumps are provided for redundancy. All pumps would be equipped with VFDs.

Reclaimed Water Force Main

The reclaimed water would be sent to the natural wetland in a 12-inch force main. This option would be very difficult to implement for two reasons. The first is that the natural wetlands will have a discharge to creeks or streams that discharge into the Centralia Reach of the Chehalis River. This would violate the TMDL restriction on discharge of ammonia or BOD_5 to the Centralia Reach during the dry season. The second problem with this option is that there would be no need, or perceived benefit, from recharging natural wetlands when they are still full of water in the spring. Especially if the river flows dropped and the WWTP had to stop discharging to the River, yet there was a large rainstorm that filled up the wetland before the river flows came back up. The City must have an end use that is not weather dependent.

ALTERNATIVE 4Biii - CLASS B RW FOR CONSTRUCTED BENEFICIAL USE WETLAND

This option entails producing Class B RW in the dry season that would be used to supply water to a constructed beneficial use wetland during the summer. Nutrient removal (nitrogen and phosphorus) is also required to prevent algae blooms and to minimize the impact on the constructed wetland habitat. With Class B RW, no human contact with the wetland would be allowed. The reuse standards have very strict limits for nitrogen and phosphorus, which are 3 mg/l TKN and 1 mg/l P.

Design Criteria:

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES permit for wet weather conditions. Limits for silver, zinc and copper to be established through further water quality analysis and testing.

- Class B reclaimed water standards for constructed beneficial use wetland recharge without body contact:

  -BOD_5 concentration does not exceed 20 mg/l and loading is less than 5 Kg/ha/d on an annual average basis.

  -TSS concentration does not exceed 20 mg/l and loading is less than 9 Kg/ha/d on an average annual basis.

  -Disinfected to 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.

  -TKN concentration is less than 3 mg/l and loading is less than 1.2 Kg/ha/d on an annual average basis.

  -Total Phosphorus concentration is less than 1 mg/l and loading is less than 0.2 Kg/ha/d on an annual average basis.

  -Ammonia concentration is less than the chronic toxicity standards in WAC 173-201A-040(3).

  -Metals concentrations are less than the surface water quality standards in WAC 173-201A.

  -Maximum average daily flow rate is 3.5 MGD that requires 4.0 MG of equalization storage.

The following assumptions are used to estimate the amount of land required for a constructed beneficial use wetland:

1. Total hydraulic retention time (HRT) = 10 days
2. Design depth = 1.5 feet
3. Land area required based on maximum month average flow for dry weather; in 2025 this is 2.5 MGD

The required land area for a constructed beneficial use wetland is 50 acres net (2,500,000 gal/day * 10 days HRT ÷ 7.48 gal/cf ÷ 1.5 ft depth ÷ 43,560 SF/acre). The wetland area should be at least 60 acres to provide adequate detention time during prolonged periods of high flow. There is an adequate supply of appropriate land within a mile of the WWTP.

Since this option does not have a surface water discharge to the Chehalis River during the dry season, there would not be any dry weather flow limitation. However, equalization storage is still required to keep the reclaimed water pump station sized at 3.5 MGD and to handle winter storm flows in excess of 12.0 MGD. The equalization storage basin would have a volume of 4.0 MG and would be constructed just east of the aeration basins using an earthen dike. Figure VII-27 shows the schematic diagram for this option and FigureVII-28 shows the site plan for this option.

Reclaimed Water Pump Station

This option would require a 3.5 MGD reclaimed water pump station to convey the water to the natural wetland. The pump station would consist of three 1.75 MGD vertical turbine pumps that would be installed in Chlorine Contact Tank No. 2. Three pumps are provided for redundancy. All pumps would be equipped with VFDs.

Reclaimed Water Force Main

The reclaimed water would be sent to the natural wetland in a 12-inch force main.

This option would be very difficult to implement for two reasons. The first is that the constructed beneficial use wetland would require an emergency overflow that may discharge to creeks or streams that discharge into the Centralia Reach of the Chehalis River. This would violate the TMDL restriction not allowing any amount of ammonia or BOD_5 to enter the Centralia Reach during the dry season. There can also be a

INSERT FIGURE VII-27 4Biii SCHEMATIC DIAGRAM

INSERT FIGURE VII-28 4Biii SITE PLAN

surface water discharge from the wetland if there is heavy rainfall and no evaporation which causes the wetland water level to rise. The second problem with this option is that if the wetland was still full in the spring, there would be no place to discharge the Class B RW, especially if the river flows dropped and the WWTP had to stop discharging to the River, yet there was a large rainstorm that filled up the wetland before the river flows came back up. The City must have an end-use that is not weather dependent.

ALTERNATIVE 5 - STORE ALL DRY WEATHER EFFLUENT FLOWS AND DISCHARGE TO THE RIVER IN THE WINTER

This option entails storing all disinfected effluent from the WWTP during periods of low flow and then discharging it once wet weather limits apply. The storage lagoon will have to be located off-site due to space limitations. Based on eight years of flow data, a storage volume of 460 MG would be required to store all effluent flows in the year 2025 when dry weather limits apply. This volume also takes rainfall into account. Based on a depth of ten feet, an area of 140 acres would be required for the storage lagoon. This option was eliminated from further consideration due to the vast size of the storage pond that is required.

ALTERNATIVE 6: CLASS A RECLAIMED WATER FOR STREAMFLOW AUGMENTATION

The reuse standards also allow Class A RW to be used for the beneficial use of streamflow augmentation. This option entails producing a Class A RW that would be used to augment the flow in the Centralia Reach during dry weather conditions. During the summer months, flow in the Centralia Reach is low and the stream velocity is very slow. The lack of habitat from low flow and the utrofication from low velocities (stagnation) makes it difficult for fish to use the Reach as a migration corridor. Also, DOE has stated that the available water within the Reach has been over-appropriated. This means that there are more water rights issued than the available water. Therefore, any additional water that is added to the Reach has beneficial use for both fish and downstream water users.

Design Criteria

- Loadings for BOD_5, TSS and ammonia are shown in Section V of this report.
- Effluent limitations: Per the new NPDES Permit for wet weather conditions. Limits for silver, zinc and copper to be established through further water quality testing and analysis.
- Class A reclaimed water standards for streamflow augmentation:

-BOD_5 and TSS does not exceed 30 mg/l

-Coagulated

-Filtered to 2 NTU monthly average and 5 NTU daily maximum

-Disinfected to 2.2 total coliforms/100 ml weekly average and 23/100 ml daily maximum.

-Meets requirements in 90.48 RCW

- Ammonia concentration of 1 mg/l daily maximum

Equalization Storage

Under this option, equalization storage will be required for wet weather conditions to store inflows in excess of 12.0 MGD. The required equalization basin volume is 2.0 MG. The equalization storage basin for this option would need to be constructed east of the existing aeration basins since they would be kept in service. Because of the small volume required, the equalization storage basin would be constructed using an earthen dike. Figure VII-29 shows a schematic diagram for this option and Figure VII-30 shows the proposed site layout for this option. The equalization storage basin for this option has the same characteristics as the one in Option 1A, except that it is smaller.

INSERT FIGURE VII-29 SCHEMATIC DIAGRAM

INSERT FIGURE VII-30 SITE LAYOUT

Advanced Treatment

The advanced treatment for this option is the same for Alternative 4Ai and 4Aii, except that denitrification is not required. This option would also use Dynasand filters to produce an effluent with low turbidity. The ammonia and BOD_5 would be reduced to very low levels with the extended aeration process.

OUTFALL DIFFUSER

The Class A reclaimed water would be added to the Centralia Reach at the existing outfall location during dry weather conditions. The existing outfall would need to be replaced with a new, deeper diffuser. A detailed field investigation should be conducted to determine the best location to site the flow augmentation point.

TMDL

This option is based on the reuse standards that allow for stream flow augmentation. However, it must also comply with the TMDL. The TMDL, as written claims that there is no capacity in the Centralia Reach during dry weather conditions based on DOE's modeling results. In DOE's modeling, a 0.2 mg/l drop in DO is deemed acceptable for evaluating impacts to water quality. This 0.2 mg/l exception is the basis for allowing Darigold to continue to discharge into the River at flows greater than 500 cfs, but less than 1,000 cfs. Appendix A contains a letter report from Cosmopolitan Engineering that summarizes their modeling effort which shows beyond a doubt that there is capacity in the Centralia Reach for ammonia and BOD when river flow is correlated with expected discharge. Also, if Centralia does move their outfall location out of the Centralia Reach to a location downstream of the Skookumchuck River, there will indeed be ammonia and BOD capacity in the Centralia Reach that could be used by Chehalis. It is recommended that DOE and Chehalis work together to establish exactly how much capacity there really is in the Centralia Reach and under what conditions it can be allocated. The management and control of this option is more complex if loading capacities are tied to incremental river flows, but extremely feasible when environmental and cost benefits are considered.

PRELIMINARY EVALUATION

All of the options will be evaluated based on whether or not there are operational or regulatory factors that may prevent them from being implemented. The options that cannot be implemented will be eliminated from further consideration.

Option 1A is to pump all raw sewage to a regional plant and abandon the existing plant. This option does not have operational or regulatory issues that would prevent it from being implemented.

Option1B is to pump raw sewage to the new Centralia plant only during dry weather and use the upgraded existing plant for wet weather flows. This option would be very difficult to implement because it requires that the existing Chehalis plant be shut down during dry weather periods. When the raw sewage is being pumped to the Centralia plant, there will not be any food for the microorganisms in the aeration basins and they would die. It would take two to six weeks to re-establish the microorganism population for effective treatment after raw sewage is re-introduced to the basins. The start-up time is very dependent on temperature. Regardless of river flows, the plant must be started up by October 1st to assure that there is enough warmth to reestablish a microbiological population. However, dry weather discharge limits could be in effect until the end of the year. The only place to discharge the treated effluent during dry weather conditions is to the Centralia plant. Also, during prolonged cold spells in the winter when dry weather limits apply, the plant cannot be shut down at all.

The city is also very concerned with staffing issues related to this option. During the summer when the plant is shut down for months at a time, there is no need for an operations staff at the existing plant. It is unlikely that the City can retain qualified operators and lab technicians if seasonal layoffs occur.

For these reasons, the option for a “dry weather regional” plant will not be considered any further.

Option 2 is to discharge downstream of the Skookumchuck River during dry weather conditions and use the upgraded existing plant for wet weather conditions. This option is what the Consent Decree is predicated on and therefore does not have any operational or regulatory issues that will make it difficult to implement. However, there is a legal concern regarding potential compensation requirements or lawsuits from water right owners along the Chehalis River between the existing WWTP and downstream of the Skookumchuck River.

Option 3 is to enhance the River and continue to discharge at the current outfall location all year long with the existing upgraded plant. This option does not have any operational issues but there are several regulatory issues that must be resolved prior to implementation. They are as follows:

- The TMDL must be modified, and approved by EPA, to allocate a BOD_5 and ammonia loading during dry weather conditions predicated on enhancement of the river.
- The NPDES permit must be modified to allow effluent discharge at the current location during dry weather conditions and to establish a monitoring protocol for river enhancement.

- It must be determined whether or not there is adequate BOD_5 and ammonia capacity in the River for flows less than 1,000 cfs during the winter when water temperatures are low and DO is high.

With this option, the plant will continue to discharge to the River 365 days a year in conjunction with river enhancement for flows less than 1,000 cfs. The TMDL Study did not evaluate potential river quality impairment during the winter months. Since the saturation level of dissolved oxygen (DO) at a river temperature of 5° C is 12.8 mg/l there should be more than adequate capacity in the river to accept the Chehalis WWTP effluent without causing the DO level to drop below 8 mg/l which is the river water quality standard after September 15th.

This option does have some regulatory issues that must be resolved prior to implementation but not enough to eliminate it from further consideration. See Appendix A for pertinent correspondence with DOE concerning river enhancement.

Option 4Ai is to use Class A reclaimed water for groundwater recharge via underground infiltration galleries. This option does not have any operational issues but there is one regulatory issue that concerns the TMDL. The only suitable soils in the area that have substantial infiltration capabilities are the Newberg soil type that is located exclusively next to the Chehalis and Newaukum rivers. So, the infiltration galleries would have to be located adjacent to the Chehalis River where the groundwater may be hydraulically connected to the River. The reuse standards require that the reclaimed water meet drinking water standards at the point where the reclaimed water reaches the groundwater table. The secondary effluent would undergo nitrification and denitrification and then would be treated to Class A reuse standards. As the reclaimed water passes through the vadose zone there would be further reduction in ammonia and BOD_5 (see Table VII-1). This option also assumes that there will be no monitoring requirements at the infiltration areas for either BOD_5 or ammonia. All monitoring will be consistent with the Water Reclamation and Reuse Standards (September 1997) pertaining to groundwater recharge. It should also be noted that DOE has indicated that beneficial reuse of the effluent is a very high priority of the Department (see correspondence in Appendix A). This option has some regulatory issues that must be resolved prior to implementation but not enough to eliminate it from further consideration.

Option 4Aii is to use Class A reclaimed water for poplar tree irrigation in conjunction with groundwater recharge. This option does not have any operational issues but does have one regulatory issue that must be resolved prior to implementation. The regulatory issue with this option concerns the periods of deep freezes when dry weather discharge limits apply but the ground is frozen and cannot accept reclaimed water for groundwater recharge. This option calls for irrigating poplar trees during the growing season and continuing to apply reclaimed water when the poplars are dormant or during rainfall so that the reclaimed water percolates through the ground and is used for groundwater recharge. However, when the ground at the poplar tree farm is frozen, the reclaimed water cannot be applied since it may not infiltrate properly. The best way to handle this issue is to construct a berm around the poplar plantation to allow the reclaimed water to pond. When the temperature rises, the ground will thaw and allow the stored water to infiltrate. DOE has indicated that they would support this concept.

Option 4Bi is to use Class B reclaimed water for poplar tree irrigation in conjunction with a storage pond. The reclaimed water would be used for irrigation only at agronomic rates. When the precipitation exceeds evapotranspiration, the reclaimed water would be sent to a storage pond. This option has been eliminated from further consideration due to the enormous size of the storage pond that is required.

Option 4Bii is to use Class B reclaimed water for natural wetland recharge. This option can not be implemented because there is a strong possibility that the reclaimed water would commingle with the wetland water and then reach the Centralia reach through streams or creeks. Even though the reclaimed water has undergone thorough treatment and would likely promote assimilation of natural BOD_5 and ammonia in the wetland, there would still be BOD_5 and ammonia in it. This is consistent with overall TMDL goals of reducing net loading, but is in direct conflict with the TMDL Study recommendations for no BOD_5 or ammonia going into the river during low river flows. In addition, during the spring the wetlands would already be full and there will not be any benefit from recharging them with reclaimed water. This option was therefore eliminated from further consideration.

Option 4Bii is to use Class B reclaimed water for recharge of beneficial constructed use wetlands. This option has the same regulatory and operational issues as the natural wetland recharge option and was also eliminated from further consideration.

Option 5 is to store all of the WWTP effluent flows when dry weather limits apply. This option was eliminated from further consideration due to the vast size of the storage pond that would be required.

Option 6 is to use Class A RW for stream flow augmentation in the Centralia Reach. This option does not have any operational issues, but there are several regulatory issues that must be resolved prior to implementation. They are as follows:

- The TMDL must be modified and approved by EPA to allocate a BOD_5 and ammonia loading during dry weather conditions predicated on meaningful modeling results.
- The NPDES permit must be modified to allow reclaimed water to be added to the river with an appropriate mass limit for BOD_5 and ammonia.
- The critical balance between model parameters, fish benefits and water resource laws must be acknowledged.

Although this option has some regulatory issues that must be resolved prior to implementation, they are not enough to eliminate it from further consideration.

CONCLUSION

Several of the options have been eliminated from further consideration based on operational or regulatory issues that will make them very difficult to implement. There are still numerous options remaining that will be evaluated in further detail and are listed below:

1A Regional WWTP

2 Dry weather discharge below the Skookumchuck River

3 River enhancement with continued discharge at the current outfall location all year long

4Ai Class A reclaimed water for groundwater recharge via underground infiltration galleries

4Aii Class A reclaimed water for poplar irrigation in conjunction with groundwater recharge beneath the poplars

6 Class A reclaimed water for stream flow augmentation

DETAILED ALTERNATIVE EVALUATION

The remaining alternatives will now be evaluated in greater detail based on the following criteria:

- Low capital and O&M cost
- Increases probability of grant funding
- Level of treatment required
- Maintains effluent within the same stream units it originated in
- Provides beneficial reuse of the effluent
- Will maintain or improve water quality and fish habitat or migration routes
- Is not affected by future surface water quality restrictions
- Is able to be easily and reliably implemented under current regulations
- Ease of operation

CAPITAL COSTS

The cost estimates presented in this report are planning level cost estimates, which are considered to be "order-of-magnitude" only. The expected accuracy of this type of estimate is plus 50% to minus 30% of the estimated cost shown. Cost estimates have been prepared for each alternative and include the following:

1. An opinion of probable construction costs for a competitively bid public works project. Constructions cost amounts are based on cost data from similar projects. Information provided by manufacturer's representatives is also used for estimating costs of treatment units.
2. An allowance of construction contingencies at 30%. This allowance is intended to provide for additions to the project scope during the design process, unknown subsurface conditions (such as large boulders, groundwater, hazardous waste, etc.), and construction change orders.
3. Washington State sales tax at the City of Chehalis rate of 7.7%.
4. An allowance for permits, engineering, administration, and legal costs during pre-design, design and construction at 35%. Costs for preparation of construction contract documents, engineering and inspection services during construction, administrative and legal fees and for obtaining required permits are all included in this allowance.
5. All costs are in 1999 dollars.
6. The cost estimates shown are for the complete option including treatment and end use and are based on the assumption that the existing plant would be upgraded as described previously in this section. The costs are shown for the following categories:

    - Amount required to meet the NPDES permit
    - Amount required for other capital improvements
    - Amount required for operational enhancement

The amounts shown required to meet the NPDES permit are for upgrades that must be made to the plant to meet the new NPDES permit conditions, as well as, the cost required for the end use of the wastewater.

The capital improvement costs shown are required to upgrade the plant so that it is able to operate reliably throughout the planning period. Most of the costs under this category are for modifications to the solids train and disinfection system.

Modifications shown under the operating enhancement category include items that would allow the plant to operate more easily and cost-effectively, but are not required to meet the NPDES permit or to maintain the plant's reliability over the planning period.

PRESENT WORTH OF ALTERNATIVES

The present worth of capital costs and additional O&M costs is included with the cost estimate tables in this section to identify the relative costs of all of the viable alternatives over the next 20 year period. Present worth was also evaluated over a 40-year period and the results did not vary significantly with regard to relative cost difference. Present worth analysis spreadsheets for 20 and 40 year planning periods are included in Appendix E.

The analysis of present worth for capital costs distinguishes between the various life cycles and salvage values for electrical equipment, mechanical equipment, real property (land) and structural components. For this analysis, life cycles for electrical, mechanical and structural components are 15, 20 and 50 years respectively. A sensitivity analysis conducted by utilizing various life cycles inputs shows that there is little effect on the comparative costs if life cycles are varied by plus or minus 5 years. A percentage of each cost component (i.e. electrical, mechanical, etc.) is estimated from the individual cost estimates and all of the cost component percentages were adjusted relative to other alternatives.

Additional O&M costs identified in the analysis are electrical power, chemicals and employees (FTEs). The analysis of the present worth of additional O&M costs considers the additional O&M cost for each alternative relative to the other alternatives. For each O&M component considered, the alternative or alternatives with the least O&M cost is set as the base level of O&M (zero) for that specific O&M component. Actual O&M for some of the alternatives may be less than current O&M costs. Therefore, it is important to distinguish between the actual additional O&M addressed in Section VIII (Financial Considerations) and the additional O&M relative to each alternative shown in this Section of the report.

The analysis utilizes a moderate rate of return of 5% and an inflation estimate of 3% per year. Typically inflation is not used in present worth analysis of O&M costs per federal guidelines, but is used in this analysis to account for the reality of escalating costs in wastewater treatment. The rate of return (discount rate) is selected as a moderate range of realistic returns on investment. A sensitivity analysis of the relative effects of different inflation and rates of return shows that there can be a significant difference in the total present worth of all projects with different rates, but not a substantial difference in the relative present worth of each alternative when compared to the other alternatives.

OPTION 1A is to pump all raw sewage to a new regional plant to be located in Centralia and abandon the existing plant. Table VII-3 shows the cost summary for this option which is based on a forcemain to Site Alternative 1. The forcemain cost to the revised location of the proposed Centralia plant is considerably more than presented herein due to the increased length. Chehalis' share of the estimated capital cost of this option is $45.7 million, the estimated present worth of additional O&M cost relative to other alternatives is $ 0 and the total present worth cost is $ 37.5 million. Detailed cost estimates and present worth analysis are included in Appendix E. The high cost of this option is due to the need for a large raw sewage pump station, seven-mile dual force main and capital facility charge for the new regional plant. The Chehalis portion (capital facilities charge) of the regional plant cost was estimated based on the proportion of flow that Chehalis would contribute to a regional facility. The cost of a regional facility was estimated by increasing the cost estimate that was presented in the Centralia Facilities Plan. By adding the Chehalis flow, the Centralia Plant needs to be 80% larger to be a regional facility. A 20% cost reduction was then applied to the regional cost estimate to reflect an "economy of scale" reduction factor. The calculation for the capital facility charge is shown below:

Assumptions:

- Chehalis 2025 annual average flow= 3.2 MGD
- Centralia 2025 annual average flow= 4.0 MGD
- Centralia WWTP cost= $37,046,000 (from CH2M Facilities Plan)
  Regional Plant Cost= $37,046,000* (4.0+3.2)/4.0* 80%= $53,346,000
  Chehalis share= 3.2/7.2* $53,346,000= $23,709,000

It is likely that a regional facility would increase the probability of significant grant funding. However, this option is considerably more expensive than the other options. There is also very limited grant funding available. For instance, last year DOE only had approximately $10 million in grant funds available for the entire state.

TABLE VII-3
OPTION 1 COST ESTIMATE REGIONAL PLANT

ITEM	AMOUNT REQUIRED TO MEET PERMIT	AMOUNT FOR CAPITAL IMPROVEMENTS	AMOUNT FOR OPERATIONAL ENHANCEMENT	TOTAL
Equalization Storage (6MG)	$508,000	0	0	$508,000
Raw Sewage Pump Sta. And Dual Force main	$9,088,000	0	0	$9,088,000
Demolish Existing Plant	0	0	$1,500,000	$1,500,000
SUBTOTAL	**$9,596,000**	**0**	**$1,500,000**	**$11,096,000**
Mobilization @ 5% Subtotal	$480,000 $10,076,000	0 0	$75,000 $1,575,000	$555,000 $11,651,000
Construction Contingency @ 30% Subtotal	$3,023,000 $13,099,000	0 0	$473,000 $2,048,000	$3,496,000 $15,147,000
Sales Tax @ 7.7% Subtotal	$1,009,000 $14,108,000	0 0	$158,000 $2,206,000	$1,167,000 $16,314,000
Engineering, Admin. & Legal @ 35%	$4,938,000	0	$772,000	$5,710,000
Total Capital Cost	**$19,046,000**	**0**	**$2,978,000**	**$22,024,000**
Capital Facility Charge for Regional Plant (Chehalis share)	$23,709,000	0	0	$23,709,000
Total Project Cost	**$42,755,000**	**0**	**$2,978,000**	**$45,733,000**
Present Worth of O & M Cost (Relative)				0
Total Estimated Present Worth				$37,505,000

This option only requires treatment to a high quality secondary effluent with complete nitrification during the dry weather conditions and partial nitrification during the wet weather conditions. This is easily achievable and does not require any advanced treatment.

This option does not keep the effluent in the same tributary basin as it originated. The potable water for Chehalis is withdrawn from the Newaukum and Chehalis Rivers upstream of the WWTP. The treated effluent would be discharged below the confluence with the Skookumchuck River that is approximately seven miles downstream. It is very important for the water to remain in the same tributary basin to maintain the water supply balance and avoid water right issues. This is a very high priority with DOE.

This option does not provide beneficial use of the effluent. The effluent is still discharged to surface water and the discharge point is downstream of the confluence with the Skookumchuck River where the Chehalis River flows are higher than they are near the WWTP site. The surface water in the Chehalis basin is already over-allocated and it would be very helpful to have the effluent used in a beneficial manner as far upstream as practical.

This option does not significantly improve water quality or fish habitat and migration routes. The treatment plant effluent would be moved seven miles downstream where the dilution is greater and the river has a higher dissolved oxygen (DO) level. But this would not significantly improve the water quality in the Centralia Reach to the point where it would meet water quality standards. DOE's modeling results in the TMDL Study show that removing all of the point source discharges from the Centralia Reach would not significantly increase the DO level in the River, and in some isolated river segments would actually lower the DO level (TMDL Study Appendix I, Tables 1.8-1.12). By moving the effluent discharge point out of the Centralia Reach, there would be less water for fish to utilize in migration through the reach.

This option requires continued discharge to surface waters of the State and is therefore affected by any potential future water quality restrictions. The current TMDL Study in this stretch of the river is for DO and ammonia, but DOE could also perform TMDLs for pH, temperature, fecal coliforms, metals, etc. at some point in the future. Any of these TMDLs could force additional effluent limitations on the City's discharge in the future. In addition, with the potential to have certain salmon species put on the threatened or endangered species list, there could also be further water quality restrictions to ensure the survival of the listed fish species. At present, there is no indication that there would be further restrictions on surface water discharge. But, as time goes on, the environmental regulations continue to get stricter and the point source dischargers are the easiest to regulate and therefore take the brunt of the impacts.

The only way to avoid these potential future water quality restrictions is to repair the environment or to remove the discharge from the river by implementing land application or wastewater reuse.

This option can be easily and reliably implemented under current regulations. The Consent Decree and the new NPDES permit are written around a discharge downstream during the critical dry weather conditions. There are no TMDL or NPDES permit issues that need to be resolved with DOE prior to implementation. This option does have several governance issues with regard to regional ownership and operation that would need to be resolved prior to implementation.

This option is relatively easy to operate. There would be a large raw sewage pump station and equalization storage basin at the existing site that would require daily attention by the City of Chehalis or regional operations staff. It is anticipated that a regional operations staff would be assembled that would maintain and operate the new regional facility.

There has been a lot of discussion with the Chehalis/Centralia area concerning the regional plant concept. There is widespread belief among the public that a regional plant would cost less and lead to lower sewer rates for both communities. However, the cost of the long forcemain from the existing Chehalis WWTP to the proposed regional WWTP site is so great that it cannot be offset by an "economy of scale" savings for a regional facility.

In order to establish whether or not a regional plant is indeed less expensive than individual plants, the elected officials from both cities directed their respective engineers to work together to prepare a regional WWTP cost evaluation. During the summer of 1999 CH2M Hill (Centralia) and Gibbs & Olson (Chehalis) met to review each other's design criteria, design assumptions, and cost estimates. After agreeing to each other's basic design criteria for each of the plants, cost criteria were evaluated and agreed to and an evaluation was made relative to the operation and maintenance cost savings that may result from a regional plant. The evaluation showed clearly that a regional facility is substantially more expensive than separate plants. The main reason that a regional facility is not cost effective is the long Chehalis forcemain and the requirement for primary clarifiers at the regional facilities (ie: primary clarifiers are not required for Centralia alone). So instead of an economy of scale savings with a regional facility, it actually would cost more than individual plants on a per gallon basis. The regional WWTP cost memorandum is included in Appendix E. A detailed forcemain cost analysis is included in Appendix E.

OPTION 2 is to upgrade the existing plant and discharge downstream of the Skookumchuck River during dry weather conditions. Table VII-4 shows the cost summary for this option. The estimated capital cost of this option is $25.4 million, the present worth of estimated additional O&M cost relative to other alternatives is $3.2 million and the total present worth cost is $26.4 million. A detailed cost estimate is included in Appendix E. This option will probably not increase the potential for grant funding over any of the other options since it does not incorporate wastewater reuse or a regional facility.

Most of the alternative evaluation discussion for option 1A also applies to this option since they both have a dry weather discharge point downstream of the Skookumchuck River. This option requires only secondary treatment with complete nitrification during dry weather and partial nitrification during wet weather. It does not make for beneficial reuse of the effluent and does not improve water quality or fish habitat or migration routes. This option is also affected by any future restrictions on surface water quality. As with Option 1A, this option can be easily and reliably implemented under current regulations. This option would be slightly more difficult to operate

TABLE VII-4 OPTION 2 COST ESTIMATE DISCHARGE D/S OF THE SKOOKUMCHUCK RIVER				
Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
Upgrade Existing Plant				
1. Two New Secondary Clarifiers	$995,000	$40,000	$0	$1,035,000
2. Rehabilitate Primary Clarifiers	$0	$75,000	$300,000	$375,000
3. Headworks Improvements	$120,000	$10,000	$10,000	$140,000
4. Aeration Basin Improvements	$475,600	$0	$0	$475,600
5. Solids Train Improvements	$0	$1,180,000	$0	$1,180,000
6. Equalization Storage Basin (6 MG)	$1,366,000	$0	$0	$1,366,000
7. Flood Protection Dike	$1,528,000	$0	$0	$1,528,000
8. Disinfection System Upgrade	$0	$275,000	$0	$275,000
9. Yard Piping Upgrades	$400,000	$100,000	$0	$500,000
10. Electrical System Upgrades	$400,000	$100,000	$50,000	$550,000
11. Instrumentation & Control System Upgrades	$200,000	$50,000	$50,000	$300,000
12. pH Adjustment	$200,000	$0	$0	$200,000
13. Miscellaneous Plant Upgrades	$0	$30,000	$450,000	$480,000
End-Use Facility Force Main, Pump Station and New Outfall Downstream of the Skookumchuck River	$4,335,000	$0	$0	$4,335,000
SUBTOTAL	**$10,020,000**	**$1,860,000**	**$860,000**	**$12,740,000**
Mobilization @ 5%	$501,000	$93,000	$43,000	$637,000
Subtotal	$10,521,000	$1,953,000	$903,000	$13,377,000
Construction Contingency @ 30%	$3,156,000	$586,000	$271,000	$4,013,000
Subtotal	$13,677,000	$2,539,000	$1,174,000	$17,390,000
Sales Tax @ 7.7%	$1,053,000	$196,000	$90,000	$1,339,000
Subtotal	$14,730,000	$2,735,000	$1,264,000	$18,729,000
Engineering, Admin. & Legal @ 35%	$5,156,000	$957,000	$442,000	$6,555,000
Purchase Properties East of Site	$150,000	$0	$0	$150,000
Total Capital Cost	**$20,036,000**	**$3,692,000**	**$1,706,000**	**$25,434,000**
Present Worth O & M Cost (Relative)				$3,245,000
Total Estimated Present Worth				$26,397,000

relative to the existing plant. There would be a large equalization storage basin and an effluent pump station that would require attention. But, there is not an advanced treatment train (such as filtration) that must be operated.

OPTION 3 is to provide enhancement of the River by adding oxygen to it and continue discharging all year long at the current outfall location. Table VII-5 shows the cost summary for this option. The estimated capital cost of this option is $19.4 million, the present worth of estimated additional O&M cost relative to other alternatives is $2.9

TABLE VII-5 OPTION 3 COST ESTIMATE RIVER ENHANCEMENT				
Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
Upgrade Existing Plant				
1. Two New Secondary Clarifiers	$995,000	$40,000	$0	$1,035,000
2. Rehabilitate Primary Clarifiers	$0	$75,000	$300,000	$375,000
3. Headworks Improvements	$120,000	$10,000	$10,000	$140,000
4. Aeration Basin Improvements	$476,000	$0	$0	$476,000
5. Solids Train Improvements	$0	$1,180,000	$0	$1,180,000
6. Equalization Storage Basin (3 MG)	$630,000	$0	$0	$630,000
7. Flood Protection Dike	$1,528,000	$0	$0	$1,528,000
8. UV Disinfection	$0	$500,000	$0	$500,000
9. Yard Piping Upgrades	$400,000	$100,000	$0	$500,000
10. Electrical System Upgrades	$400,000	$100,000	$50,000	$550,000
11. Instrumentation & Control System Upgrades	$200,000	$50,000	$50,000	$300,000
12. pH Adjustment	$200,000	$0	$0	$200,000
13. Miscellaneous Plant Upgrades	$0	$30,000	$450,000	$480,000
End-Use Facility				
New Outfall/Diffuser and Aeration Facilities	$1,700,000	$0	$0	$1,700,000
SUBTOTAL	**$6,649,000**	**$2,085,000**	**$860,000**	**$9,593,200**
Mobilization @ 5%	$332,000	$104,000	$43,000	$479,660
Subtotal	$6,981,000	$2,189,000	$903,000	$10,073,000
Construction Contingency @ 30%	$2,094,000	$657,000	$271,000	$3,021,860
Subtotal	$9,074,000	$2,846,000	$1,174,000	$13,094,000
Sales Tax @ 7.7%	$699,000	$219,000	$90,000	$1,008,300
Subtotal	$9,774,000	$3,065,000	$1,264,000	$14,103,000
Engineering, Admin. & Legal @ 35%	$3,421,000	$1,073,000	$442,000	$4,936,000
Land and Right-of-Way for Aeration Facilities	$200,000	$0	$0	$200,000
Purchase Properties East of Site	$150,000	$0	$0	$150,000
Total Capital Cost	**$13,545,000**	**$4,138,000**	**$1,706,000**	**$19,389,000**
Present Worth of O & M Cost (Relative)				$2,878,000
Total Estimated Present Worth				$20,540,000

million and the total present worth cost is $20.5 million. A detailed cost estimate is included in Appendix E. This option may have a higher probability of receiving grant funding because it is innovative and will allow water quality standards to be met.

This option requires the same level of treatment as option 1A and 2 which is a high quality secondary effluent with complete dry weather nitrification and partial wet weather nitrification. This option would continue to discharge at the current outfall location all year long which keeps the effluent in the same sub-basin as it originated. This helps to keep a balanced water cycle in the Chehalis area.

This option does not make beneficial reuse of the effluent, but the effluent is discharged into the Centralia Reach, which needs all the water it can get during the summer. This is the only option that provides a significant improvement to the water quality in the Centralia Reach. This option would assure that the Class A water quality standard of 8.0 mg/l can be met all year long by adding oxygen to the River in the places where it is needed. The water quality would be improved beyond the seasonal limit of 5.0 mg/l to 8.0 mg/l DO all of the time. Even by removing all of the point source discharges from the Centralia Reach and assuming a dramatic reduction in non-point source pollution, the TMDL model shows that the water quality standards would still not be met. And since DOE has limited control over non-point source pollution, it is very doubtful that the predicted reduction from non-point sources would ever take place. In short, implementing all of the non-point source reduction recommendations is not likely in this planning and therefore water quality in the Centralia Reach would not attain water quality standards. Conversely, this option will actually substantially enhance the River and guarantee that the Class A water quality standard of 8.0 mg/l DO can be met 365 days a year. Increasing the DO in the River would also improve the water quality for the benefit of native fish and wildlife.

Since this option calls for continued discharge to surface waters of the State it is affected by potential future water quality restrictions just as the previous options are. This option may be difficult to implement since it is a pioneering approach to solving water quality problems. This is the first proposed river enhancement project in the State. There are no published guidelines or standard design criteria already established which means that there would be numerous issues that must be resolved with DOE and EPA prior to implementation. However, since this option will allow water quality goals to be met without seasonal variance, it is worth the effort to pursue all the required approvals.

This option entails special operational duties because it requires operation of aeration facilities that would be located off-site. However, the blowers and diffusers would not be much different than the ones typically used in wastewater treatment. The main operational advantage of this option is that there is no equalization storage, raw sewage or effluent pump stations, or long force mains that must be maintained. In addition, it would probably be necessary to develop and perform pilot testing of different aeration techniques.

OPTION 4Ai

This option uses Class A reclaimed water for groundwater recharge via underground infiltration galleries. Table VII-6 shows the cost summary for this option. The estimated capital cost of this option is $26.7 million, the present worth of the estimated additional O&M cost relative to other alternatives is $4.3 million and the total present worth cost is $28.4 million. A detailed cost estimate is included in Appendix E. This option would have a greater chance of receiving grant funding than the options utilizing a downstream discharge because it is based on wastewater reuse. The Legislature and DOE have placed an emphasis on funding reuse projects the past few years and it is expected that reuse projects will continue to have funding priority in the future. This option calls for beneficial recharge of much needed groundwater stores, which is a high priority of DOE.

TABLE VII-6
OPTION 4Ai COST ESTIMATE
GROUNDWATER RECHARGE VIA UNDERGROUND INFILTRATION GALLERY

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
Upgrade Existing Plant				
1. Two New Secondary Clarifiers	$995,100	$40,000	$0	$1,035,100
2. Rehabilitate Primary Clarifiers	$0	$75,000	$300,000	$375,000
3. Headworks Improvements	$120,000	$10,000	$10,000	$140,000
4. Aeration Basin Improvements	$475,600	$0	$0	$475,600
5. Solids Train Improvements	$0	$1,180,000	$0	$1,180,000
6. Equalization Storage Basin (6 MG)	$1,366,000	$0	$0	$1,366,000
7. Flood Protection Dike	$1,528,000	$0	$0	$1,528,000
8. Disinfection System Upgrade	$0	$275,000	$0	$275,000
9. Yard Piping Upgrades	$400,000	$100,000	$0	$500,000
10. Electrical System Upgrades	$400,000	$100,000	$50,000	$550,000
11. Instrumentation & Control System Upgrades	$200,000	$50,000	$50,000	$300,000
12. pH Adjustment	$200,000	$0	$0	$200,000
13. Miscellaneous Plant Upgrades	$0	$30,000	$450,000	$480,000
14. Advanced Treatment	$1,374,000	$0	$0	$1,374,000
End-Use Facility				
Pump Station, Force Main and Underground	$3,534,000	$0	$0	$3,534,000

Infiltration Gallery				
SUBTOTAL	**$10,593,000**	**$1,860,000**	**$860,000**	**$13,313,000**
Mobilization @ 5% Subtotal	$530,000 $11,123,000	$93,000 $1,953,000	$43,000 $903,000	$666,000 $13,979,000
Construction Contingency @ 30% Subtotal	$3,337,000 $14,460,000	$586,000 $2,539,000	$271,000 $1,174,000	$4,194,000 $18,173,000
Sales Tax @ 7.7% Subtotal	$1,113,000 $15,573,000	$196,000 $2,735,000	$90,000 $1,264,000	$1,399,000 $19,572,000
Engineering, Admin. & Legal @ 35%	$5,451,000	$957,000	$442,000	$6,850,000
Purchase Properties East of Site	$150,000	$0	$0	$150,000
Land for Infiltration Gallery	$125,000	$0	$0	$125,000
Total Capital Cost	**$21,299,000**	**$3,692,000**	**$1,706,000**	**$26,697,000**
Present Worth O & M Cost (Relative)				$4,296,000
Total Estimated Present Worth				$28,368,000

This option requires a very high level of treatment to produce Class A reclaimed water. The reclaimed water must meet drinking water standards by the time it reaches the groundwater table. The secondary effluent must undergo both nitrification and denitrification to reduce nitrate concentrations to less than 10 mg/l. Class A reclaimed water requires advanced treatment consisting of coagulation, flocculation and filtering.

This option would recharge the groundwater in the same sub-basin as it was originally withdrawn from. The reclaimed water would be used for groundwater recharge at a location just south of the treatment plant site, which is just downstream of a City of Chehalis potable water intake.

This option makes for beneficial reuse of the effluent by using it to recharge the groundwater. The water rights in this basin have been over-allocated and any addition to the groundwater stores is of great benefit. Although this option does not directly reduce potable water demand through a consumptive use of the reclaimed water, it does create the basic infrastructure needed for consumptive use and replenishes the water source for beneficial downstream uses.

This option provides a marginal improvement to the water quality in the Chehalis River by removing all effluent discharged during low flow conditions. Although the pollution contribution of the effluent is very small, it still has some affect on water quality. This option can have both positive and negative affects on fish habitat and migration routes depending on actual groundwater to surface water dynamics.

Since this option will preclude all surface water discharge during the critical dry weather periods, it is not likely to be affected by potential future restrictions on surface water quality. This is a major advantage given the increased environmental regulations during the past few years that have placed severe restrictions on surface water discharge. There may still be additional regulations that affect wet weather discharge, but it is less likely.

This option requires nitrification and denitrification, as well as advanced treatment to produce drinking water quality effluent. This level of treatment would require additional operation time and experience. There is also more testing required for this option because it has the potential to contaminate groundwater stores in the area. There would need to be monitoring wells installed in and around the infiltration area to sample and test groundwater quality.

OPTION 4Aii is to use Class A reclaimed water for poplar irrigation in conjunction with groundwater recharge. Table VII-7 shows the cost summary for this option. The estimated capital cost of this option is $22.9 million, the estimated present worth additional O&M cost relative to other alternatives is $4.3 million and the total present worth cost is $24.4 million. A detailed cost estimate is included in Appendix E. As with the other reuse option, this option has a greater chance of getting grant funding since it based on beneficial reuse of the effluent.

This option requires the same level of treatment that Option 4Ai does which is to meet drinking water quality standards. The most likely locations for a poplar tree farm are south and west of the WWTP site. The reclaimed water would therefore be discharged in the same tributary basin as it originated from.

This option makes for beneficial use of the reclaimed water for poplar irrigation and groundwater recharge. The poplar trees would be dependent solely on rainfall and reclaimed water for irrigation. No potable water or surface water would be used for irrigation. During the dormant period and when the trees do not need water because it is raining, the reclaimed water would be used for groundwater recharge. As with Option 4Ai, this would serve to replenish groundwater stores. Both DOH and EPA staff have encouraged pursuit of this option.

As with Option 4Ai, this option would provide marginal improvement to water quality in the Chehalis River by removing all dry weather discharge to it. This option is also not affected by any potential future dry weather surface water quality restrictions during dry weather conditions.

TABLE VII-7
OPTION 4Aii COST ESTIMATE
POPLAR IRRIGATION WITH GROUNDWATER RECHARGE

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
Upgrade Existing Plant				
1. Two New Secondary Clarifiers	$995,000	$40,000	$0	$1,035,000
2. Rehabilitate Primary Clarifiers	$0	$75,000	$300,000	$375,000
3. Headworks Improvements	$120,000	$10,000	$10,000	$140,000
4. Aeration Basin Improvements	$476,000	$0	$0	$476,000
5. Solids Train Improvements	$0	$1,180,000	$0	$1,180,000
6. Equalization Storage Basin (4 MG)	$678,000	$0	$0	$678,000
7. Flood Protection Dike	$1,528,000	$0	$0	$1,528,000
8. Disinfection System Upgrade	$0	$275,000	$0	$275,000
9. Yard Piping Upgrades	$400,000	$100,000	$0	$500,000

10. Electrical System Upgrades	$400,000	$100,000	$50,000	$550,000
11. Instrumentation & Control System Upgrades	$200,000	$50,000	$50,000	$300,000
12. pH Adjustment	$200,000	$0	$0	$200,000
13. Miscellaneous Plant Upgrades	$0	$30,000	$450,000	$480,000
14. Advanced Treatment	$1,474,000	$0	$0	$1,474,000
End-Use Facility Pump Station, Force Main and Poplar Tree Farm	$1,985,000	$0	$0	$1,985,000
SUBTOTAL	**$8,458,000**	**$1,860,000**	**$860,000**	**$11,176,800**
Mobilization @ 5% Subtotal	$423,000 $8,881,000	$93,000 $1,953,000	$43,000 $903,000	$558,000 $11,735,000
Construction Contingency @ 30% Subtotal	$2,664,000 $11,545,000	$586,000 $2,539,000	$271,000 $1,174,000	$3,520,000 $15,256,000
Sales Tax @ 7.7% Subtotal	$889,000 $12,434,000	$196,000 $2,735,000	$90,000 $1,264,000	$1,174,000 $16,431,000
Engineering, Admin. & Legal @ 35%	$4,352,000	$957,000	$442,000	$5,750,000
Land for Poplar Tree Farm	$600,000	$0	$0	$600,000
Purchase Property East of Site	$150,000	$0	$0	$150,000
Total Capital Cost	**$17,536,000**	**$3,692,000**	**$1,706,000**	**$22,934,000**
Present Worth of O & M Cost (Relative)				$4,296,000
Total Estimated Present Worth				$24,384,000

This option has the same operational issues as Option 4Ai, plus an irrigation system and poplar tree farm to maintain. The land area required for the poplar tree farm is very large and it would require significant operations time to maintain however, the City may contract out for commercial tree farm management of the poplars.

OPTION 6 is to use Class A reclaimed water for stream flow augmentation of the Centralia Reach during dry weather conditions. Table VII-8 shows the cost summary for this option. The estimated capital cost of this option is $19.7 million, the estimated present worth of additional O&M cost is $3.4 million relative to the other options and the total present worth cost is $21.4 million. A detailed cost estimate is included in Appendix E. As with the other reuse options, this option has a greater chance of getting funding since it is based on beneficial reuse of the effluent. This option calls for beneficial stream flow augmentation of the Centralia Reach, which would help fish and downstream water rights holders.

This option requires a very high level of treatment to produce Class A reclaimed water. The high quality secondary effluent must also pass through an advanced treatment train that consists of coagulation, flocculation and filtration.

This option makes for beneficial reuse of the effluent by using it to augment low streamflows in the Centralia Reach during dry weather conditions. This option will serve to replenish much needed surface water that can be used beneficially by downstream water users. This option also creates Class A RW that can be used at a later date for potential consumptive uses, which would result in potable water conservation and provide additional relief on upstream water supply.

This option has an insignificant impact on water quality in the Chehalis River. The Class A RW is the cleanest grade of RW as presented by DOE and is cleaner than a lot of natural water bodies in the State.

This option would be affected by potential future water quality restrictions. This option may be difficult to implement because it will require the TMDL Study modeling to be re-evaluated to acknowledge that there is capacity in the Centralia Reach for BOD and ammonia. However, the Centralia Reach needs as much water in it as possible during the summer so that it can be used as a fish passage and to be able to supply downstream water rights holders. It just makes sense to clean up the effluent to a very high level (Class A RW) where it is almost drinking water quality and discharge into the Centralia Reach where it is needed.

This option requires advanced treatment to produce the Class A RW that takes additional operator time. Additional testing would be required to prove that the water quality in the Centralia Reach is not degraded as a result of implementing this option.

TABLE VII-8
OPTION 6 COST ESTIMATE
CLASS A RW FOR STREAMFLOW AUGMENTATION

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
Upgrade Existing Plant				
1. Two New Secondary Clarifiers	$995,000	$40,000	$0	$1,035,000
2. Rehabilitate Primary Clarifiers	$0	$75,000	$300,000	$375,000
3. Headworks Improvements	$120,000	$10,000	$10,000	$140,000
4. Aeration Basin Improvements	$476,000	$0	$0	$476,000

5. Solids Train Improvements	$0	$1,180,000	$0	$1,180,000
6. Equalization Storage Basin (2 MG)	$630,000	$0	$0	$630,000
7. Flood Protection Dike	$1,528,000	$0	$0	$1,528,000
8. UV Disinfection	$0	$500,000	$0	$500,000
9. Yard Piping Upgrades	$400,000	$100,000	$0	$500,000
10. Electrical System Upgrades	$400,000	$100,000	$50,000	$550,000
11. Instrumentation & Control System Upgrades	$200,000	$50,000	$50,000	$300,000
12. pH Adjustment	$200,000	$0	$0	$200,000
13. Miscellaneous Plant Upgrades	$0	$30,000	$450,000	$480,000
14. Advanced Treatment	$1,474,000	$0	$0	$1,474,000
End-Use Facility New Outfall Diffuser	$500,000	$0	$0	$500,000
SUBTOTAL	**$6,923,000**	**$2,085,000**	**$860,000**	**$11,176,800**
Mobilization @ 5% Subtotal	$346,000 $7,269,000	$104,000 $2,189,000	$43,000 $903,000	$493,000 $10,359,000
Construction Contingency @ 30% Subtotal	$2,181,000 $9,450,000	$657,000 $2,846,000	$271,000 $1,174,000	$3,109,000 $13,470,000
Sales Tax @ 7.7% Subtotal	$728,000 $10,178,000	$219,000 $3,065,000	$90,390 $1,264,000	$1,037,000 $14,507,000
Engineering, Admin. & Legal @ 35%	$3,562,000	$1,073,000	$442,000	$5,077,000
Purchase Property East of Site	$150,000	$0	$0	$150,000
Total Capital Cost	**$13,890,000**	**$4,138,000**	**$1,706,000**	**$19,734,000**
Present Worth of O & M Cost (Relative)				$3,434,000
Total Estimated Present Worth				$21,360,000

SUMMARY

The following three tables summarize the preliminary evaluation process: Table VII-9 shows the capital, O&M and present worth cost for each of the options, Table VII-10 shows an evaluation matrix for all of the options based on the evaluation criteria that was presented herein and Table VII-11 shows a list of advantages and disadvantages for each option. The evaluation matrix is meant to be a summary only and is not used to eliminate options from further consideration.

TABLE VII-9
CHEHALIS WWTP CAPITAL AND PRESENT WORTH COSTS (EXISTING WWTP UPGRADE)

Alternative		Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total Amount	Present Worth O&M Cost	Present Worth
1A.	Regional Plant	$42.8	$0.0	$ 3.0	$45.8	$0	$37.5
1B.	Dry Weather Regional	**eliminated**					
2.	Discharge Downstream	$20.0	$3.7	$1.7	$25.4	$3.2	$26.4
3.	River Enhancement	$13.5	$4.1	$1.7	$19.4	2.9	$20.5
4Ai.	Class A RW to Infiltration	$21.3	$3.7	$1.7	$26.7	$4.3	$28.4
4Aii.	Class A RW to Poplars w/GW Recharge	$17.5	$3.7	$1.7	$22.9	$4.3	$24.4
4Bi.	Class B RW to Poplars w/Storage Pond	**eliminated**					
4Bii.	Class B RW to Natural Wetland	**eliminated**					

4Biii.	Class B RW to Constructed Wetland	**eliminated**					
5.	Store All Dry Weather Flows	**eliminated**					
6.	Class A RW for Streamflow Augmentation	$13.9	$4.1	$ 1.7	$19.7	$3.4	$21.4

The regional plant (Option No. 1A) has a capital cost of almost $20 million more than the next most expensive option, which is to use reclaimed water for groundwater infiltration during dry weather conditions. The reduction in relative present worth from less O&M still results in a relative cost difference of more than $9 million. It is unlikely that there will be a large enough grant to make up the difference. Therefore, the regional option will not be considered any further. From the evaluation matrix, the option to discharge downstream (Option 2) has a considerably higher (less desirable) score than the other options. But since it is relatively easy to implement, it will be retained for further consideration. From this preliminary analysis, the river enhancement option (No. 3) has the lowest capital and present worth cost and also has a very favorable ranking score from the evaluation matrix. Option No. 3 will be retained for further consideration. The first reuse option to recharge the groundwater (Option 4Ai) has a

INSERT TABLE VII-10 TREATMENT AND END USE ALTERNATIVES

INSERT TABLE VII-11 ADVANTAGES AND DISADVANTAGES very favorable score, but the capital and present worth cost is considerably more than Option 4Aii. It will also be more difficult to implement than the other reuse options. Therefore, Option 4Ai will not be considered any further. The second reuse option which is poplar irrigation in conjunction with groundwater recharge (Option 4Aii) has the best evaluation score and the third lowest capital and present worth cost. It will therefore be retained for further consideration. The third reuse option is to use Class A RW for stream flow augmentation (No.6), which has a very favorable score and the second lowest capital and O&M costs. It may be difficult to implement due to the TMDL, but will be retained for further consideration.

In summary, the following four options will be retained for further consideration:

2	Discharge downstream of the Skookumchuck River
3	River enhancement
4Aii	Class A reclaimed water for poplar irrigation and groundwater recharge
6	Class A reclaimed water for stream flow augmentation

TREATMENT PROCESS EVALUATION

After narrowing the end use alternatives, the next step in the analysis is to determine whether the existing secondary treatment processes at the plant should be upgraded or replaced with a new treatment process plant at the existing site or a new site. The required upgrades to the existing plant were presented at the beginning of this section. Several secondary treatment processes were evaluated with regard to WWTP loading and flow variations. The evaluation included contact stabilization, conventional complete mix activated sludge coupled with extended aeration and a Sequencing Batch Reactor (SBR).

The design criteria for the treatment process are included in section V of this report. The design flow is 5.91 MGD which is the future maximum wet weather monthly average flow. There are no existing or planned industries that will require special handling of the waste at the plant. The city may require certain industries to provide pre treatment to assure compatibility with the WWTP. The City currently does not have any septage receiving facilities at the plant and does not plan on providing them in the future.

The new or upgraded plant must be capable of reliably producing a high quality secondary effluent that is suitable for further treatment to a Class A reuse water. The secondary process must also be capable of complete nitrification and denitrification so that the nitrate concentration is less than 10 mg/l. The secondary treatment system must also be able to produce a high quality effluent with a high variability of flows and loadings. Expandibility is also an important criteria for process selection. The following table VII-12 summarizes the secondary process screening evaluation.

TABLE VII-12
SECONDARY TREATMENT PROCESS EVALUATION

SECONDARY TREATMENT PROCESS	PROCESS DESCRIPTION	ADVANTAGES	DISADVANTAGES	DECISION
Contact Stabilization	Suspended growth biological treatment process, uses a contact basin where aerated microorganisms are mixed with incoming raw sewage for a short time. Most of the microorganisms are stored in the stabilization basin which is also aerated.	• Reliably produces effluent with low TSS and BOD. • Avoids wash outs due to high flows. • Very high procress flexibility easy to expand.	• Difficult to achieve nitrification • Difficult to achieve denitrification	Eliminate due to difficulty with nitrification and denitrification
Conventional complete mix activated sludge coupled with extended aeration	Suspended growth biological treatment process uses aerated basins where mixing occurs with incoming raw sewage and return activated sludge. Extended aeration mode has longer detention times with a lower solids loading rate.	• Reliably produces effluent with low TSS and BOD. • Excellent nitrification. • Existing basins are suitable for use.	• Denitrification requires additional tankage. • Difficulty to switch between process modes.	Keep for further evaluation

		• Easy to expand.		
Sequencing Batch Reactor (SBR)	Suspended growth biological process, uses basin process where aeration, mixing and clarification occur in same basin.	• Reliably produces effluent with low TSS and BOD. • Excellent nitrification. • Excellent denitrification w/out additional tankage. • Handles high flow variations well. • Easy to expand.	• High rate of discharge usually requires equalization storage to avoid oversizing disinfection components. • Existing basins are not suitable for use.	Keep for further evaluation

A detailed discussion for using the existing plant in a combined complete mix and extended aeration process mode was presented earlier in this section.

Advantages of the SBR system include:

1.Elimination of secondary clarifier and RAS pumping.
2. High tolerance for peak flows and shock loadings.
3. Avoidance of MLSS washout during peak flow events.
4. Clarification under ideal quiescent conditions.
5. Process flexibility to control filamentous bulking.
6. Minimal operator attention.
7. Easy nutrient removal.
8. Complete nitrification.

SBR PROCESS

The batch activated sludge process utilized in a SBR is a relatively old technology and, in fact, initial biological treatment systems with suspended growth were of the batch activated sludge type. The process consists of adding new sewage to a basin where solids from the previous batch have been retained, mixing and aerating the solids-sewage mixture, shutting off the feed by diversion to another basin, allowing to settle, and decanting the clear effluent from above the retained solids. A portion of the solids are wasted from the aeration basin at the end of the supernatant withdrawal period. The SBR process utilizes a single, complete-mix reactor in which all steps of treatment occur. Discrete cycles are used during prescribed time intervals. The MLSS remain in the reactor during all cycles, thereby eliminating the need for a separate clarifier. Specific treatment cycles are:

1. Fill (raw or settled wastewater fed to the reactor).
2. React (aeration/mixing of the reactor contents).
3. Settle (quiescent settling and separation of MLSS from the treated wastewater).
4. Draw (withdrawal of treated wastewater from the reactor).
5. Idle (removal of waste sludge from the reactor bottom).

The idle cycle may be omitted by wasting sludge near the end of the react or draw cycles. Due to the batch nature of the process, flow equalization or multiple reactors are required to accommodate the continual inflow of wastewater to the facility.

An SBR plant must utilize either a storage or equalization tank and an SBR tank or a minimum of two SBR tanks to accommodate continuous influent flow.

Recent technological developments in system and component control (programmable logic controllers, PLCs) have allowed batch activated sludge technology to compete with continuous flow technology on an economic basis and consequently many of these facilities have been recently constructed or are currently in implementation. The batch activated sludge facilities would be expected to produce an effluent with an average BOD_5 and TSS concentration of 10 mg/l or less, and an effluent ammonia nitrogen concentration of 3 mg/l or less. The facilities can also be operated in a mode to achieve enhanced biological phosphorous reduction and denitrification of the oxidized nitrogen (nitrates formed during removal of ammonia) by implementing sequenced anaerobic-aerobic conditions. Phosphorus is not a primary pollutant of concern, although the SBR activated sludge treatment system can be operated to enhance phosphorus removal by the biological organisms by "luxury uptake" (biological phosphorus removal). The system, when operated to accomplish denitrification, would develop dominant organism strains that have the capability to store excess energy in phosphorus compounds. The energy is used during the anaerobic and anoxic cycles to sustain the organisms and to allow them to preferentially compete for food. This would result in total nitrogen concentration in the effluent at approximately 10 mg/l and phosphorous concentration of approximately 4 mg/l.

Design Sizing of SBR Treatment Plant Components

Preliminary sizing of the SBR treatment plant components are based on the design waste flow and loading values and the desired effluent concentrations as discussed in Section V and VII of this Plan. All components were sized based on guidelines contained in the DOE's "Criteria For Sewage Works Design", where applicable. Since the NPDES permit restrictions require a higher quality effluent during dry weather conditions, the SBR plant is designed to produce a better effluent during the summer than in the winter. The SBR will be designed for an effluent with BOD_5 and TSS concentrations of less than 10 mg/l for flows up to 3.5 MGD, which is a little more than the future dry weather maximum monthly average flow rate. The design effluent quality above 3.5 MGD will be 30 mg/l for both BOD_5 and TSS.

An SBR treatment plant utilizing three sequencing batch reactors (basins) would be used to provide flexibility for the large variation of inflows. The SBR system is sized based on operating through four cycles per day at the design dry weather flow of 3.5 MGD. Each cycle duration is 360 minutes (6 hours). For the first 120 minutes of each cycle the SBR would be in a fill phase. Following the fill phase the SBR would continue to aerate, settle and then decant. These three phases (which also include sludge wasting) total 210 minutes. Three basins are utilized so that when the fill cycle is complete for the first basin, inflow can be diverted to the second basin, and then the third basin. As flows increase over 3.5 MGD, the number of cycles and cycle times are adjusted. At the 2025 peak daily flow rate of 13.0 MGD, there would be eight cycles per day in each basin that would last three hours each.

The SBR basin is designed to operate as an extended aeration, plug flow unit. Basin sizing is based on a food to microorganism (F:M) ratio of 0.15 gBOD/gMLSS/day, a MLSS concentration of 3,000 mg/l and four 360 minute cycles per day. The aeration system is sized to provide 1.30 pounds of oxygen per pound of BOD_5 removed plus an allowance for oxygen required for nitrification which is 4.6 pounds/pound TKN oxidized. The design AOR is 14,654 lbs. O2/day. Four blowers will be required for the process including one spare blower in order to meet reuse water reliability standards. A DO probe in each basin is used to determine the time that the positive displacement blower would operate. This is necessary to maximize the nitrification/denitrification process. Waste activated sludge would be transferred by a pump outside the basin. Process control would be provided by a control panel with integral programmable logic controller (PLC). A dry pit centrifugal pumping station is required for mixing of the SBR cells. Four pumps are required in the pump station, including one spare.

The SBR needs to be sized to treat inflows from 1.0 MGD up to 13.0 MGD. 1 MGD is the current dry weather average flow and 13 MGD is the 2025 projected peak daily average flow that usually occurs during flooding events. The SBR would have a design flow of 3.5 MGD which is a little more than the projected 2025 maximum monthly average flow during dry weather conditions. Up to 3.5 MGD, the SBR would be expected to produce an excellent quality effluent with BOD_5 and TSS both below 10 mg/l, ammonia concentration below 4 mg/l and total nitrogen less than 10 mg/l. This effluent is much better than the dry weather permit limit of 20 mg/l for BOD_5 and TSS. However, as flows increase over 3.5 MGD, the effluent requirements become less stringent because the wet weather limits would generally apply for these higher flows. So, for flows over 3.5 MGD, the SBR would only be expected to produce a good effluent with BOD_5 and TSS of approximately 30 mg/l and ammonia of 15 mg/l.

The SBR has a lot of flexibility that allows it to produce a good effluent even when inflows are 3.5 to 4.0 times greater than the design flow. This means that the SBR would be able to treat inflows up to 12.25 - 14.0 MGD and still meet the permit limits. It should also be noted that the flows exceed 8 MGD very infrequently. These very high flows usually coincide with flooding events and do last very long.

The SBR would be designed to handle up to 13.0 MGD and would therefore not need any influent equalization storage. Equalization storage would be provided, when necessary on the SBR effluent. The equalization storage volume is different for each end use option as discussed previously.

The preliminary basin sizing is based on the "Jet Tech" SBR. The preliminary design calls for three basins that are 95' by 95' with a side water depth of 23'. This makes for a total design volume of 4.65 MG. The basins would have a total depth of 25' to allow for a two-foot freeboard. The minimum water level will be a 14'. At the design flow of 3.5 MGD, the HRT is 1.33 days and the aerobic sludge age is 8 days minimum.

One drawback to the SBR is that the basins are drained in a batch mode that only takes about 30 minutes at design flow. The corresponding flow rate is 9,700 GPM (14.0 MGD) which requires post SBR equalization storage. Without this storage, the disinfection process would need to have a larger capacity as would discharge pumping and force mains. The basin volume should be at least 300,000 gallons to evenly distribute the SBR discharge flow between decant events from each of the three basins. This is especially important for the reuse options because the advanced sand filters require a steady flow rate to produce a high quality effluent. The recently built secondary clarifier would be suitable for this purpose at the existing site. The new site would utilize an in ground concrete basin or the EQ storage basin depending on the end use option.

A new SBR plant can either be built at the existing site or a potential new WWTP site. The preliminary siting analysis presented in Appendix E identified a preferred site alternative located between the Darigold WWTP and I-5 just south of Main Street (see Figure VII-31). Final site selection will be presented in the Facility Plan and will be based on a more detailed analysis which may require an Environmental Impact Statement (EIS).

With an SBR plant all of the other treatment basins in the existing plant would not be needed for treatment of the sewage. The SBR basins would be constructed where the existing aeration basins are presently located. The trickling filters, primary clarifiers and secondary clarifiers would no longer be needed. To make room for floodwater and to clean up the site it is recommended that any unused structures be demolished. However, the unused basins may be able to serve a different purpose in the SBR plant layout. A potential use for the recently built secondary clarifier is discussed below.

The SBR basins at the existing site need to be high enough so that they can discharge by gravity into the recently built secondary clarifier. The basins should be built with a top of wall elevation of 189.0' that is approximately 14' above the existing ground level at the existing site. This elevation is high enough that the new basins would be well above the 100-year and 500-year flood level. The ground elevation at the proposed new site is above the 100-year elevation and would allow for a conventional wall height of 10-feet above grade. A new headworks structure and lift station is required at either site to allow gravity flow between the headworks and the SBR basins.

The reliability requirements are more stringent for reuse applications than for plants with surface water discharge. If the SBR plant is used for reuse, the regulations require that with one basin out of service, the remaining basins can still meet secondary treatment standards. The recommended configuration calls for three basins with a volume of 1.55 MG each. So with one basin out of service, the remaining volume is 3.1 MG At the design dry weather flow rate of 3.5 MGD, the HRT is 0.9 days and the SBR will still be able to produce a 30/30 effluent.

INSERT FIGURE VII-31 SITE PLAN

SBR FOR OPTION 2

Enduse option 2 is to discharge downstream of the Skookumchuck River during dry weather conditions. The SBR would produce an effluent that would meet all permit conditions including ammonia removal without any additional treatment steps. Figure VII-32 shows a schematic for this option and Figure VII-33 shows the site plan for this option with a new SBR at the existing site and a 6.0 MG equalization storage basin. The estimated capital cost for this option with a new SBR at the existing site is $25.2 million and is shown in Table VII-13. The estimated present worth cost of O&M is $1.9 million and the present worth cost is $23.9 million. A detailed cost estimate is included in Appendix E.

A new SBR can also be built at a new site that is out of the floodplain of the Chehalis River and Dillenbaugh Creek. The new plant would consist of an SBR and pumping facilities for the effluent and the sludge. The solids handling facilities at the existing plant would be retained for use if needed. Riverside and Prindle Street pump stations would be reconfigured to pump to the new site. The pump stations would not require significant upgrades, which results in a large cost saving from recommended options to upgrade the pump stations presented in Section VI. A new 12-inch raw sewage force main is required from the Prindle pump station to the new site. The existing 10-inch force main from Prindle pump station to the existing site can be used to convey flows from the Riverside Pump Station to the Prindle pump station. New dual effluent force mains of 18 and 12-inch diameter would also be needed from the new site to the existing outfall diffuser and to the downstream discharge location. A significant portion of the existing 18-inch force main can be utilized to convey effluent to the existing outfall. A 6.0 MG equalization storage basin would also be provided at the new site. Figure VII-34 shows the site plan and potential force main routes between the existing and new site. The force main route between the existing site and the proposed downstream discharge location is the same as shown in Figure VII-5.

INSERT FIGURE VII-32 OPTION 2 SCHEMATIC DIAGRAM

INSERT FIGURE VII-33 OPTION 2 SITE PLAN

TABLE VII-13 ESTIMATED CAPITAL COST WITH NEW SBR AT THE EXISTING SITE FOR OPTION 2				
Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant 1. SBR Basin and Equipment	$2,925,000	$0	$0	$3,318,600
2. Headworks Modifications	$270,000	$0	$0	$270,000

3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (6.0 MG)	$685,000	$0	$0	$685,000
5. Disinfection Upgrade	$0	$275,000	$0	$275,000
6. Yard Piping	$293,000	$0	$0	$293,000
7. Electrical (Including I&C)	$293,000	$0	$0	$293,000
8. Additional Standby Generator	$108,000	$0	$0	$108,000
9. New Control Building	$160,000	$0	$0	$160,000
10. Misc. Improvements	$330,000	$100,000	$325,000	$755,000
11. Modification to Prevent Flood Damage	$1,000,000	$0	$0	$1,000,000
End-Use Facility Force Main, Pump Station and New Outfall Downstream of the Skookumchuck River	$4,335,000	$0	$0	$4,335,000
SUBTOTAL	**$10,793,100**	**$1,555,000**	**$325,000**	**$12,673,100**
Mobilization @ 5% Subtotal	$540,000 $11,333,000	$78,000 $1,633,000	$16,000 $341,000	$634,000 $13,307,000
Construction Contingency @ 30% Subtotal	$3,400,000 $14,733,000	$490,000 $2,123,000	$102,000 $443,000	$3,992,000 $17,299,000
Sales Tax @ 7.7% Subtotal	$1,134,000 $15,867,000	$163,000 $2,286,000	$34,000 $477,000	$1,331,000 $18,630,000
Engineering, Admin. & Legal @ 35%	$5,553,000	$800,000	$167,000	$6,520,000
Purchase Property East of Site	$50,000	$0	$0	$50,000
Total Capital Cost	**$21,470,000**	**$3,086,000**	**$644,000**	**$25,200,000**
Present Worth of O&M Cost (Relative)				$1,924,000
Total Estimated Present Worth				$23,905,000

INSERT FIGURE VII-34 SITE PLAN AND POTENTIAL FORCE MAIN ROUTES

The estimated capital cost for this option with a new SBR at the new site is $ 31.1 million and is shown in Table VII-14. The estimated present worth of O&M cost is $2.3 million and the total present worth cost is $26.4 million. A detailed cost estimate is included in Appendix E.

TABLE VII-14
ESTIMATED CAPITAL COST WITH NEW SBR AT THE NEW SITE FOR OPTION 2

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$3,424,000	$0	$0	$3,424,000
2. Headworks Modifications	$515,000	$0	$0	$515,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (6.0 MG)	$664,000	$0	$0	$664,000
5. Sitework	$1,700,000	$0	$0	$1,700,000
6. Raw Sewage Lift Station	$312,000	$0	$0	$312,000
7. Electrical (Including I&C)	$1,121,000	$0	$0	$1,121,000
8. Raw Sewage Pumping & Conveyance	($984,000)	$0	$0	($984,000)
9. New Control Building	$830,000	$0	$0	$830,000
10. Misc. Improvements	$0	$313,000	$300,000	$613,000
11. Effluent Equalization and Conveyance	$1,368,000	$0	$0	$1,368,000
End-Use Facility				
Force Main, Pump Station and New Outfall Downstream of the Skookumchuck River	$4,335,000	$0	$0	$4,335,000
SUBTOTAL	**$13,285,000**	**$1,493,000**	**$300,000**	**$15,078,000**
Mobilization @ 5%	$664,000	$75,000	$15,000	$754,000
Subtotal	$14,949,000	$1,568,000	$315,000	$15,832,000
Construction Contingency @ 30%	$4,185,000	$470,000	$95,000	$4,750,000
Subtotal	$18,134,000	$2,038,000	$410,000	$20,582,000
Sales Tax @ 7.7%	$1,396,000	$157,000	$32,000	$1,585,000
Subtotal	$19,530,000	$2,195,000	$442,000	$22,167,000
Engineering, Admin. & Legal @ 35%	$6,836,000	$768,000	$155,000	$7,759,000
Purchase Property for New Site	$1,200,000	$0	$0	$1,200,000
Total Capital Cost	**$27,566,000**	**$2,963,000**	**$597,000**	**$31,126,000**
Present Worth of O&M Cost (Relative)				$2,341,000
Total Estimated Present Worth				$26,428,000

OPTION 3

Enduse option 3 is to enhance the river and continue to discharge to the current outfall location all year long. The SBR would produce an effluent that would meet all permit conditions including ammonia removal without any additional treatment steps.

Figure VII-35 shows a schematic for this option and Figure VII-36 shows the site plan for this option with a new SBR at the existing site with no equalization storage basin. The estimated capital cost for this option with a new SBR at the existing site is $20.1 million and is shown in Table VII-15. The estimated present worth of O&M cost is $1.4 million and the total present worth cost is $18.7 million. A detailed cost estimate is included in Appendix E.

INSERT FIGURE VII-35 OPTION 3 SCHEMATIC

INSERT FIGURE VII-36 OPTION 3 SITE PLAN

TABLE VII-15 ESTIMATED CAPITAL COST WITH NEW SBR AT THE EXISTING SITE FOR OPTION 3				
Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$3,050,000	$0	$0	$3,050,000
2. Headworks Modifications	$270,000	$0	$0	$270,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Disinfection Upgrade	$0	$500,000	$0	$500,000
5. Yard Piping	$293,000	$0	$0	$293,500
6. Electrical (Including I&C)	$293,000	$0	$0	$293,000
7. Additional Standby Generator	$108,000	$0	$0	$108,000
8. New Control Building	$160,000	$0	$0	$160,000
9. Misc. Improvements	$1,024,000	$100,000	$325,000	$1,024,000
10. Modifications to Prevent Flood Damage	$1,000,000	$0	$0	$1,000,000
End-Use Facility				
New Outfall/Diffuser and Aeration Facilities	$1,700,000	$0	$0	$1,700,000
SUBTOTAL	**$7,898,000**	**$1,780,000**	**$325,000**	**$10,003,000**
Mobilization @ 5%	$395,000	$89,000	$16,000	$500,000
Subtotal	$8,293,000	$1,869,000	$341,000	$10,503,000
Construction Contingency @ 30%	$2,488,000	$561,000	$102,000	$3,151,000
Subtotal	$10,781,600	$2,430,000	$443,000	$13,654,000
Sales Tax @ 7.7%	$830,000	$87,000	$34,000	$951,000
Subtotal	$11,611,000	$2,617,000	$477,000	$14,605,000
Engineering, Admin. & Legal @ 35%	$4,064,000	$916,000	$167,000	$5,147,000
Land and Right-of-Way for Aeration Facilities	$200,000	$0	$0	$200,000
Purchase Property East of Site	$50,000	$0	$0	$50,000
Total Capital Cost	**$15,925,000**	**$3,533,000**	**$644,000**	**$20,102,000**
Present Worth of O&M Cost (Relative)				$1,390,000
Total Estimated Present Worth				$18,728,000

The SBR at the new site will be the same as Option 2 except there is not a large equalization storage basin. Figure VII-37 shows the site plan and potential force main routes between the existing and new site. The estimated capital cost for this option with a new SBR at the new site is $26.1 million and is shown in Table VII-16. The estimated additional O&M cost is $1.6 million and the total present worth cost is $21.8 million. A detailed cost estimate is included in Appendix E

INSERT FIGURE VII-37 SITE PLAN AND POTENTIAL FORCE MAIN ROUTES

TABLE VII-16 ESTIMATED CAPITAL COST WITH NEW SBR AT THE NEW SITE FOR OPTION 3				
	Amount Required to	Amount for Capital	Amount for Operational	

Item	Meet Permit	Improvements	Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$3,424,000	$0	$0	$3,424,000
2. Headworks Modifications	$515,000	$0	$0	$515,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Sitework	$1,700,000	$0	$0	$1,700,000
5. Raw Sewage Lift Station	$312,000	$0	$0	$312,000
6. Electrical (Including I&C)	$1,121,000	$0	$0	$1,121,000
7. Raw Sewage Pumping & Conveyance	($984,000)	$0	$0	($984,000)
8. New Control Building	$830,000	$0	$0	$830,000
9. Misc. Improvements	$500,000	$500,000	$250,000	$1,250,000
10. Effluent Equalization and Conveyance	$1,368,000	$0	$0	$1,368,000
End-Use Facility				
New Outfall/Diffuser and Aeration Facilities	$1,700,000	$0	$0	$1,700,000
SUBTOTAL	**$10,486,000**	**$1,730,000**	**$250,000**	**$12,466,000**
Mobilization @ 5%	$524,000	$87,000	$13,000	$624,000
Subtotal	$11,010,000	$1,817,000	$263,000	$13,090,000
Construction Contingency @ 30%	$3,303,000	$545,000	$79,000	$3,927,000
Subtotal	$14,313,000	$2,362,000	$342,000	$17,017,000
Sales Tax @ 7.7%	$1,102,000	$182,000	$26,000	$1,310,000
Subtotal	$15,415,000	$2,544,000	$368,000	$18,327,000
Engineering, Admin. & Legal @ 35%	$5,395,000	$890,000	$129,000	$6,414,000
Land and Right-of-Way for Aeration Facilities	$200,000	$0	$0	$200,000
Purchase Property for New Site	$1,200,000	$0	$0	$1,200,000
Total Capital Cost	**$22,210,000**	**$3,434,000**	**$497,000**	**$26,141,000**
Present Worth of O&M Cost (Relative)				$1,618,000
Total Estimated Present Worth				$21,756,000

OPTION 4Aii

Option 4Aii is to use Class A reclaimed water for poplar irrigation in conjunction with groundwater recharge. The SBR would produce an effluent that is both nitrified and denitrified. The SBR effluent would then pass through the advanced treatment train that would consist of coagulation and filtering. Denitrification using methanol is not needed under SBR options for Class A RW. During wet weather conditions, the advanced treatment train would not be used since Class A reclaimed water is not required. However, the filters may be used to facilitate 85% reduction of TSS if needed in rare storm events. A 4.0 MG equalization storage basin is included in this option. Figure VII-38 shows a schematic for this option and Figure VII-39 shows the site plan for this option with a new SBR at the existing site with a 4.0 MG equalization storage basin. The estimated capital cost for this option with a new SBR at the existing site is $23.2 million and is shown in Table VII-17. The estimated present worth of O&M cost is $2.6 million and the total present worth cost is $22.0 million. A detailed cost estimate is included in Appendix E.

INSERT FIGURE VII-38 OPTION 4Aii SCHEMATIC

INSERT FIGURE VII-39 OPTION 4Aii SITE PLAN

TABLE VII-17
ESTIMATED CAPITAL COST WITH NEW SBR AT THE EXISTING SITE FOR OPTION 4Aii

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
<u>New SBR Plant</u>				
1. SBR Basin and Equipment	$2,925,000	$0	$0	$2,925,000
2. Headworks Modifications	$270,000	$0	$0	$270,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (5.0 MG)	$419,500	$0	$0	$419,500
5. Disinfection Upgrade	$0	$275,000	$0	$275,000
6. Yard Piping	$75,000	$0	$0	$75,000
7. Electrical (Including I&C)	$125,000	$0	$0	$125,000
8. Additional Standby Generator	$108,000	$0	$0	$108,000
9. New Control Building	$160,000	$0	$0	$160,000
10. Misc. Improvements	$330,000	$100,000	$325,000	$755,000
11. Modification to Prevent Flood Damage	$1,000,000	$0	$0	$1,000,000
12. Advanced Treatment	$1,424,000	$0	$0	$1,424,000
<u>End-Use Facility</u>				
Reclaimed Water Pump Station, Force Main and Poplar Tree Farm	$1,985,000	$0	$0	$1,985,000
SUBTOTAL	**$9,502,000**	**$1,555,000**	**$325,000**	**$11,382,000**
Mobilization @ 5%	$475,000	$77,750	$16,250	$568,985
Subtotal	$9,977,000	$1,632,750	$341,250	$11,948,685
Construction Contingency @ 30%	$2,993,000	$489,825	$102,375	$3,584,606
Subtotal	$12,970,000	$2,122,575	$443,625	$15,533,290
Sales Tax @ 7.7%	$999,000	$163,438	$34,159	$1,196,064
Subtotal	$13,969,000	$2,286,013	$477,784	$16,729,354
Engineering, Admin. & Legal @ 35%	$4,889,000	$800,105	$167,244	$5,855,274
Land for Poplar Tree Farm	$600,000	$0	$0	$600,000
Purchase Property East of Site	$50,000	$0	$0	$50,000
Total Capital Cost	**$19,508,000**	**$3,086,000**	**$644,000**	**$23,238,000**
Present Worth of O&M Cost (Relative)				$2,662,000
Total Estimated Present Worth				$22,044,000

The SBR at the new site will be the same as Option 2. Figure VII-40 shows the site plan and potential force main routes between the existing and new site. The estimated capital cost for this option with a new SBR at the new site is $29.5 million and is shown in Table VII-18. The estimated present worth of O&M cost is $3.0 million and the total present worth cost is $25.5 million. A detailed cost estimate is included in Appendix E.

Option 6 is to use Class A reclaimed water for streamflow augmentation in the Centralia Reach. The SBR would produce a high quality secondary effluent that is both nitrified and denitrified within the SBR process basins. The SBR effluent would then pass through the advanced treatment train that would consist of coagulation and filtering to produce a Class A reclaimed water. This option does not require any equalization storage because there is no dry weather discharge limit or need for pumping of the reclaimed water. Figure VII-41 shows a schematic for this option and Figure VII-42 shows the site plan for this option with a new SBR at the existing site without any equalization storage. The estimated capital cost for this option with a new SBR at the existing site is $20.1 million and is shown in Table VII-19. The estimated present worth of relative O&M cost is $1.8 million and the present worth cost is $19.1 million. A detailed cost estimate is included in Appendix E.

The SBR at the new site would be the same as Option 2 except there is not an equalization storage basin. Figure VII-43 shows the site plan and potential force main routes between the existing and new sites. The estimated capital cost of this option with a new SBR at a new site is $26.4 million and is shown in Table VII-20. The estimated present worth of relative O&M cost is $2.1 million and the present worth cost is $22.4 million. A detailed cost estimate is included in Appendix E.

INSERT FIGURE VII-40 SBR NEW AND EXISTING SITE PLAN AND POTENTIAL FORCE MAIN ROUTES

TABLE VII-18
ESTIMATED CAPITAL COST WITH NEW SBR AT THE NEW SITE FOR OPTION 4Aii

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$3,424,000	$0	$0	$3,424,000
2. Headworks Modifications	$515,000	$0	$0	$515,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (4.0 MG)	$546,000	$0	$0	$546,000
5. Sitework	$1,700,000	$0	$0	$1,700,000
6. Raw Sewage Lift Station	$312,000	$0	$0	$312,000
7. Electrical (Including I&C)	$1,121,000	$0	$0	$1,121,000
8. Raw Sewage Pumping & Conveyance	($984,000)	$0	$0	($984,000)
9. New Control Building	$830,000	$0	$0	$830,000
10. Misc. Improvements	$0	$313,000	$300,000	$613,000
11. Effluent Equalization and Conveyance	$1,518,000	$0	$0	$1,518,000
12. Advanced Treatment	$1,374,000	$0	$0	$1,374,000
End-Use Facility				
Force Main, Pump Station and New Outfall Downstream of the Skookumchuck River	$1,985,000	$0	$0	$1,985,000
SUBTOTAL	**$12,191,000**	**$1,493,000**	**$300,000**	**$13,984,000**
Mobilization @ 5%	$610,000	$75,000	$15,000	$700,000
Subtotal	$12,801,000	$1,568,000	$315,000	$14,684,000
Construction Contingency @ 30%	$3,840,000	$470,000	$95,000	$4,405,000
Subtotal	$16,641,000	$2,038,000	$410,000	$19,089,000
Sales Tax @ 7.7%	$1,281,000	$157,000	$32,000	$1,470,000
Subtotal	$17,922,000	$2,195,000	$442,000	$20,559,000
Engineering, Admin. & Legal @ 35%	$6,273,000	$768,000	$155,000	$7,196,000
Land for Poplar Tree Farm	$600,000	$0	$0	$600,000
Purchase Property for New Site	$1,200,000	$0	$0	$1,200,000
Total Capital Cost	**$25,995,000**	**$2,963,000**	**$597,000**	**$29,555,000**
Present Worth of O&M Cost (Relative)				$2,972,000
Total Estimated Present Worth				$25,459,000

INSERT FIGURE VII-41 SCHEMATIC FLOW DIAGRAM

INSERT FIGURE VII-42 OPTION 6 W/SBR

TABLE VII-19
ESTIMATED CAPITAL COST WITH NEW SBR AT THE EXISTING SITE FOR OPTION 6

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$2,925,000	$0	$0	$2,925,000
2. Headworks Modifications	$270,000	$0	$0	$270,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (5.0 MG)	$420,000	$0	$0	$420,000
5. Disinfection Upgrade	$0	$500,000	$0	$500,000
6. Yard Piping	$293,000	$0	$0	$293,000
7. Electrical (Including I&C)	$293,000	$0	$0	$293,000
8. Additional Standby Generator	$108,000	$0	$0	$108,000
9. New Control Building	$160,000	$0	$0	$160,000
10. Misc. Improvements	$624,000	$100,000	$325,000	$755,000
11. Modification to Prevent Flood Damage	$1,000,000	$0	$0	$1,000,000
12. Advanced Treatment	$1,424,000	$0	$0	$1,424,000
End-Use Facility				
New Outfall	$500,000	$0	$0	$500,000
SUBTOTAL	**$8,017,000**	**$1,780,000**	**$325,000**	**$10,122,000**
Mobilization @ 5%	$401,000	$89,000	$16,000	$506,000
Subtotal	$8,418,000	$1,869,000	$341,000	$10,628,000
Construction Contingency @ 30%	$2,525,000	$561,000	$102,000	$3,188,000
Subtotal	$10,943,000	$2,430,000	$443,000	$13,816,000
Sales Tax @ 7.7%	$843,000	$187,000	$34,000	$1,064,000
Subtotal	$11,786,000	$2,617,000	$477,000	$14,880,000
Engineering, Admin. & Legal @ 35%	$4,125,000	$916,000	$167,000	$5,208,000
Purchase Property for New Site	$50,000	$0	$0	$50,000
Total Capital Cost	**$15,961,000**	**$3,533,000**	**$644,000**	**$20,138,000**
Present Worth of O&M Cost (Relative)				$1,800,000
Total Estimated Present Worth				$19,130,000

INSERT FIGURE VII-43 NEW WWTP SITE PLAN (OPTION 6)

TABLE VII-20
ESTIMATED CAPITAL COST WITH NEW SBR AT THE NEW SITE FOR OPTION 6

Item	Amount Required to Meet Permit	Amount for Capital Improvements	Amount for Operational Enhancement	Total
New SBR Plant				
1. SBR Basin and Equipment	$3,424,000	$0	$0	$3,424,000
2. Headworks Modifications	$515,000	$0	$0	$515,000
3. Solids Train Modifications	$0	$1,180,000	$0	$1,180,000
4. Equalization Storage (4.0 MG) and Conveyance	$546,000	$0	$0	$546,000
5. Sitework	$1,700,000	$0	$0	$1,700,000
6. Raw Sewage Lift Station	$312,000	$0	$0	$312,000
7. Electrical (Including I&C)	$1,121,000	$0	$0	$1,121,000
8. Raw Sewage Pumping & Conveyance	($984,000)	$0	$0	($984,000)
9. New Control Building	$830,000	$0	$0	$830,000
10. Misc. Improvements	$0	$50,000	$250,000	$300,000
11. Effluent Equalization and Conveyance	$1,368,000	$0	$0	$1,368,000
12. Advanced Treatment	$1,374,000	$0	$0	$1,374,000
End-Use Facility				
New Outfall, UV Disinfection	$500,000	$0	$0	$500,000
SUBTOTAL	**$10,706,000**	**$1,730,000**	**$250,000**	**$12,686,000**
Mobilization @ 5%	$535,000	$87,000	$13,000	$635,000
Subtotal	$11,241,000	$1,817,000	$263,000	$13,321,000
Construction Contingency @ 30%	$3,372,000	$545,000	$79,000	$3,996,000
Subtotal	$14,613,000	$2,362,000	$342,000	$17,317,000
Sales Tax @ 7.7%	$1,125,000	$182,000	$26,000	$1,333,000
Subtotal	$15,738,000	$2,544,000	$368,000	$18,650,000
Engineering, Admin. & Legal @ 35%	$5,508,000	$890,000	$129,000	$6,527,000
Purchase Property for New Site	$1,200,000	$0	$0	$1,200,000
Total Capital Cost	**$22,446,000**	**$3,434,000**	**$497,000**	**$26,377,000**
Present Worth of O&M Cost (Relative)				$2,111,000
Total Estimated Present Worth				$22,380,000

SUMMARY

The following Table VII-21 shows a summary of the capital costs for the treatment plant options for the four remaining end use options. The summary shows that upgrading the existing plant has the lowest total capital cost for all four end use options. The capital cost required for a new SBR at the existing site is only slightly more than upgrading the existing plant. Considering the accuracy range of planning level cost estimates, these plant options are considered to have equal capital costs. There is a significant cost increase to build an SBR at a new site.

Of the end use options, Option 3 (River Enhancement) and Option 6 (Streamflow Augmentation) have a considerably lower capital cost than either discharging downstream or using Class A reclaimed water for poplar irrigation in conjunction with groundwater recharge.

Table VII-22 shows a present worth cost summary of the treatment plant and end use options. The present worth costs are determined for the period through the year 2025. The table shows both capital cost plus O&M present worth and total present worth which includes salvage values at the end of the period. The lowest present worth alternative is to build a new SBR at the existing site and either enhance the river to use reclaimed water for streamflow augmentation to allow continued discharge at the current location all year long. The present worth analysis shows that it is much more cost-effective to build an SBR rather than upgrade the existing plant.

Table VII-23 shows a list of advantages and disadvantages for the three treatment plant options. From this table, it is obvious that a new SBR should be built to replace the existing plant. However, it is not as clear whether or not to build the SBR at a new site. This decision will be made during the facilities planning portion of this project.

INSERT TABLE VII-21 CAPITAL COST SUMMARY

INSERT TABLE VII-22 PRESENT WORTH COST SUMMARY

INSERT TABLE VII-23 ADVANTAGES AND DISADVANTAGES

The most important element of this GSP is the selection of an enduse option for the treated wastewater. It is essential that DOE agree with the selected option so that this GSP can be approved and the project proceeds to the facility planning stage. The City and Gibbs & Olson have had numerous meetings with DOE concerning the river enhancement (No. 3) and stream flow augmentation (No. 6) options. DOE does not support either of these options. They will therefore be dropped from further consideration so that the planning process is not delayed. That leaves downstream discharge (No.2) and Class A RW to poplars with groundwater recharge (No. 4Aii) as the remaining end use options.

Class A RW to poplars with groundwater discharge is the preferred option for the following reasons:

- It has a lower capital, O&M and present worth cost than downstream discharge.
- It has beneficial reuse for poplar irrigation and groundwater recharge.
- It complies with the TMDL and Consent Decree by not having surface water discharge during low flow conditions.
- Since there is no surface water discharge during low flow conditions, there is no risk of additional regulatory restraints imposed by future TMDL's or efforts to protect salmon.
- Requires a relatively small amount of equalization storage.
- Has support from DOE, DOH and EPA.
- Establishes a reclaimed water infrastructure.

RECOMMENDED TREATMENT AND END USE OPTION

The recommended treatment option is to construct a new SBR plant at either the existing WWTP site or a new site. The facilities plan will present an alternative evaluation and recommendation of where the new SBR plant will be built. Design sizing of the SBR was presented earlier in this section. The SBR will be required to produce an effluent that has very low BOD_5, TSS, ammonia and nitrate concentrations. It must also be suitable for tertiary treatment to produce Class A reclaimed water. The advanced treatment train will consist of coagulation and filtering. A static mixer will be used to mix alum and polymer prior to filtering. The advanced filters will be continuous backwash sand filters. The disinfection method will be changed to UV light. The

Facilities Plan will present an evaluation of open and closed type UV systems. A small amount of chlorine will be kept on hand for process control.

The recommended enduse option is to produce Class A reclaimed water that will be used for poplar irrigation in conjunction with groundwater recharge. The poplar plantation will be bermed to allow reclaimed water to be stored temporarily if the ground is frozen during low flow conditions. The best soils for this application are the "Newberg" type because they have a high permeability and the chance that are underlain with clay is small. The Chehalis and Reed soil types would also work but would be better suited for the driest months of the year. 125 acres of trees is required for the expected year 2025 flows and nitrogen concentration discharged from the plant.

There are still numerous tasks to be completed in the Facilities Plan in order to further refine this option. They include:

- A comprehensive hydrogeological evaluation including the drilling of test wells to determine if there is indeed a clay layer beneath potential tree farm sites. Hydrogeological data to be collected includes depth to groundwater, depth to clay layer, estimate of groundwater movement and direction and suitability of sites for groundwater recharge.
- Evaluation of potential nitrate impacts to the groundwater will be done in accordance with the Permit Writer's Manual Chapter VIII.
- An analysis of transport and fate of BOD and ammonia in the reclaimed water.
- Complete agronomic analysis including method of irrigation, estimated effective crop depth, an irrigation protocol, leaching index and suitability of application sites.

SOLIDS HANDLING CAPACITY

The plant currently has inadequate drying bed capacity to properly dewater the biosolids after digestion. During most years, the sludge is withdrawn from the drying beds in a semi-dry form. This creates a lot of extra expense since the biosolids fees are based on the amount of wet tons that are trucked off the site. The DOE Criteria for Sewage Works Design (Orange Book) recommends that the drying beds be sized at 2 to 3 sf/person for covered beds. The City is currently using 20,000 sf of total bed capacity of 28,000 sf. 8,000 sf of drying beds are presently used for equipment storage because the underdrains in the drying beds are not working properly. The existing population is 8,671 that requires a drying bed capacity of 26,013 sf. So, even for current conditions, the City would need to restore 6,000 sf of drying bed capacity. The projected population in 2025 is 14,588 that would require restoration of 8,000 sf and building an additional 15,764 sf of drying bed capacity.

The total amount of sludge expected to be produced in the year 2025 is estimated to be approximately equivalent in pounds per day as 50% of the BOD_5 and TSS removed during the treatment process. The anticipated loading of BOD_5 in 2025 is 3,970 PPD on an annual average basis. The anticipated TSS loading in 2025 is 4,105 PPD on an annual average basis. The mass of the sludge to be produced in 2025 is calculated below based on an assumption of 90% removal of both BOD_5 and TSS.

$$
\begin{aligned}
\text{Sludge Mass} &= (BOD_5 \text{ x } 90\% \text{ x } 50\%) + (\text{TSS x } 90\% \text{ x } 50\%) \\
&= 3{,}970 \text{ PPD x } 0.45 + 4{,}105 \text{ PPD x } 0.45 \\
&= 3{,}634 \text{ PPD}
\end{aligned}
$$

At a concentration of 1.0% solids this amounts to 43,573 GPD or 15.9 MG per year.

ANAEROBIC DIGESTER SIZING

Recommended design criteria for an anaerobic digestion sludge stabilization facility for the Chehalis WWTP is shown in Table VII-24 and is based on utilizing an SBR treatment process and thickening only WAS prior to sending the solids to the digester facility.

TABLE VII-24
ANAEROBIC DIGESTER DESIGN CRITERIA
FOR SBR WWTP WITH WAS SLUDGE

Design Parameter	1998	2025
Population	8,671	14,588
Solids Wasted to Digesters (lbs/day)	2,160	3,634
Solids Wasted to Digesters (lbs/cap/day)	0.25	0.25
Volatile Solids Content of Solids Wasted (%)	70	70
VSS Wasted to Digesters (lbs/day)	1,512	2,544
Solids Concentration after Thickening (%)	3	3
Sludge Volume (gpd)	8,632	14,523
Minimum Sludge Retention Time(days at 95° F)	20	20
VSS Reduction (%)	50	50

Based on a VSS loading rate of 0.10 lbs/day-ft^3 the ultimate digester capacity required is 25,440 ft^3 or 190,304 gallons. Based on the minimum sludge retention time of 20 days, the ultimate digester capacity required is 290,460 gallons (38,829 ft^3). Based on volume per capita of 2.6 ft^3/person, the ultimate digester capacity required is 37,929 ft^3 (283,727 gallons). The volume per capita sizing guideline is often utilized as a "first-cut" when limited or no solids data exists, and is therefore more conservative than the VSS loading criteria or the solids retention time (SRT) sizing criteria. All of the sizing criteria above are for heated and mixed digester tanks operated at approximately 95°F such as the primary digester currently utilized by the City of Chehalis.

The City currently has two anaerobic digestion tanks, each with a volume of 158,230 gallons (21,152 ft^3). With a total of available volume of 316,460 gallons the existing anaerobic digesters are adequately sized to treat the projected 2025 sludge volume produced by an SBR plant.

The existing heat exchanger has an input capacity of 735,000 BTU/hr and can heat up to 17,790 gallons/day of digesting sludge which is 1.22 times the expected solids loading. A second heat exchanger is recommended to provide redundant heating capacity in the event of a heat exchanger failure.

SLUDGE TREATMENT AND HANDLING

INTRODUCTION

Sludge treatment or stabilization processes are the key to reliable performance of any wastewater treatment plant. These processes treat the solids generated in the treatment of the wastewater, converting them to a stable product for ultimate utilization or disposal. Sludge stabilization also reduces pathogens in the sludge, thus producing a safer and less odorous end product. Similar to the wastewater treatment options, there are several combinations of processes that can be used to properly treat, handle and dispose of sludge. The various options and combinations of processes are too numerous to discuss or even list. This section will consider four of the more common methods used by treatment plants of similar size and type to the Chehalis WWTP.

The Chehalis WWTP uses two anaerobic digesters to treat and thicken sludge produced in the primary and secondary clarifiers. After treatment in the digesters, the thickened sludge is transferred an aerated sludge storage basin prior to being pumped to covered drying beds for dewatering. The dried biosolids are then trucked to eastern Washington and applied as a soil amendment. The final product meets the requirements for a Class B biosolid.

BIOSOLIDS HANDLING REGULATIONS

This report only considers alternatives which can meet the 503 Regulations and the State Rule for Biosolids Management. Applicable federal regulations which govern the final use or disposal of biosolids are *"40 CFR Part 503 - Standards for the Use or Disposal of Sewage Sludge,"* which were enacted on February 19, 1993 and established standards, consisting of general requirements, pollutant limits, management practices, and operational standards, for the final use or disposal of sewage sludge generated during the treatment of domestic sewage in a treatment works. These rules were developed to meet the requirements of the 1987 Clean Water Act. The standards in the 503 Regulations are for sewage sludge applied to the land, placed on a surface disposal site, or fired in a sewage sludge incinerator. A summary of the relevant sections of the 503 Regulations is included in Appendix E. The summary is meant to provide a quick overview of the regulations and does not contain all of the requirements, exceptions or details of the regulations.

The Washington State Department of Ecology recently developed a new State Rule for Biosolids Management that applies to all wastewater treatment plants. The new state rule applies to facilities which produce biosolids or products derived from biosolids, and also to those who apply biosolids to the land, or own or manage land on which biosolids are applied. The new State Rule for Biosolids Management was enacted under the Washington Administrative Code (WAC), and is listed as Chapter 173-308 WAC - Biosolids Management.

SLUDGE TREATMENT AND HANDLING ALTERNATIVES

The four areas discussed for Chehalis are thickening, stabilization, dewatering and utilization. Each of the alternatives evaluated for biosolids treatment and handling at the Chehalis WWTP are listed below and described in that order.

- Thickening: Gravity, Gravity Belt, Rotating Drum, and Centrifuge.
- Stabilization: Aerobic Digestion, Autothermal Thermophilic Aerobic Digestion (ATAD), Anaerobic Digestion, Composting, Lime Stabilization.
- Dewatering: Drying Beds, Filter Press, Centrifuge.
- Utilization: Forest Land Application, Agricultural Land Applications, Land Reclamation, Transferring to the New Centralia WWTP for Stabilization and Utilization.

Thickening Alternatives

Thickening is often used in wastewater treatment to increase the solids concentration in a sludge stream and thereby reduce the volumetric loading to subsequent solids treatment and handling processes. This can significantly decrease the size requirement for equipment and tankage of the subsequent processes and allows them to operate more efficiently.

- Gravity Thickening: Sludge is concentrated by gravity induced settling and compaction of sludge solids. The process is very similar to that used in sedimentation/clarification basins. Gravity thickening provides two benefits, 1) solids concentration and 2) equalization and storage of sludges, which improve performance of subsequent processes. Gravity thickeners are well suited for thickening waste activated sludge (WAS). Influent WAS solids concentrations are typically 0.5 to 1.5%. Solids concentrations of up to 3 to 4% are achievable with the addition of polymer. However without polymer addition, only about 2% solids concentration can be reliably achieved. Aeration is desirable to reduce odors. Advantages and disadvantages of gravity thickening are summarized in Table VII-25.

TABLE VII-25 ADVANTAGES AND DISADVANTAGES OF GRAVITY THICKENING	
Advantages	Disadvantages
Simple operational theory Low operating cost Low operator attention required Provides some storage as well as thickening Conditioning chemicals not required Minimal power requirements	Relatively large tankage requirements for WAS

The Chehalis plant currently uses gravity thickening of digested sludge prior to sending it to the drying beds.

Gravity Belt Thickening: In gravity belt thickening (GBT), the solids concentration of a sludge increases as its free water drains by gravity through a porous horizontal belt. Successful GBT requires chemical conditioning, typically using a polymer. GBT is particularly suitable for thickening of WAS prior to further processing in a digester and for thickening stabilized biosolids before transportation for utilization.

WAS sludge can typically be thickened to concentrations of 4 to 8%. Increased operator attention is required with GBT due to the addition of a polymer and the mechanized equipment utilized in the process. Polymer dosage is typically 6-14 lbs/Ton. Table VII-26 shows advantages and disadvantages of the GBT process.

TABLE VII-26 ADVANTAGES AND DISADVANTAGES OF GRAVITY BELT THICKENERS	
Advantages	Disadvantages
Space requirements Control capability for process performance Relatively low capital cost Relatively low power consumption High solids capture & minimum polymer dosage High thickened solids concentrations	Maintenance requirements Polymer dependent Moderate operator attention required Odor potential High capital and O&M cost

- Rotary Drum Thickening: Rotary drum thickeners (RDT) operate in a manner similar to GBT units in that free water from a sludge drains through a porous media with sludge solids being retained on the media, and chemical/polymer conditioning of feed sludge is required to induce thickening. Typical RDT's utilize a rotating drum with wedge wires, perforations, stainless steel or polyester fabric as the porous media. An RDT typically rotates at 5-20 revolutions per minute (rpm) using a variable-speed drive unit. Washwater periodically flushes the inside and outside of the drum to clear the screen openings of solids. The success of RDT units in thickening WAS is variable and highly dependent on actual sludge characteristics. The potential of high conditioning chemical/polymer requirements can be a concern in RDT thickening due to floc sensitivity and shear potential in the rotating drum. Relative advantages and disadvantages of RDT's are presented in Table VII-27.

TABLE VII-27
ADVANTAGES AND DISADVANTAGES OF ROTARY DRUM THICKENERS

Advantages	Disadvantages
Space requirements	Polymer dependent
Low capital cost	Sensitivity to polymer type
Relatively low power consumption	Housekeeping
High solids capture	Moderate operator attention requirements
	Odor potential
	High Capital and O&M cost

- <u>Centrifugal Thickening:</u> Separation of the liquid-solid slurry in a centrifuge is similar to a rotary drum thickener, however, the applied force is centrifugal rather than gravitational and is typically between 500-3,000 times the force of gravity. Centrifuges are commonly utilized for thickening WAS. They can also be used to reduce the volume of stabilized biosolids to minimize costs associated with transportation for final utilization. Solid bowl conveyor centrifuge technology is most often utilized and has proven to be widely successful. As with GBT and RDT methods of thickening, chemical/polymer conditioning is typically utilized with centrifuges to provide better solids capture efficiencies. It is recommended that effective degritting, screening or grinding equipment precede the centrifuge to avoid plugging problems and excessive wear. WWTPs with centrifuge thickening generally have degritting or screening equipment within the headworks of the treatment plant.

Due to relatively high equipment capital costs and sophistication, centrifuges are most commonly found in medium to large WWTPs, (plants with design flows of 2 MGD or greater). Advantages and disadvantages of centrifugal thickening are presented in Table VII-2.

TABLE VII-28
ADVANTAGES AND DISADVANTAGES OF CENTRIFUGAL THICKENING

Advantages	Disadvantages
Space requirements Control capability for process control Effective for WAS Contained process minimizes housekeeping and odor considerations High thickened solids concentrations	High capital cost and power consumption Sophisticated maintenance requirements Best suited for continuous operation and high volume Moderate operator attention required

<u>Stabilization Alternatives</u>

The key to reliable performance of any wastewater treatment plant is the stabilization process utilized to treat the waste biosolids generated in the main wastewater treatment process. Stabilization processes convert the waste biosolids to a stable product for ultimate beneficial use or disposal. Stabilization processes reduce pathogen in the biosolids, thereby providing a safer and less odorous final product. The four most common stabilization processes used in the United States today are 1) aerobic digestion, 2) anaerobic digestion, 3) composting, and 4) lime stabilization. In recent years a variation of aerobic digestion, known as autothermal thermophilic aerobic digestion (ATAD), has begun to gain acceptance and is being utilized at a growing number of treatment plants.

- <u>Aerobic Digestion:</u> Aerobic digestion is a sludge stabilization process in which the biological oxidation of degradable organic solids is accomplished by microorganisms utilizing air. The process is similar to and is often considered a continuation of the activated sludge wastewater treatment process. Aerobic digestion is most commonly utilized in plants with design flows of less than 5 MGD.

  The operating temperature of an aerobic digestion system greatly affects process performance. One of the major disadvantages of aerobic digestion processes is the change in process efficiency that results from changes in operating temperature. There are three temperature zones of bacterial action that apply to aerobic digestion, they are:

- Cryophilic zone - liquid temperature is below 10° C (<50° F).
- Mesophilic zone - liquid temperature is between 10-42° C (50 - 108° F).
- Thermophilic zone - liquid temperature is higher than 42° C (>108° F).

Most aerobic digestion systems operate within the mesophilic range. The recently built existing secondary clarifier could be converted for use as an aerobic digester. Relative advantages and disadvantages of mesophilic aerobic digestion are listed in Table VII-29.

TABLE VII-29
ADVANTAGES AND DISADVANTAGES OF MESOPHILIC AEROBIC DIGESTION

Advantages	Disadvantages
Low initial capital cost for small plants	High energy costs associated with aeration/mixing equipment
Works well for digesting WAS	Less VSS reduction than anaerobic digestion
Supernatant less objectionable than anaerobic digestion supernatant	Reduced pH and alkalinity
Simple operational control	May experience foaming
Low odor potential with proper design/operation	Biosolids are typically difficult to dewater by mechanical means
Reduces total sludge mass	Performance adversely affected by cold temperatures

In order to meet 503 Regulations, a minimum solids retention time (SRT) of 90 days is recommended in order to reduce pathogens in the sludge and to provide storage during wet parts of the year.

- Autothermal Thermophilic Aerobic Digestion (ATAD): Since EPA published the

503 Regulations in February, 1993, increasing attention has been focused on a variation of aerobic digestion known as autothermal thermophilic aerobic digestion (ATAD). ATAD optimizes, through containment, the heat (energy) released by the biochemical oxidation of organic substances by microorganisms utilizing air, and uses the heat to operate the process in the thermophilic zone of biological activity, (temperatures greater than 42°C). Digestion tanks are typically covered to further increase the amount of heat retained within the system. Some ATAD systems being marketed have received an EPA rating as a Process to Further Reduce Pathogens (PFRP), and claim to be able to guarantee a Class A final biosolid in regards to pathogen concentrations. These systems may offer significant operational cost advantages over traditional aerobic digester systems which are only capable of producing a Class B final biosolid, due to reduced record keeping requirements. Nitrification is normally inhibited at the operating temperatures employed by ATAD systems. This inhibition of nitrification reduces the total oxygen requirement and eliminates pH depression, which can occur in standard aerobic digesters due to alkalinity consumption.
VSS concentrations in the range of 2.5 to 5.0% are required to provide sufficient energy to maintain the elevated digester operating temperature. This will require thickening of WAS prior to feeding to the digester. Digester tankage size requirements are decreased due to the reduction in sludge volumes being fed to the system.

ATAD is a relatively new digestion technology. The first ATAD facility went into service in the Federal Republic of Germany in 1977. Currently, there are approximately 40 operating ATAD systems in the world. Reported keys to proper ATAD performance include adequate thickening of the feed sludge, efficient aeration, sufficient tank insulation, good mixing and foam control. Table VII-30 lists some of the reported advantages and disadvantages of ATAD systems.

TABLE VII-30
ADVANTAGES AND DISADVANTAGES OF ATAD SYSTEMS

Advantages	Disadvantages
Ability to achieve Class A biosolid without external heat supply Reduced SRT required to achieve a given level of VSS reduction Good to excellent pathogen inactivation Stabilized biosolid that is reasonably dewaterable Process stability Ease of operation	Lack of long-term operational data Relatively small number of operating facilities Requirement for thickening of feed WAS Potential odor control requirements Requirement for foam control equipment Requirement for feed sludge and stabilized biosolids storage facilities

- Anaerobic Digestion: Anaerobic digestion is a relatively complex process which requires both proper design and careful operation and maintenance. It is one of the most widely utilized processes for stabilizing wastewater treatment plant sludge. Anaerobic digestion has been used for plants having average wastewater flows of less than 1.0 MGD to more than 200 MGD. Anaerobic digestion is most applicable to WWTP sludges that; 1) have a high concentration of biodegradable organics, 2) are free from any materials present in high enough concentrations to be toxic, and 3) are relatively uniform in characteristics from day to day. Primary sludges are the most easily anaerobically digested and yield the largest amount of methane gas per pound of sludge stabilized. WAS and other biological sludges are more difficult to digest, due to less biodegradable material being present, and because of the low TSS concentrations and difficulty in thickening above 3% without polymer addition. The City of Chehalis currently uses two anaerobic digesters to stabilize all of the sludge produced by the plant. A correctly operated anaerobic digester produces a high quality biosolid with minimal SRT. Table VII-31 lists advantages and disadvantages of anaerobic digestion.

TABLE VII-31
ADVANTAGES AND DISADVANTAGES OF ANAEROBIC DIGESTION

Advantages	Disadvantages
VSS destruction between 40-60 percent Low operational costs if methane gas produced is utilized for heat exchangers Stabilized biosolids suitable for agricultural use Good pathogen reduction Reduced total sludge volume Production of sludge free of objectionable odors when	Requires skilled operators May experience foaming Methane formers are slow growing, i.e., "acid digester" may occur Recovers slowly from upset High initial capital cost Cleaning is difficult (scum & grit)

fully digested Short SRT for adequate VSS reduction	Supernatant strong in BOD, COD, SS, and NH_3 Potential for mineral deposits in pipelines Safety issues regarding flammable gas

The plant currently uses two digesters for anaerobic digestion. Average HRT is 59 days based on a current sludge volume of 2,723 GPD.

- Composting: Composting is the aerobic decomposition by bacteria and fungi of the organic material in dewatered sludges, with the end result being a stabilized biosolid. The transformations which occur during composting are irreversible, and therefore a fully stabilized compost product cannot generate objectionable odors, even if wetted or stored for a long time period. Typically composting systems utilize the following steps:

    - Dewatered sludge is mixed with a bulking agent, such as wood chips to increase porosity, reduce the bulk moisture content and supply additional carbon.
    - Heat generated by microbial decomposition of sludge solids evaporates excess water and neutralizes many of the pathogens in the sludge.
    - The compost mixture is aerated for 15 to 30 days either by blowers or periodic remixing. This step provides oxygen, controls temperature and removes water vapor.
    - The bulking agent is recovered by screening for reuse.
    - Compost is cured for an additional time period to complete the stabilization process.

    Table VII-32 presents advantages and disadvantages of composting as a stabilization method.

TABLE VII-32
ADVANTAGES AND DISADVANTAGES OF COMPOSTING

Advantages	Disadvantages
High-quality, potentially salable product suitable for agricultural use	Requires 40-60 percent solids Requires bulking agent

Can be combined with other processes Low initial capital cost for some variations	Requires either forced air or turning May require significant land area Requires carbon source Potential for odors associated with incomplete stabilization Potential for high operational cost - power, labor, and/or chemical

- <u>Lime Stabilization:</u> The effectiveness of lime stabilization depends on maintaining the pH at a high enough level for a sufficient period of time to inactivate the microorganism populations in the sludge.

  This stops the microbial reactions that can otherwise lead to odor production and vector attraction. Lime stabilization can also inactivate viruses, bacteria, and other microorganisms that are present. Generally, stabilization is achieved if a pH of 12 is maintained for at least 2 hours. The effects of lime stabilization on some of the physical and chemical characteristics of wastewater sludges include:

  - A reduction of the VSS concentration of the sludge by 10-35%.
  - An increase in the total suspended solids (TSS) concentration due to the addition of inert solids and excess lime and the precipitation of dissolved solids.
  - A reduction in the nitrogen content of sludge because of the volatilization of ammonia.
  - An increase in the alkalinity of sludge.
  - A reduction of the mobility of heavy metals; they are precipitated as hydroxides.

  Lime stabilization consists of two main tasks; 1) lime handling, and 2) the mixing of lime and sludge. Lime handling includes receiving, storing, transferring and delivering lime to a lime and sludge mixing unit. Lime as either a slurry or in dry form is added to the sludge.

  Lime stabilization is sometimes used as either a backup for existing stabilization facilities, or as an interim sludge stabilization process. This is because lime stabilization can be started or stopped quickly. Table VII-33 lists advantages and disadvantages of lime stabilization methods.

TABLE VII-33 ADVANTAGES AND DISADVANTAGES OF LIME STABILIZATION	
Advantages	Disadvantages
Low capital cost Fairly easy operation Good as emergency or interim stabilization method	Chemical and labor intensive Volume of solids to be disposed of is increased pH drop after treatment can lead to odors and biological growth Increased operator attention required

Dewatering Alternatives

Dewatering of stabilized biosolids is similar in theory and practice to thickening of unstabilized sludges. The goal of dewatering is to reduce the volume of the stabilized biosolids which must be transported to final utilization. Three methods of dewatering were evaluated to determine their applicability to improving the Chehalis WWTP's biosolids handling facilities. The first method of dewatering is drying beds which is the method the plant currently uses. The other two methods of dewatering sludge are mechanical in nature and are centrifugal and belt filter press (BFP) dewatering.

A natural method of dewatering stabilized biosolids is a sludge-drying bed. Drying beds have been utilized for dewatering sludge for over 70 years, and have been utilized predominately at smaller treatment plants (average flows $\leq$ 2 MGD). This is the current method of dewatering being used in Chehalis. Stabilized biosolids are dewatered on asphalt/sand beds primarily by drainage and evaporation. Drainage consists of two components, 1) water is drained through the sludge into the sand and removed through the underdrains, and 2) decanting of the sludge supernatant layer. Drying beds can be covered to prohibit precipitation from adding water to a dewatering biosolid, however, evaporation can still occur if the drying bed is covered with a roof structure only. Table VII-34 shows advantages and disadvantages associated with sand sludge-drying beds.

TABLES VII-34 ADVANTAGES AND DISADVANTAGES OF SAND SLUDGE-DRYING BEDS

Advantages	Disadvantages
Low requirement for operator attention and skill Low electric power consumption Low sensitivity to sludge variability Low chemical consumption High dry cake solids contents Low capital cost for small plants if land is available, and lining & leachate control is not necessary	Lack of rational design approach for sound economic analysis Large land requirement Impact of climatic effects on design High visibility to general public Labor-intensive biosolids removal Real or perceived odor and visual nuisances

In order to provide adequate dewatering capabilities for the anticipated 2025 sludge production, an additional 18,764 sf of covered drying beds need to be constructed. It would be very difficult to find available space at the existing site for this purpose. Also, the existing drying beds need to be protected from floods. Currently, they are low enough that floodwater frequently enters the beds and saturates the drying sludge. The plant operations staff spends a considerable amount of time drying the sludge. A rototiller is used to turn the sludge often to promote drying.

The existing drying beds could be protected from floods by placing additional stop logs in the doorways. The existing channels that hold the stop logs could be lengthened so that the stop logs could reach an elevation of 180.5-feet. It will also be necessary to block off the vents that are at the bottom of the drying bed walls. The City has recently installed powered ventilators to help dry the sludge.

A sludge centrifuge can be used to either thicken or dewater sludge. The operation of a centrifuge in the dewatering mode is very similar to thickening which was discussed earlier in this section. The advantages and disadvantages of a centrifuge for dewatering are the same as shown previously in Table VII-28. Centrifuges can produce a cake with solids content of 20-30%.

Sludge can also be dewatered with a BFP. The BFP is similar to a GBT except that a press is used to apply pressure to the sludge which forms a cake. The solids content of the cake usually ranges from 15-25%. Polymer dosage is typically 8-14 lbs/Ton. The advantages and disadvantages of a BFP are the same as for a GBT and are shown previously on Table VII-26.

Ashbrook Corporation manufactures a piece of equipment that is suitable for both sludge thickening and dewatering. The unit is called a Klampress and uses two porous fiber belts which allow both gravity thickening and pressure filtration. This unit is well suited for Chehalis since only one piece of equipment is required to perform two functions.

Utilization Alternatives

Utilization alternatives evaluated for Chehalis' stabilized biosolids are two land application alternatives. One utilization alternative is to truck unstabilized biosolids to the new Centralia WWTP for stabilization and utilization. Land application is defined as any beneficial use project that applies biosolids to the land. These includes application of biosolids on tree farms, pasture land, and agricultural land, as well as, application of biosolids in large quantities to aid in reclaiming land such as old mining sites. The 503 Regulations control the type of application practice which can be practiced based on ten pollutant concentrations and the level of pathogen reduction achieved and documented during stabilization. Biosolids that meet both "clean" biosolid and Class A biosolid requirements can be land applied to any type of approved site without restriction. Biosolids that meet "clean" biosolid and Class B biosolid contain site restrictions to limit or omit human contact with the biosolid for a predetermined period of time.

The three utilization options that have been identified are: land application on forest land, land application on agricultural land and trucking to the new Centralia WWTP. Each of these options are discussed below.

- Land Application on Forest Land: Land application on forest land would require hauling and spray irrigation. The 503 Regulations allow only one application per year and cannot be applied during wet weather, which would cause runoff of sludge. Forest land required for continual application, based on a rough estimation of concentrations and applications rates, may be as high as 170 acres. This assumes a nitrogen concentration of 5% of total solids and acceptable annual loading rate of 390 lb/acre. Other limitations and loading rates are defined in the 503 Regulations.

- <u>Land Application on Agricultural Land:</u> Land Application on agricultural land includes the same options for the City. The 503 Regulations for land application on agricultural land are more stringent than forest land and would require more land depending on agricultural use.

  The City currently recycles all of the biosolids via agricultural land application. The dried biosolids are trucked to the application sites in a cake form. All of the current application sites are owned and managed by a contractor which accepts biosolids from numerous treatment plants in the south sound area and would be able to handle the entire projected biosolids quantity up through the year 2025.

- <u>Trucking to the New Centralia WWTP:</u> Trucking biosolids to the new Centralia plant for processing and utilization is also an option. The Centralia Facility Plan recommends lime treatment for sludge stabilization. The biosolids would then be turned over to a private contractor for transportation, management and ultimate utilization. If sludge were sent to the Centralia plant for processing and utilization, it is not required to be digested. The WAS would be thickened or dewatered and stored at the Chehalis plant and trucked to the Centralia plant. This would avoid having to upgrade the existing digesters. A GBT would be best suited for thickening and a BFP would be best suited for dewatering. Since equipment and polymer costs are similar for GBTs and BFPs, it is more cost-effective to dewater the sludge to further reduce the amount of water that is hauled with the sludge.

EVALUATION OF SOLIDS HANDLING ALTERNATIVES

This part of the report will evaluate the alternatives for solids handling that include thickening, dewatering and utilization. The amount of sludge generated is expected to increase dramatically because of the increased solids removal efficiency of a new plant, and additional population growth. It is essential that the WAS be thickened prior to digestion so that the existing digester capacity is adequate for future conditions. It is necessary to thicken to at least 3% which would reduce the sludge volume by a third. The plant currently uses gravity thickening for digested sludge and it works very well. Unfortunately, the basin that is used for sludge storage and thickening is not protected from floods and would not be used with a SBR plant. Gravity thickening of the digester feed sludge for a plant this large is not feasible due to the large tankage that is required. The other three options for thickening are all mechanical and are centrifuge, rotary drum and gravity belt filter (GBT). Centrifuges are very expensive and are more suited for larger plants. The rotary drum thickener is well suited for the Chehalis plant and has a reasonable capital cost with minimal polymer requirements. The GBT is also well suited for the Chehalis plant because it has a reasonable capital cost and minimal polymer requirements. The Ashbrook "Klampress" can be used to both thicken and dewater sludge.

The current stabilization method is anaerobic digestion in two existing digesters, one of which that is equipped with heating and mixing equipment. The easiest and least expensive method of sludge stabilization for the anticipated future sludge volume is to use both of the digesters in the anaerobic mode with heating and mixing provided. Anaerobic digestion produces a Class B biosolid that meets the 503 regulations with an HRT of only 20 days. The existing heat exchanger is adequate to supply heat to both digesters but would not allow for any redundancy. Both digesters also need to be equipped with new covers and gas mixing equipment. The existing floating covers on the anaerobic digesters are in need of replacement. The new covers can be the floating or fixed type.

The recommended method of sludge stabilization is anaerobic digestion. A new heat exchanger is required to provide a heat source in the event of heat exchanger failure. A new building would be required to house the new heat exchanger since there is inadequate space for a second unit in the present location. The preliminary recommendation for the digester covers is to use the fixed type covers. However, a final recommendation will not be presented until the Facilities Plan is prepared. If it is decided to transport unstabilized sludge to Centralia for lime treatment, it will not be necessary to upgrade the digesters since they will only be used as sludge storage tanks.

The three alternatives for sludge dewatering are drying beds, centrifuge and belt filter press (BFP). The Chehalis plant has used drying beds for sludge dewatering for decades. However, as the sludge volume has increased over the years, the final product is becoming wetter and wetter. There is currently a shortage of drying bed capacity and it will only get worse as the amount of sludge increases. It is impractical to construct any more drying beds at the existing site because they take up so much room. Space at the existing site is at a premium and any construction needs to be above elevation 182.5' for flood protection. Sludge dewatering can also be accomplished with a centrifuge. However, the centrifuge is more suitable for larger plants. The belt filter press is the best piece of equipment to use for dewatering for a plant the size of Chehalis. A BFP would produce a cake with a solids content of at least 15%. This would reduce the volume of stabilized biosolids for final utilization. And since the plant needs to thicken and dewater the sludge, it makes sense to use the Klampress that can perform both operations with a single unit.

The three utilization options are to land apply to forest land, land apply to agricultural land and to truck unstabilized sludge to the new Centralia plant for processing and ultimate utilization. Land application to forest land is feasible for Chehalis since there is suitable forests within 30 miles of the plant. However, this option is very labor intensive due to trucking and application requirements. The dried biosolids must be mixed with water at the application site to allow the biosolids to be sprayed on the trees. The permitting process for a new application site would also be very difficult given the current political conditions in Lewis County. The best alternative for biosolids utilization is to continue to use the DOE permitted sites that are operated by the biosolids contractor. This method of biosolids utilization has a reasonable cost and is relatively easy for the plant operations staff. However, given the current political situation in Lewis County with regard to biosolids application sites, the City should consider partnering with Centralia for sludge stabilization and utilization.

RECOMMENDED SLUDGE TREATMENT AND UTILIZATION

In order to determine the best method of sludge treatment and utilization, a present worth analysis must be completed. This will be done as part of the Facilities Plan that will be prepared in 2001. The preliminary recommendation in this report is use a Klampress for both thickening and dewatering. The WAS would be thickened prior to digestion in two anaerobic digesters. The stabilized biosolids would then be dewatered and stored at the existing site prior to trucking to the biosolids contractor's application sites. Two or three of the existing drying beds should be converted for use as sludge storage pads for the dewatered biosolids cake. The converted beds should also be retrofitted to make sure that they are safe from future floods. Recommended improvements to the digesters are to install two new fixed covers with gas mixing equipment and a new heat exchanger and building to house it. Modifications to the sludge pumping equipment will also be required. The estimated capital cost for these recommended improvements is $2.34 million including mobilization, contingency, sales tax, engineering, administration and legal costs. These costs are shown in the cost tables previously presented in this section. However, if the present worth analysis shows that it would be more cost effective to thicken or dewater raw sludge and truck it to Centralia for processing and utilization, these modifications will not be necessary.

SECTION VIII

FINANCIAL CONSIDERATIONS

INTRODUCTION

This section will focus on possible funding considerations for the City of Chehalis, Napavine and LCSD No. 1 Regional WWTP upgrade. This section will also present a preliminary look at the potential impact to sewer rates for completion of the proposed work. A project of this magnitude will be extremely difficult to implement without grant and low interest loan assistance from state and/or federal funding agencies. Even with optimum financial assistance, Chehalis, Napavine and LCSD No. 1 will experience substantial increases in their sewer rates as a result of the proposed project. The most likely sources of funding for this project will be:

1. The Washington State Department of Ecology (DOE), Centennial Clean Water Fund Program (CCWF).
2. The Washington State DOE, State Revolving Loan Fund (SRF).
3. Department of Community, Trade and Economic Development (CTED) Block Grant Program (CDBG).
4. The Department of Community, Trade and Economic Development (CTED), Public Works Trust Fund (PWTF) program.
5. U.S. Department of Agriculture, Rural development (USDA-RD).
6. U.S. Department of Agriculture, Forest Service (FS)
7. Revenue Bonds, local rates and connection charges.

DEPARTMENT OF ECOLOGY (DOE)

The DOE Water Quality Program administers two funding programs that provide grants and low-interest loans to projects that improve and protect water quality. The funding programs are the Centennial Clean Water Fund (CCWF), which provides grants or low-interest loans and the State revolving Fund (SRF), which provides low-interest loans only.

For planning projects, the Centennial Grant Program provides 75% grants. In February 1998, the City submitted a grant application to DOE for the General Sewer Plan and related environmental work and a loan application for the Facility Plan. The City was not successful in obtaining funding for any of the work. For construction projects, the Centennial grant program provides grant funds only if current sewer rates are above the hardship level (1.5% of MHI). Design costs are not eligible for grant funding but are eligible under DOE's SRF loan program.

In both the grant and loan programs, applications from throughout the State are accepted in February of each year. Each project is assigned priority rating points by DOE personnel and all projects are prioritized. Projects receiving the highest priority points, and falling within the budgets available to DOE will be given a grant and/or loan offer. In recent years, the projects that have received the highest priority points are projects that address the states highest priority water quality protection and water pollution control needs.

Based on the TMDL Study, it appears the need to protect water quality in the Centralia Reach is one of the State's highest priority projects. Therefore, it is recommended that the City apply for a DOE loan in February of 2002 for design of the recommended WWTP improvements. Subsequent grant/loan funding applications can then be made to DOE in February 2003 for construction of the WWTP improvements. Because this project is required to meet new DOE water quality requirements it is anticipated that the construction costs for the entire project are eligible for DOE grant funding. The final determination as to which parts of the project are grant eligible will be made by DOE as part of their approval of the final plans and specifications. It should be noted that DOE does not fund commercial/industrial flows in excess of 30%. This Plan projects a C/I flow component of 39.7%.

The City submitted a grant application in February 1999 for additional water quality testing for metals and was successful. The water quality testing is the water effects rate (WER) Study that is discussed in Section III of this report.

The Centennial grant program will only fund improvements for existing capacity while the Centennial loan program will fund existing capacity plus an allowance of 10% for growth. The

SRF loan program will fund residential capacity for a 20-year period. Until DOE released the TMDL Study, Chehalis has focused on their I/I removal program to reduce high flows to allow for growth. Therefore, none of the recommended improvements are necessary to provide capacity for growth and all WWTP improvements should be 100% eligible.

The State Revolving Fund (SRF) includes federal and state funds. Since federal funds are involved through the Environmental Protection Agency (EPA), projects must comply with all the federal requirements as has been required in the past for projects receiving an EPA grant. This will include completion of a NEPA Environmental Report.

Currently the general terms of loans issued under the SRF program are as follows:

> All 6-20 year loans have an interest rate of 1.5%, and loans to be repaid in 5 years or less shall be 0.5 percent interest. If a community's monthly residential sewer rates are at or above 1½% of the median household income (MHI) then DOE can adjust interest rates and/or provide grant funding in an attempt to keep rates at or below this hardship level.

Under the SRF program applications are accepted in January and February of each year. DOE then prepares an Intended Use Plan (IUP) that identifies projects that may be funded. This plan is generally finalized by the first part of September and the agency is then in a position to begin making loans.

Chehalis' project will qualify for a SRF loan, and the City should apply for an SRF loan to help pay the cost of the recommended improvements. The SRF loan can be used by the City as the City's matching funds for a PWTF loan. By obtaining an SRF loan the City may qualify for a PWTF loan with a 1% interest rate. To be eligible for the SRF program, the Facilities Plan must be approved in order to meet the Federal requirements.

PUBLIC WORKS TRUST FUND (PWTF)

This program offers low interest loans to communities for a wide variety of projects. PWTF loans may be used only for the repair, replacement, rehabilitation, reconstruction or improvement of eligible public works systems to meet current standards for existing users. Trust fund loans are not designed to finance growth related project expenditures. As previously discussed, all of the proposed WWTP improvements are required to meet today's standards, to provide reliability and to protect water quality in the Chehalis River. To be eligible for the program, the community must have an approved Capital Improvement Plan (CIP) identifying its public works needs and how they may be financed and must have implemented the ¼ percent real estate excise tax. Chehalis complies with both of these threshold requirements.

In 1998, $53 million was available statewide. The program has a ceiling of $10 million per biennium and provides a payback period of 20 years for major projects. The interest rates are either 0.5 percent, 1 percent, or 1.5 percent depending on the amount of matching City funds committed toward the project. A 5 percent City match qualifies the community for a 1.5 percent loan; a 10 percent City match qualifies for a 1.0 percent loan; and a 15 percent matching share qualifies for a 0.5 percent loan. As previously mentioned, the City can use other loan or grant funding as their share of matching funds.

Applications are accepted in the spring of each year (usually March or April) and projects that receive a loan offer can expect the funding to be available in spring of the following year (usually May or June). A project funded by PWTF must begin no later than October 1st following legislative approval and be completed within 48 months of the loan agreement. It is anticipated the City will apply in 2002 for a PWTF loan for design and some of the early construction work. The City should also apply for a second loan (construction only) in 2003. The total of these two loans will take the total loan amount, for the biennium to $10 million. A third loan application can then be made in 2004 which will also be for $10 million but will be for the next biennium.

PWTF also has monies available for pre-construction design, engineering and right-of-way acquisition that are not subject to the one-year application delay. The major difference between this money and the traditional PWTF loans is the payback period is set at 5 years and the maximum amount available is $1 million. The application open and monies are available within a few months. If the City's application to DOE for design is not funded in 2002, then the City should apply for the design costs under this PWTF program.

COMMUNITY DEVELOPMENT BLOCK GRANT (CDBG)

The Department of Community, Trade and Economic Development (CTED) administers a grant program (The Washington State Community Block Grant (CDBG) program) which has been established "to enhance the quality of life for low-and moderate-income residents and as a result, benefit the entire community."

The CDBG grant program may be used to fund projects in five categories as follows:

1. Housing
2. Economic Development
3. Community Facilities
4. Public Facilities
5. Comprehensive (projects with activities in at least two of the other categories).

Currently, Chehalis is not included in the list of communities maintained by CDBG as a community with a high percentage of low- and moderate-income (LMI) families. The City has been successful in obtaining CDBG funding for projects that are specifically designed to benefit all or mostly LMI families, but the WWTP proposal benefits the City as a whole and therefore is not eligible for CDBG grants.

In 1996, approximately $8 million in CDBG grant funds were distributed for projects on a competitive basis. The maximum funding that projects such as Chehalis' can receive in any given year is $750,000. Unless a separate work item that mostly benefits LMI families can be identified, it is not recommended that a CDBG application be submitted.

UNITED STATE DEPARTMENT OF AGRICULTURE (USDA)-RURAL DEVELOPMENT (RD)

RD (formerly known as Farmers Home Administration, FmHA and RDA) primarily provides loans for a variety of projects to rural communities at lower interest rates than can be obtained through the sale of revenue bonds. They do have a grant program, but the procedure for qualifying for a grant is complicated and is determined on a case by case basis, generally after all other funding sources are in place. The threshold determination for qualifying for a grant is tied into the amount that residential customers are paying for debt service, the amount of grant money obtained from all funding sources, and the amount other communities with similar sewage facilities are paying for their service. Because of the high cost of the proposed project, the City may be eligible for up to 75% grant funding if the money is available.

RD accepts applications throughout the year and funds projects on a first come, first serve basis up to their annual budget. Since the federal fiscal year begins in October, there is generally more money available for loans at this time. RD will work with a community to help put a funding package together. However, they prefer to see other funding sources in place prior to making their commitment.

Currently, revenue bonds sold through RD have a 30 to 40 year term at approximately 4.5% annual interest. RD requires a coverage factor amounting to one year's loan payment over a 10-year period. It is recommended that Chehalis pursue funding from DOE and PWTF before considering RD. One draw back with using RD funding is the 30 to 40 year term. This term generally exceeds the design life of a treatment plant and, although the annual payments will be lower, the total amount repaid for the money borrowed is much higher. If the City is unable to obtain funding from DOE and PWTF, then RD may be considered as a potential alternative funding source. The City may want to consider seeking RD funding concurrently with the effort to obtain DOE and PWTF funding. By doing so, a potential delay in the schedule may be avoided.

U.S. DEPARTMENT OF AGRICULTURE, FOREST SERVICE (FS)

The U.S. Forest Service provides rural communities within 100 miles of a national forest and have 15% dependency on natural resource-based industries with monies for infrastructure development.

Three different programs, each with specific goals, are available. They are: the Northwest Economic Adjustment Initiative (the President's Forest Plan), Economic Recovery Program, and the Rural Development Program. The goals of the program range from development of new economic opportunities for high skilled jobs to organization of action teams and community planning for projects that are linked to natural resources.

In March 1998, the City submitted an application to fund a portion of the current planning effort through the Washington Cities Economic Revitalization Team (WA-CERT) process that accepts applications throughout the year. To date, the City has been unsuccessful in obtaining any funding from this source.

REVENUE BONDS

The least desirable option to the City of Chehalis is to sell revenue bonds on the open market. Currently, such bonds are selling for about five percent (5%) interest over 20 years and require a coverage factor of about 40%. The sale of revenue bonds should only be considered as a viable funding option for these projects as a last report.

OTHER POSSIBLE SOURCES

The Infrastructure Assistance Coordinating Council (IACC) has developed a directory of funding sources for all communities. The Infrastructure Assistance Directory outlines other possible funding alternatives. City Planners should consult this directory and gain the assistance of expert financial advisers while developing a funding package for projects outlined in the GSP. A copy of the directory can be obtained by calling (360) 586-765 or visiting the website:

http://www.wsdot.wa.gov/eesc/environmental/FSDatabase.htm.

SEWER RATES

The new WWTP could cost about $25.7 million and have an additional O&M cost of $165,000 per year plus debt service. The City will need to aggressively seek grant and low interest loan funding to implement this project. One of the first things that funding agencies look at is the currently monthly charge for sewer service. The rule of thumb is that rates need to be at least 1.5% percent of the median household income (MHI) for the community to qualify for hardship funding. For Chehalis the 1998 MHI used by DOE is $31,226/year which results in a hardship level sewer rate of about $39.00 per month. Currently, typical rates for single-family residences are $41.41 every month. The City increased sewer rates to the anticipated year 2000 MHI level effective January 1, 1999. This should make it easier to get grant funding when applying for construction funds and will also allow a reserve fund to begin accumulating to help pay for the project. Raising rates early in the project will send a definite signal of funding agencies that the City is complying with all aspects of their obligation to meet the requirements of the TMDL and subsequent NPDES permit and Consent Decree.

FUNDING OPTION NUMBER 1

The first funding option assumes that only loan money is available for the City. If rates are increased early in the project, then the revenue that is generated can be used to help offset future costs and thus reduce the amount of loan funds needed for the project. Prior to completing the work, rates must be raised to generate revenue from the sewer customers that is adequate to cover both debt service on the loan portion of the financing package plus operation and maintenance (O&M) costs of the treatment system. Each entity must also fund O&M costs for the collection system through rates. For Chehalis this means a rate increase to fund a continued I/I removal program.

Appendix F contains a series of tables that show an example for estimating future sewer system costs and revenues based on funding $25.7 million using low interest loans and city funds. Table 1 shows the anticipated funding sources and the loan amounts. Table 2 shows the 1998 sewer budget for treatment costs only (i.e., it does not include collection system O&M nor I/I removal costs). The 1998 costs are increased over time at 3% per year. In 2002, a preliminary estimate of debt service has been added to the annual O&M cost to start payments for the first DOE loan. The remaining debt service has been added to the cost table in the year 2007. This assumes funding is obtained as shown in Table 1 and the new improvements will be completed by the year 2006.

Table 3 presents a population projection based on an annual growth rate of only 0.7 percent per year which is used for loan amortization calculations. Using this reduced growth rate gives a conservative estimate of needed rate increases in case the projected growth rate in Chapter V does not happen. Table 4 shows predicted rate increases that will be required to cover increased costs for treatment only.

Table 5 shows the estimated revenue that will be generated and the average monthly sewer bill for residential, multifamily and commercial customers. Also shown on Table 5 are the costs to Napavine and LCSD No. 1 for treatment. This table is for treatment plant costs only and does not include the costs of any upgrades or O&M for any of the entity's collection system. The rate increases have been set so that a reserve account can be established. This account will be used to help pay for costs incurred over the next 8 years it takes to implement the project and will then be used to make the loan payments and pay for the increased O&M after the project is complete. Table 6 shows the cash flow balance for the loan-only funding scenario.

FUNDING OPTION NUMBER 2

The second funding option assumes that the project will be funded with a 50% construction grant and low interest DOE and PWTF loans. As with the first option, each entity is responsible for funding collection system upgrades and O&M through rates. The tables for this funding scenario are labeled 1A through 6A and contain the same information as table 1 through 6 for the first funding scenario.

The impact to sewer rates for both of these funding options is shown below as Table VIII-1. These projections are very preliminary and are not based on a specific recommended alternative. They will be refined after the GSP is approved and a preferred alternative has been established.

At that point, each entity can review their individual rate structure. There are also ownership issues that must be resolved with regard to capacity allocation of the new plant.

CITY OF CHEHALIS COLLECTION SYSTEM FUNDING

Appendix F also contains a series of six tables that identify the cost to operate and maintain Chehalis' Collection System and fund future I/I removal projects. Table 2 shows that $408,000 (increased each year for inflation) will be budgeted for long term (yearly) I/I removal projects. The work (as identified in Section VI) is scheduled to be completed over a 40-year period.

Table 3 presents the same population projection described earlier and Table 4 shows the predicted rate increase that will be required to cover the collection system O&M and I/I removal cost.

Table 5 shows the estimated revenue generated and Table 6 shows that an I/I removal reserve account will accumulate approximately $1.34 million by the year 2010. This reserve will be used to fund "additional" I/I removal projects over-and-above the annual work.

Because this funding is independent of the treatment costs, and is scheduled over a 40-year period, the annual 3% rate adjustment is not required to fund the collection system cost. Instead, after 2006, a periodic rate adjustment is anticipated.

Table VIII-1 shows the rate adjustments that are needed to fund treatment O&M costs and a $25.7 million treatment plant project cost, as well as continual O&M and I/I removal for the Chehalis Collection System.

TABLE VIII-1
POTENTIAL MONTHLY SEWER RATES

	2000		2005		2010	
	100% Loan	**50/50**	**100% Loan**	**50/50**	**100% Loan**	**50/50**
CHEHALIS						
Treatment	$ 25.55	$ 23.81	$ 29.60	$ 23.81	$ 34.34	$ 26.16
Collection	$ 15.86	$ 15.86	$ 18.40	$ 18.40	$ 21.31	$ 21.31
Total	$ 41.41	$ 39.67	$ 48.00	$ 42.21	$ 55.65	$ 47.47
NAPAVINE						
Treatment	$ 25.04	$ 23.34	$ 29.03	$ 23.34	$ 33.65	$ 25.67

Collection	To Be Determined					
Total	To Be Determined					
LCSD NO. 1						
Treatment	$ 25.04	$ 23.34	$ 29.03	$ 23.34	$ 33.65	$ 25.67
Collection	To Be Determined					
Total	To Be Determined					

SECTION IX

IMPLEMENTATION

SUMMARY

Adoption of this report by the Cities of Chehalis and Napavine and LCSD No. 1 will initially accomplish the following five major elements:

1. It develops a plan to upgrade the WWTP to meet the requirements of the TMDL Study, the Consent Decree and the new NPDES permit.

2. It presents collection system interceptor routes and lift station locations required to serve the future sewer service area.

3. It establishes a sewer service area with a projected wasteload equivalent population of 24,180 people in the year 2025.

4. It presents modifications to the collection systems to reduce I/I from the pipes in the worst condition.

5. It presents a basis for minimum design and construction standards for all sewer work within the sewer service area.

The following is a proposed schedule for key project elements:

Activity	Month
1. City of Chehalis increases sewer rates to 1.5% of median household income (MHI)	January 1999
2. Gibbs & Olson submits draft General Sewer Plan (GSP) to local agencies	January 19, 1999
3. Chehalis submits draft GSP to DOE for review	March 30, 1999
4. City and agencies review GSP and issue comments	April – June 1999
5. Gibbs & Olson finalizes GSP	February 2001

Activity	Month
6. City and agencies approve GSP	April 2001
7. Gibbs & Olson submits draft Facilities Plan (FP) to the local agencies and DOE for review	Summer 2001
8. City and agencies review FP and issue comments	Fall 2001
9. Gibbs & Olson finalizes FP	Winter 2001
10. City and agencies approve FP	January 31, 2002
11. City submits loan application to DOE for WWTP design	February 2002
12. City submits loan application to Public Works Trust Fund (PWTF) for construction funds	March 2002
13. City receives loan from DOE for WWTP design	August 2002
14. City submits second loan application to PWTF for construction	March 2003
15. City receives loan from PWTF	May 2003
16. City prepares draft plans and specifications	February 2002 – August 2003
17. City and agencies approve plans and specifications	September 2003 – January 2004
18. City submits grant application to DOE	February 2004
19. City submits third loan application to PWTF	March 2004
20. City advertises and awards construction	February – April 2004
21. City receives second PWTF loan	May 2004
22. City receives DOE grant	August 2004
23. City begins construction	May 2004
24. City receives third PWTF loan	May 2005
25. City completes construction	December 2006
26. City completes one year certification and fulfills obligations under the Consent Decree	December 2007

Appendix B
Pe Ell Sewer Inventory

Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
A	Interceptor	A2	A1	60	10	Concrete	0.0040	0.90	1972	0.114	Soil	1
A	Interceptor	A3	A2	140	10	Concrete	0.1036	4.60	1972	0.265	Soil	1
A	Interceptor	A4	A3	440	10	Concrete	0.0068	1.18	1972	0.833	Soil	4
A	Interceptor	A5	A4	413.99	10	Concrete	0.0040	0.90	1972	0.784	Soil	4
A	Interceptor	A6	A5	405.19	10	Concrete	0.0040	0.90	1972	0.767	Soil	4
A	Interceptor	A7	A6	360	10	Concrete	0.0040	0.90	1972	0.682	Soil	3
A	Interceptor	A8	A7	324	10	Concrete	0.0040	0.90	1972	0.614	Soil	3
A	Interceptor	A9	A8	325	10	Concrete	0.0040	0.90	1972	0.616	Soil	3
A	Interceptor	A10	A9	325.02	10	Concrete	0.0040	0.90	1972	0.616	Soil/Asphalt	3
A	Interceptor	A11	A10	103.44	10	Concrete	0.0040	0.90	1972	0.196	Concrete	1
A	Interceptor	A12	A11	350	10	Concrete	0.0040	0.90	1972	0.663	Asphalt Pavement	3
A	Interceptor	A13	A12	283.28	10	Concrete	0.0040	0.90	1972	0.537	Asphalt Pavement	2
E	Interceptor	E1	A13	330	10	Concrete	0.0040	0.90	1972	0.625	Asphalt Pavement	3
E	Interceptor	E2	E1	260	10	Concrete	0.0040	0.90	1972	0.492	Gravel	2
E	Interceptor	E3	E2	72.04	8	Concrete	0.0040	0.50	1972	0.109	Gravel/Concrete	1
E	Interceptor	E4	E3	149.21	8	Concrete	0.0040	0.50	1972	0.226	Concrete/Asphalt	1
E	Interceptor	E5	E4	149.21	8	Concrete	0.0040	0.50	1972	0.226	Concrete/Asphalt	1
E	Interceptor	E6	E5	150.51	8	Concrete	0.0040	0.50	1972	0.228	Soil	1
E	Interceptor	E7	E6	150.51	8	Concrete	0.0040	0.50	1972	0.228	Soil	1
F	Interceptor	F1	PS1	200	8	Concrete	0.0040	0.50	1972	0.303	Asphalt Pavement	2
F	Interceptor	F2	F1	298.52	8	Concrete	0.0040	0.50	1972	0.452	Asphalt Pavement	3
F	Interceptor	F3	F2	77.25	8	Concrete	0.0068	0.65	1972	0.117	Asphalt Pavement	1
F	Interceptor	F4	F3	226.81	8	Concrete	0.0125	0.88	1972	0.344	Asphalt/Gravel	2
F	Interceptor	F5	F4	162.2	8	Concrete	0.004	0.50	1972	0.246	Gravel	1
F	Interceptor	F6	F5	391.98	8	Concrete	0.004	0.50	1972	0.594	Gravel	3
F	Interceptor	F7	F6	345.45	8	Concrete	0.0066	0.64	1972	0.523	Gravel	3
F	Interceptor	F8	F7	240.31	8	Concrete	0.0064	0.63	1972	0.364	Gravel	2
B	Line A	B1	A1	130	8	Concrete	0.0040	0.50	1972	0.197	Gravel	1
B	Line A	B2	B1	150	8	Concrete	0.0481	1.73	1972	0.227	Gravel	1
B	Line A	B3	B2	375.36	8	Concrete	0.0040	0.50	1972	0.569	Gravel	3
B	Line A-1	B3.1	B3	251.03	8	Concrete	0.0040	0.50	1972	0.380	Gravel	2
B	Line A	B4	B3	382.97	8	Concrete	0.0150	0.97	1972	0.580	Gravel	3
B	Line A	B5	B4	375	8	Concrete	0.0070	0.66	1972	0.568	Gravel	3
B	Line A	B5.1	B5	230.78	8	Concrete	0.0090	0.75	1972	0.350	Gravel	2
B	Line A	B5.2	B5.1	300	8	Concrete	0.0100	0.79	1972	0.455	Gravel	3
B	Line A	B5.3	B5.2	92.63	8	Concrete	0.0050	0.56	1972	0.140	Gravel	1
B	Line A	B5.4	B5.3	439.56	8	Concrete	0.0100	0.79	1972	0.666	Gravel	4

Appendix B
Pe Ell Sewer Inventory

Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
B	Line A	B5.5	B5.4	280	8	Concrete	0.0040	0.50	1972	0.424	Gravel	2
B	Line A-4	B5.5.1	B5.5	250	8	Concrete	0.0040	0.50	1972	0.379	Gravel	2
B	Line A-4	B5.5.2	B5.5.1	260	8	Concrete	0.0040	0.50	1972	0.394	Gravel	2
B	Line A	B5.6	B5.5	145.1	8	Concrete	0.0040	0.50	1972	0.220	Gravel	1
B	Line A	B5.7	B5.6	234.9	8	Concrete	0.0040	0.50	1972	0.356	Gravel	2
B	Line A-2	B6	B5	370	8	Concrete	0.0070	0.66	1972	0.561	Gravel	3
B	Line A-2	B7	B6	149.3	8	Concrete	0.0070	0.66	1972	0.226	Gravel	1
B	Line A-2	B8	B7	355	8	Concrete	0.0050	0.56	1972	0.538	Gravel	3
B	Line A-2	B9	B8	318	8	Concrete	0.0050	0.56	1972	0.482	Gravel	3
B	Line A-2	B10	B9	365.9	8	Concrete	0.0050	0.56	1972	0.554	Gravel	3
B	Line A-2	B10.1	B10	201.2	8	Concrete	0.0050	0.56	1972	0.305	Gravel	2
B	Line SA-2	B11	B10	185.5	8	Concrete	0.0040	0.50	1972	0.281	Concrete	2
B	Line SA-2	B12	B11	112.16	8	Concrete	0.0040	0.50	1972	0.170	Concrete	1
B	Line SA-2	B12.1	B12	296.91	8	Concrete	0.0040	0.50	1972	0.450	Concrete	3
B	Line SA-2	B12.1.1	B12.1	247.62	9	Concrete	0.0040	0.50	1972	0.422	Concrete	2
B	Line SA-2	B12.2	B12.1	290.45	8	Concrete	0.0040	0.50	1972	0.440	Concrete	3
B	Line SA-2	B12.3	B12.2	237.8	8	Concrete	0.0080	0.71	1972	0.360	Soil	2
B	Line SA-2	B13	B12	325	8	Concrete	0.0040	0.50	1972	0.492	Concrete	3
B	Line SA-2	B14	B13	295	8	Concrete	0.0040	0.50	1972	0.447	Concrete	3
B	Line SA-2	B15	B14	290.54	8	Concrete	0.0040	0.50	1972	0.440	Soil	3
B	Line SA-2	B15.1	B15	266.26	8	Concrete	0.0040	0.50	1972	0.403	Soil	2
B	Line SA-2	B16	B15	293.83	8	Concrete	0.0080	0.71	1972	0.445	Asphalt	3
B	Line SA-2	B17	B16	308.8	8	Concrete	0.0040	0.50	1972	0.468	Soil	3
B	Line SA-2	B17	B12.2	310	8	Concrete	0.0040	0.50	1972	0.470	Soil	3
C	Line B	C1	A11	292.03	8	Concrete	0.0046	0.54	1972	0.442	Asphalt	3
C		***C1.1***	***C1***	***37.5***	***8***	***Concrete***	***0.0040***	***0.50***	***1972***	***0.057***	***Asphalt***	***0***
C		***C1.2***	***C1.1***	***175***	***8***	***Concrete***	***0.0040***	***0.50***	***1972***	***0.265***	***Asphalt***	***2***
C	Line B-1	C1.3	C1	310	8	Concrete	0.0083	0.72	1972	0.470	Asphalt	3
C	Line B-1	C1.4	C1.3	300	8	Concrete	0.0040	0.50	1972	0.455	Asphalt	3
C	Line B-1	C1.5	C1.4	307.4	8	Concrete	0.0040	0.50	1972	0.466	Asphalt	3
C	Line B	C2	C1	357.83	8	Concrete	0.0046	0.54	1972	0.542	Asphalt	3
C	Line B	C2.1	C2	302.79	8	Concrete	0.0046	0.54	1972	0.459	Asphalt	3
C	Line B-3	C2.1.1	C2.1	340	8	Concrete	0.0060	0.61	1972	0.515	Gravel	3
C	Line B-3	C2.1.2	C2.1.1	290.34	8	Concrete	0.0060	0.61	1972	0.440	Gravel	3
C	Line B	C2.2	C2.1	171.49	8	Concrete	0.0046	0.54	1972	0.260	Asphalt	1
C	Line B-4	C2.2.1	C2.2	299.06	8	Concrete	0.0040	0.50	1972	0.453	Gravel	3
C	Line B	C2.3	C2.2	265	8	Concrete	0.0046	0.54	1972	0.402	Asphalt	2
Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals

Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
C	Line B-2	C3	C2	339.56	8	Concrete	0.0067	0.65	1972	0.514	Gravel	3
C	Line B-2	C4	C3	290	8	Concrete	0.0040	0.50	1972	0.439	Gravel	3
C		C5	C4	148	8	PVC	0.0040	0.50	1995	0.224	Gravel	1
C		C6	C5	162	8	PVC	0.0040	0.50	1995	0.245	Gravel	1
C		C6.1	C6	152	8	PVC	0.0040	0.50	1995	0.230	Asphalt	1
C		C7	C6	149	8	PVC	0.0040	0.50	1995	0.226	Asphalt	1
C		C8	C7	208	8	PVC	0.0040	0.50	1995	0.315	Asphalt	2
C		C9	C8	173	8	PVC	0.0040	0.50	1995	0.262	Asphalt	2
D	Line C-3	D0.1	PS2	230	8	Concrete	0.0217	1.16	1972	0.348	Gravel	2
D	Line C-3	D0.2	D0.1	271.25	8	Concrete	0.0092	0.76	1972	0.411	Gravel	2
D	Line C	D1	PS2	324.71	8	Concrete	0.0040	0.50	1972	0.492	Soil	3
D	Line C	D2	D1	288.87	8	Concrete	0.0040	0.50	1972	0.438	Soil	3
D		D2.1	D2	260	8	PVC	0.0110	0.83	1995	0.394	Gravel	2
D		D2.2	D2.1	217	8	PVC	0.0040	0.50	1995	0.329	Gravel	2
D		D2.3	D2	156	8	PVC	0.0100	0.79	1995	0.236	Gravel	1
D	Line C	D2.3	D2	126.79	8	Concrete	0.0040	0.50	1972	0.192	Concrete	1
D	Line C-1	D1.1	D1	260	8	Concrete	0.0040	0.50	1972	0.394	Gravel	2
D	Line C-1	D1.2	D1.1	246.06	8	Concrete	0.0264	1.28	1972	0.373	Gravel	2
D	Line C-2	**CO**	D1	125.31	8	Concrete	0.0040	0.50	1972	0.190	Gravel	1
E		E2.1	E2	142	8	PVC	0.0100	0.79	1995	0.215	Asphalt	1
E		***E2.1.1***	***E2.1***	***300***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.455***	***Asphalt***	***3***
E		E2.2	E2.1	315	8	PVC	0.0110	0.83	1995	0.477	Asphalt	3
E		***E2.2.1***	***E2.2***	***337.5***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.511***	***Asphalt***	***3***
E		CO	E2.2	78	8	PVC	0.0100	0.79	1995	0.118	Asphalt	1
E		***E2.3***	***E2.2***	***325***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.492***	***Asphalt***	***3***
E		***E2.4***	***E2.3***	***325***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.492***	***Asphalt***	***3***
E		E4.1	E4	26	8	PVC	0.0500	1.77	1995	0.039	Asphalt	0
E		***E4.2***	***E4.1***	***312.5***	***6***	***PVC***	***0.0040***	***0.50***	***1995***	***0.355***	***Gravel***	***3***
E		E6.1	E6	26	8	PVC	0.0200	1.12	1995	0.039	Asphalt	0
E		***E6.2***	***E6.1***	***312.5***	***6***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.355***	***Gravel***	***3***
E		***E6.3***	***E6.2***	***237.5***	***6***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.270***	***Gravel***	***2***
E	Line E	E7.1	E7	348.9	8	Concrete	0.0040	0.50	1972	0.529	Gravel	3
E	Line E	E7.2	E7.1	319	8	Concrete	0.0040	0.50	1972	0.483	Gravel	3
F	Line F	F1.1	F1	354	8	Concrete	0.0040	0.50	1972	0.536	Gravel	3
F	Line F	F1.2	F1.1	358.54	8	Concrete	0.0040	0.50	1972	0.543	Gravel	3
F	Line F	F1.3	F1.2	316.32	8	Concrete	0.0040	0.50	1972	0.479	Gravel	3
F	Line F	**CO**	F1.3	115	8	Concrete	0.0040	0.50	1972	0.174	Gravel	1
F	Line G	F2.1	F2	238.96	8	Concrete	0.0040	0.50	1972	0.362	Gravel	2
F	Line J	F4.1	F4	400	8	Concrete	0.0040	0.50	1972	0.606	Gravel	3
F	Line J	F4.2	F4.1	435	8	Concrete	0.0040	0.50	1972	0.659	Gravel	4

G	Line H	G1	F3	337.5	8	Concrete	0.0040	0.50	1972	0.511	Soil	3
G	Line H	G1.1	G1	305	8	Concrete	0.0168	1.02	1972	0.462	Gravel	3
G	Line H	G1.2	G1.1	323.69	8	Concrete	0.0086	0.73	1972	0.490	Gravel	3
G	Line H	G1.3	G1.2	417.61	8	Concrete	0.0040	0.50	1972	0.633	Gravel	4
G	Line H-1	G2	G1	215	8	Concrete	0.0067	0.65	1972	0.326	Soil	2
G	Line-H-1-1	G2.1	G2	122.57	8	Concrete	0.0122	0.87	1972	0.186	Soil	1
G	Line H-1	G3	G2	320.7	8	Concrete	0.0040	0.50	1972	0.486	Gravel	3
G	Line H-1	G4	G3	165.85	8	Concrete	0.0083	0.72	1972	0.251	Gravel	1
G	Line H-1	G5	G4	319.32	8	Concrete	0.0040	0.50	1972	0.484	Gravel	3
G	Line H-1-2	G5.1	G5	169	8	Concrete	0.0236	1.21	1972	0.256	Gravel	1
G	Line H-1	G6	G5	226.84	8	Concrete	0.0040	0.50	1972	0.344	Gravel	2
G	Line H-1	G7	G6	226.84	8	Concrete	0.0040	0.50	1972	0.344	Gravel	2
G	Line H-1	G8	G7	339.69	8	Concrete	0.0088	0.74	1972	0.515	Gravel	3
G	Line H-1-3	G8.1	G8	219.14	8	Concrete	0.0040	0.50	1972	0.332	Gravel	2
G	Line H-1-4	G9	G8	273.14	8	Concrete	0.0040	0.50	1972	0.414	Gravel	2
G	Line H-1-4	G10	G9	270	8	Concrete	0.0104	0.81	1972	0.409	Gravel	2
G	Line H-1-4	G11	G10	87.86	8	Concrete	0.0455	1.68	1972	0.133	Gravel	1
G	Line H-1-4-1	G11.1	G11	279.6	8	Concrete	0.004	0.50	1972	0.424	Gravel	2
G	Line H-1-4	G12	G11	310.88	8	Concrete	0.1914	3.46	1972	0.471	Gravel	3
G	Line H-1-4	G13	G12	306	8	Concrete	0.0196	1.11	1972	0.464	Gravel	3
G	Line H-1-4	G14	G13	132.12	8	Concrete	0.1438	2.99	1972	0.200	Gravel	1
G	Line H-1-4	G15	G14	158.07	8	Concrete	0.185	3.40	1972	0.240	Gravel	1

Appendix B
Pe Ell Sewer Inventory

Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
E		***E2.1.1***	***E2.1***	***300***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.455***	***Asphalt***	***1***
E		***E2.2.1***	***E2.2***	***337.5***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.511***	***Asphalt***	***1***
E		***E2.3***	***E2.2***	***325***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.492***	***Asphalt***	***1***
E		***E2.4***	***E2.3***	***325***	***8***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.492***	***Asphalt***	***1***
E		***E6.2***	***E6.1***	***312.5***	***6***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.355***	***Gravel***	***1***
E		***E6.3***	***E6.2***	***237.5***	***6***	***Clay***	***0.0040***	***0.50***	***1950's***	***0.270***	***Gravel***	***1***
				1837.5						***2.5757576***		
A	Interceptor	A2	A1	60	10	Concrete	0.0040	0.90	1972	0.114	Soil	0
A	Interceptor	A3	A2	140	10	Concrete	0.1036	4.60	1972	0.265	Soil	1
A	Interceptor	A4	A3	440	10	Concrete	0.0068	1.18	1972	0.833	Soil	2
A	Interceptor	A5	A4	413.99	10	Concrete	0.0040	0.90	1972	0.784	Soil	2
A	Interceptor	A6	A5	405.19	10	Concrete	0.0040	0.90	1972	0.767	Soil	2
A	Interceptor	A7	A6	360	10	Concrete	0.0040	0.90	1972	0.682	Soil	2
A	Interceptor	A8	A7	324	10	Concrete	0.0040	0.90	1972	0.614	Soil	1
A	Interceptor	A9	A8	325	10	Concrete	0.0040	0.90	1972	0.616	Soil	1
A	Interceptor	A10	A9	325.02	10	Concrete	0.0040	0.90	1972	0.616	Soil/Asphalt	1
A	Interceptor	A11	A10	103.44	10	Concrete	0.0040	0.90	1972	0.196	Concrete	0
A	Interceptor	A12	A11	350	10	Concrete	0.0040	0.90	1972	0.663	Asphalt Pavement	2
A	Interceptor	A13	A12	283.28	10	Concrete	0.0040	0.90	1972	0.537	Asphalt Pavement	1
E	Interceptor	E1	A13	330	10	Concrete	0.0040	0.90	1972	0.625	Asphalt Pavement	1
E	Interceptor	E2	E1	260	10	Concrete	0.0040	0.90	1972	0.492	Gravel	1
E	Interceptor	E3	E2	72.04	8	Concrete	0.0040	0.50	1972	0.109	Gravel/Concrete	0
E	Interceptor	E4	E3	149.21	8	Concrete	0.0040	0.50	1972	0.226	Concrete/Asphalt	1
E	Interceptor	E5	E4	149.21	8	Concrete	0.0040	0.50	1972	0.226	Concrete/Asphalt	1
E	Interceptor	E6	E5	150.51	8	Concrete	0.0040	0.50	1972	0.228	Soil	1
E	Interceptor	E7	E6	150.51	8	Concrete	0.0040	0.50	1972	0.228	Soil	1
F	Interceptor	F1	PS1	200	8	Concrete	0.0040	0.50	1972	0.303	Asphalt Pavement	1
F	Interceptor	F2	F1	298.52	8	Concrete	0.0040	0.50	1972	0.452	Asphalt Pavement	1
F	Interceptor	F3	F2	77.25	8	Concrete	0.0068	0.65	1972	0.117	Asphalt Pavement	0
F	Interceptor	F4	F3	226.81	8	Concrete	0.0125	0.88	1972	0.344	Asphalt/Gravel	1
F	Interceptor	F5	F4	162.2	8	Concrete	0.004	0.50	1972	0.246	Gravel	1
F	Interceptor	F6	F5	391.98	8	Concrete	0.004	0.50	1972	0.594	Gravel	2
F	Interceptor	F7	F6	345.45	8	Concrete	0.0066	0.64	1972	0.523	Gravel	2
F	Interceptor	F8	F7	240.31	8	Concrete	0.0064	0.63	1972	0.364	Gravel	1
B	Line A	B1	A1	130	8	Concrete	0.0040	0.50	1972	0.197	Gravel	1
B	Line A	B2	B1	150	8	Concrete	0.0481	1.73	1972	0.227	Gravel	1
B	Line A	B3	B2	375.36	8	Concrete	0.0040	0.50	1972	0.569	Gravel	2
B	Line A-1	B3.1	B3	251.03	8	Concrete	0.0040	0.50	1972	0.380	Gravel	1

B	Line A	B4	B3	382.97	8	Concrete	0.0150	0.97	1972	0.580	Gravel	2
B	Line A	B5	B4	375	8	Concrete	0.0070	0.66	1972	0.568	Gravel	2
B	Line A	B5.1	B5	230.78	8	Concrete	0.0090	0.75	1972	0.350	Gravel	1
B	Line A	B5.2	B5.1	300	8	Concrete	0.0100	0.79	1972	0.455	Gravel	1
B	Line A	B5.3	B5.2	92.63	8	Concrete	0.0050	0.56	1972	0.140	Gravel	0
B	Line A	B5.4	B5.3	439.56	8	Concrete	0.0100	0.79	1972	0.666	Gravel	2
B	Line A	B5.5	B5.4	280	8	Concrete	0.0040	0.50	1972	0.424	Gravel	1
B	Line A-4	B5.5.1	B5.5	250	8	Concrete	0.0040	0.50	1972	0.379	Gravel	1
B	Line A-4	B5.5.2	B5.5.1	260	8	Concrete	0.0040	0.50	1972	0.394	Gravel	1
B	Line A	B5.6	B5.5	145.1	8	Concrete	0.0040	0.50	1972	0.220	Gravel	1
B	Line A	B5.7	B5.6	234.9	8	Concrete	0.0040	0.50	1972	0.356	Gravel	1
B	Line A-2	B6	B5	370	8	Concrete	0.0070	0.66	1972	0.561	Gravel	2
B	Line A-2	B7	B6	149.3	8	Concrete	0.0070	0.66	1972	0.226	Gravel	1
B	Line A-2	B8	B7	355	8	Concrete	0.0050	0.56	1972	0.538	Gravel	2
B	Line A-2	B9	B8	318	8	Concrete	0.0050	0.56	1972	0.482	Gravel	1
B	Line A-2	B10	B9	365.9	8	Concrete	0.0050	0.56	1972	0.554	Gravel	2
B	Line A-2	B10.1	B10	201.2	8	Concrete	0.0050	0.56	1972	0.305	Gravel	1
B	Line SA-2	B11	B10	185.5	8	Concrete	0.0040	0.50	1972	0.281	Concrete	1
B	Line SA-2	B12	B11	112.16	8	Concrete	0.0040	0.50	1972	0.170	Concrete	0
B	Line SA-2	B12.1	B12	296.91	8	Concrete	0.0040	0.50	1972	0.450	Concrete	1
B	Line SA-2	B12.1.1	B12.1	247.62	9	Concrete	0.0040	0.50	1972	0.422	Concrete	1
B	Line SA-2	B12.2	B12.1	290.45	8	Concrete	0.0040	0.50	1972	0.440	Concrete	1
B	Line SA-2	B12.3	B12.2	237.8	8	Concrete	0.0080	0.71	1972	0.360	Soil	1
B	Line SA-2	B13	B12	325	8	Concrete	0.0040	0.50	1972	0.492	Concrete	1
B	Line SA-2	B14	B13	295	8	Concrete	0.0040	0.50	1972	0.447	Concrete	1
B	Line SA-2	B15	B14	290.54	8	Concrete	0.0040	0.50	1972	0.440	Soil	1
B	Line SA-2	B15.1	B15	266.26	8	Concrete	0.0040	0.50	1972	0.403	Soil	1
B	Line SA-2	B16	B15	293.83	8	Concrete	0.0080	0.71	1972	0.445	Asphalt	1
B	Line SA-2	B17	B16	308.8	8	Concrete	0.0040	0.50	1972	0.468	Soil	1
B	Line SA-2	B17	B12.2	310	8	Concrete	0.0040	0.50	1972	0.470	Soil	1
C	Line B	C1	A11	292.03	8	Concrete	0.0046	0.54	1972	0.442	Asphalt	1
C		***C1.1***	***C1***	***37.5***	***8***	***Concrete***	***0.0040***	***0.50***	***1972***	***0.057***	***Asphalt***	***0***
C		***C1.2***	***C1.1***	***175***	***8***	***Concrete***	***0.0040***	***0.50***	***1972***	***0.265***	***Asphalt***	***1***
C	Line B-1	C1.3	C1	310	8	Concrete	0.0083	0.72	1972	0.470	Asphalt	1
C	Line B-1	C1.4	C1.3	300	8	Concrete	0.0040	0.50	1972	0.455	Asphalt	1
C	Line B-1	C1.5	C1.4	307.4	8	Concrete	0.0040	0.50	1972	0.466	Asphalt	1
C	Line B	C2	C1	357.83	8	Concrete	0.0046	0.54	1972	0.542	Asphalt	2
C	Line B	C2.1	C2	302.79	8	Concrete	0.0046	0.54	1972	0.459	Asphalt	1
C	Line B-3	C2.1.1	C2.1	340	8	Concrete	0.0060	0.61	1972	0.515	Gravel	1
C	Line B-3	C2.1.2	C2.1.1	290.34	8	Concrete	0.0060	0.61	1972	0.440	Gravel	1
C	Line B	C2.2	C2.1	171.49	8	Concrete	0.0046	0.54	1972	0.260	Asphalt	1
C	Line B-4	C2.2.1	C2.2	299.06	8	Concrete	0.0040	0.50	1972	0.453	Gravel	1
C	Line B	C2.3	C2.2	265	8	Concrete	0.0046	0.54	1972	0.402	Asphalt	1

C	Line B-2	C3	C2	339.56	8	Concrete	0.0067	0.65	1972	0.514	Gravel	1
C	Line B-2	C4	C3	290	8	Concrete	0.0040	0.50	1972	0.439	Gravel	1
D	Line C-3	D0.1	PS2	230	8	Concrete	0.0217	1.16	1972	0.348	Gravel	1
D	Line C-3	D0.2	D0.1	271.25	8	Concrete	0.0092	0.76	1972	0.411	Gravel	1
D	Line C	D1	PS2	324.71	8	Concrete	0.0040	0.50	1972	0.492	Soil	1
D	Line C	D2	D1	288.87	8	Concrete	0.0040	0.50	1972	0.438	Soil	1
D	Line C	D2.3	D2	126.79	8	Concrete	0.0040	0.50	1972	0.192	Concrete	1
D	Line C-1	D1.1	D1	260	8	Concrete	0.0040	0.50	1972	0.394	Gravel	1
D	Line C-1	D1.2	D1.1	246.06	8	Concrete	0.0264	1.28	1972	0.373	Gravel	1
D	Line C-2	**CO**	D1	125.31	8	Concrete	0.0040	0.50	1972	0.190	Gravel	1
E	Line E	E7.1	E7	348.9	8	Concrete	0.0040	0.50	1972	0.529	Gravel	2
E	Line E	E7.2	E7.1	319	8	Concrete	0.0040	0.50	1972	0.483	Gravel	1
F	Line F	F1.1	F1	354	8	Concrete	0.0040	0.50	1972	0.536	Gravel	2
F	Line F	F1.2	F1.1	358.54	8	Concrete	0.0040	0.50	1972	0.543	Gravel	2
F	Line F	F1.3	F1.2	316.32	8	Concrete	0.0040	0.50	1972	0.479	Gravel	1
F	Line F	**CO**	F1.3	115	8	Concrete	0.0040	0.50	1972	0.174	Gravel	1
F	Line G	F2.1	F2	238.96	8	Concrete	0.0040	0.50	1972	0.362	Gravel	1
F	Line J	F4.1	F4	400	8	Concrete	0.0040	0.50	1972	0.606	Gravel	2
F	Line J	F4.2	F4.1	435	8	Concrete	0.0040	0.50	1972	0.659	Gravel	2
G	Line H	G1	F3	337.5	8	Concrete	0.0040	0.50	1972	0.511	Soil	1
G	Line H	G1.1	G1	305	8	Concrete	0.0168	1.02	1972	0.462	Gravel	1
G	Line H	G1.2	G1.1	323.69	8	Concrete	0.0086	0.73	1972	0.490	Gravel	1
G	Line H	G1.3	G1.2	417.61	8	Concrete	0.0040	0.50	1972	0.633	Gravel	2
G	Line H-1	G2	G1	215	8	Concrete	0.0067	0.65	1972	0.326	Soil	1
G	Line-H-1-1	G2.1	G2	122.57	8	Concrete	0.0122	0.87	1972	0.186	Soil	1
G	Line H-1	G3	G2	320.7	8	Concrete	0.0040	0.50	1972	0.486	Gravel	1
G	Line H-1	G4	G3	165.85	8	Concrete	0.0083	0.72	1972	0.251	Gravel	1
G	Line H-1	G5	G4	319.32	8	Concrete	0.0040	0.50	1972	0.484	Gravel	1
G	Line H-1-2	G5.1	G5	169	8	Concrete	0.0236	1.21	1972	0.256	Gravel	1
G	Line H-1	G6	G5	226.84	8	Concrete	0.0040	0.50	1972	0.344	Gravel	1
G	Line H-1	G7	G6	226.84	8	Concrete	0.0040	0.50	1972	0.344	Gravel	1
G	Line H-1	G8	G7	339.69	8	Concrete	0.0088	0.74	1972	0.515	Gravel	1
G	Line H-1-3	G8.1	G8	219.14	8	Concrete	0.0040	0.50	1972	0.332	Gravel	1
G	Line H-1-4	G9	G8	273.14	8	Concrete	0.0040	0.50	1972	0.414	Gravel	1
G	Line H-1-4	G10	G9	270	8	Concrete	0.0104	0.81	1972	0.409	Gravel	1
G	Line H-1-4	G11	G10	87.86	8	Concrete	0.0455	1.68	1972	0.133	Gravel	0
G	Line H-1-4-1	G11.1	G11	279.6	8	Concrete	0.004	0.50	1972	0.424	Gravel	1
G	Line H-1-4	G12	G11	310.88	8	Concrete	0.1914	3.46	1972	0.471	Gravel	1
G	Line H-1-4	G13	G12	306	8	Concrete	0.0196	1.11	1972	0.464	Gravel	1
G	Line H-1-4	G14	G13	132.12	8	Concrete	0.1438	2.99	1972	0.200	Gravel	1
G	Line H-1-4	G15	G14	158.07	8	Concrete	0.185	3.40	1972	0.240	Gravel	1
				30213.65						47.385731		

C		C5	C4	148	8	PVC	0.0040	0.50	1995	0.224	Gravel	1
C		C6	C5	162	8	PVC	0.0040	0.50	1995	0.245	Gravel	1
C		C6.1	C6	152	8	PVC	0.0040	0.50	1995	0.230	Asphalt	1
C		C7	C6	149	8	PVC	0.0040	0.50	1995	0.226	Asphalt	1
C		C8	C7	208	8	PVC	0.0040	0.50	1995	0.315	Asphalt	1
C		C9	C8	173	8	PVC	0.0040	0.50	1995	0.262	Asphalt	1
D		D2.1	D2	260	8	PVC	0.0110	0.83	1995	0.394	Gravel	1
D		D2.2	D2.1	217	8	PVC	0.0040	0.50	1995	0.329	Gravel	1
D		D2.3	D2	156	8	PVC	0.0100	0.79	1995	0.236	Gravel	1
E		E2.1	E2	142	8	PVC	0.0100	0.79	1995	0.215	Asphalt	1
E		E2.2	E2.1	315	8	PVC	0.0110	0.83	1995	0.477	Asphalt	1
E		CO	E2.2	78	8	PVC	0.0100	0.79	1995	0.118	Asphalt	0
E		E4.1	E4	26	8	PVC	0.0500	1.77	1995	0.039	Asphalt	0
E		***E4.2***	***E4.1***	***312.5***	***6***	***PVC***	***0.0040***	***0.50***	***1995***	***0.355***	***Gravel***	***1***
E		E6.1	E6	26	8	PVC	0.0200	1.12	1995	0.039	Asphalt	0
				2524.5						3.7066288		
Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
Basin	Line	Upstream MH #	Downstream MH #	Length of Pipe (ft)	Dia. of Pipe (in)	Type of Pipe	Slope of Pipe	Pipe Capacity (MGD)	Approx. Year Pipe Installed	Length of Pipe (in-mi)	Type of Surface Over Pipe	Est. Number of Side Laterals
Total				69151.3						107.33623		

23A160 Chehalis River @ Dryad: continued; more parameters

Date	Time	NO2+NO3 Nitrog. (mg/L)	Dissol. Nitrite (mg/L)
90/10/29	1715	0.478	0.004
90/11/26	1640	0.87	0.010 K
90/12/17	1405	0.59	
91/01/28	1530	0.56	0.010 K
91/02/25	1430	0.57	0.010 K
91/03/25	1550	0.4	0.010 K
91/04/22	1430	0.398	0.010 K
91/05/27	1425	0.118	0.010 K
91/06/24	1430	0.167	0.010 K
91/07/29	1400	0.105	0.010 K
91/08/26	1530	0.038	0.010 K
91/09/23	1510	0.025	0.010 K
91/10/29	1540	0.122	0.010 K
91/11/19	1410	0.171	0.010 K
91/12/17	1505	0.613	0.010 K
92/01/28	1630	0.847	0.010 K
92/02/25	1555	0.567	0.010 K
92/03/24	1650	0.229	0.010 K
92/04/28	1625	0.295	0.010 K
92/05/26	1640	0.089	0.010 K
92/06/23	1650	0.086	0.010 K
92/07/28	1900	0.032	0.010 K
92/08/25	1725	0.01	0.010 K
92/09/29	1350	0.132	0.010 K
92/10/26	1120	0.025	0.010 K
92/11/22	1130	0.959	0.010 K
92/12/20	1130	0.634	0.010 K
93/01/25	1250	0.397	0.010 K
93/02/22	1105	0.387	0.010 K
93/03/22	1200	0.362	0.010 K
93/04/26	1140	0.263	0.010 K
93/05/24	1110	0.209	0.010 K
93/06/28	1130	0.166	0.010 K
93/07/26	1155	0.111	0.010 K
93/08/23	1105	0.065	0.010 K
93/09/27	1200	0.045	0.010 K
93/10/27	935	0.061	
93/11/22	930	0.098	

Date	Time	NO2+NO3 Nitrog. (mg/L)	Dissol. Nitrite (mg/L)
93/12/21	1030	0.63	
94/01/26	1015	0.507	
94/02/23	840	0.649	
94/03/30	750	0.441	
94/04/27	830	0.271	
94/05/25	930	0.153	
94/06/29	830	0.145	
94/07/27	910	0.07	
94/08/24	825	0.055	J
94/09/28	830	0.037	
94/10/26	935	0.31	
94/11/30	1015	0.829	
94/12/29	1145	0.71	
95/01/25	1010	0.588	
95/02/28	1035	0.548	
95/03/28	1035	0.481	
95/04/26	945	0.221	
95/05/24	900	0.216	
95/06/28	920	0.122	
95/07/26	910	0.054	
95/08/27	1000	0.023	
95/09/27	915	0.205	
95/10/25	845	0.417	
95/11/29	805	0.573	
95/12/20	835	0.579	
96/01/31	845	0.553	
96/02/28	815	0.537	
96/03/27	845	0.278	
96/04/30	840	0.506	
96/05/29	905	0.268	
96/06/25	850	0.059	
96/07/31	1005	0.118	
96/08/28	945		

Remarks codes: U,K - Below reporting limits; B - analyte found in blank; - many background organisims; J - Estimate; S - Spreader colony.

Town of Pe Ell General Sewer Plan

Main Line and Pump Stations Cost Estimate (for urban growth service)

Item	Unit	Quantity	Unit Price	Estimated Cost
8" Line @ 4-8 ft depth (East of Town)	Linear foot	8,650	$ 60.00	$ 519,000
8" Line @ 4-8 ft depth (West of Town)		1,400	$ 60.00	$ 84,000
4" dia. Force Main	Linear foot	4,000	$ 30.00	$ 120,000
Manholes (@ 350' intervals)	Each	29	$ 2,500.00	$ 72,500
Pump Stations	Each	5	$ 75,000.00	$ 375,000
Surface Restoration	Linear foot	10,050	$ 35.00	$ 351,750
		Subtotal		$ 1,522,250
Construction Contingency @ 35%				$ 532,788
		Subtotal		$ 2,055,038
		Sales Tax @ 7.60%		$ 156,183
		Subtotal		$ 2,211,220
Admin., Survey, Design, Construction Man., and Inspection @ 35% of Construction Budget				$ 773,927
		Total Construction Cost		**$ 2,985,147**

Estimated Operation and Maintenance Cost

Labor to Maintain Line	
2 hrs/wk*52 wks/yr*$20/hr	$ 2,080
Materials/Equipment to Maintain Line	
2% of Construction Cost/Year	$ 18,865
Labor to Maintain Pump Station	
4 hrs/day*260days/yr*$20/hr	$ 41,600
Materials/Equipment to Maintain Pump Station & F.M.	
5% of Construction Cost/Year	$ 24,750
Power Cost @ $0.025 per KWH	
.746 KWH/HP*2.5HP/pump*5duplexpumps*1680hrs/yr*$KWH	$ 783
Total Yearly Operation and Maintenance Costs	$ 88,078

APPENDIX E EQ
EQUALIZATION STORAGE ANALYSIS AND DESIGN CRITERIA

GENERAL

Equalization (EQ) storage is required as part of many of the alternatives presented in Section VII of this report. The total amount of EQ storage required for each option is dependent upon several factors including the end use of the treated wastewater, permitted discharge limits, rainfall, WWTP capacity, river flow and daily flow to the WTTP. The following discussion of EQ storage capacity requirements is intended for use in developing the alternatives and costs in the analysis in Section VII.

EQUALIZATION MODEL

A detailed spreadsheet model has been developed to determine future EQ storage requirements for the various Consent Decree compliance alternatives. The Consent Decree parameters establish WWTP discharge limits for flow conditions within the Centralia Reach of the Chehalis River as discussed in Section III of this report. EQ storage requirements were modeled based on the historical river conditions and WWTP flow data from 1990 through 1997. The model uses the historical data to determine the periods when the historical WWTP discharge would have been "In the River" (wet weather) or "Out of the River" (dry weather). The model encompasses a wide range of daily river flow (from historic low flow to record flood conditions) and corresponding daily WWTP flow.

Individual daily WWTP flows in the model are projected to 2025 conditions. The estimates of 2025 WWTP flow projections include additional flow from new customers (as identified in Section V) as well as adjustments to the specific daily flow for additional future I/I in proportion to future peak I/I. As a result, the 2025 peak daily WWTP flow in the model corresponds directly to the 2025 peak daily flow estimate in Section V of this report. In addition, projected EQ basin requirements are adjusted in all cases to account for rainfall contribution directly to the basin.

EQUALIZATION STORAGE FOR EACH END USE ALTERNATIVE

The model was utilized to evaluate projected EQ storage requirements for each end use alternative considered. A copy of a spreadsheet showing results of a model run based on Consent Decree discharge limitations, peak WWTP capacity of 13.0 MGD and discharge below the Skookumchuck River is included in this Appendix. A discussion of the model runs for each end use alternative is as follows.

Regional Alternatives

EQ storage volumes for regional alternatives are based on conveyance of dry weather treated effluent downstream of the Skookumchuck River or year round conveyance of raw sewage to a regional WWTP in Centralia. Under all of the regional alternatives, WWTP effluent discharge and raw wastewater pumping capacity are limited by river flow as specified within the consent Decree. Although there may be some flexibility to utilize excess capacity in Centralia's proposed WWTP under their future NPDES Permit, this analysis assumes that peak flows for both collection systems will occur during the same periods and discharge limits will be strictly enforced.

Table EQ-1 shows the required EQ storage for regional alternatives at peak WWTP or raw sewage pumping capacities ranging from 7.0 to 14.0 MGD.

Table EQ-1 EQ Storage Required for Regional Alternatives

WWTP Capacity (MGD)	EQ Storage Requirement (MG)		
	Dry Weather *	Wet Weather	Dry to Wet Transition
7	4.9	24	14
8	4.9	17	12
9	4.9	10	9.5
10	4.9	7.1	7.3
11	4.9	4.1	6.2
12	4.9	2.0	5.2
13	4.9	0.1	4.2
14	4.9	0.0	4.1

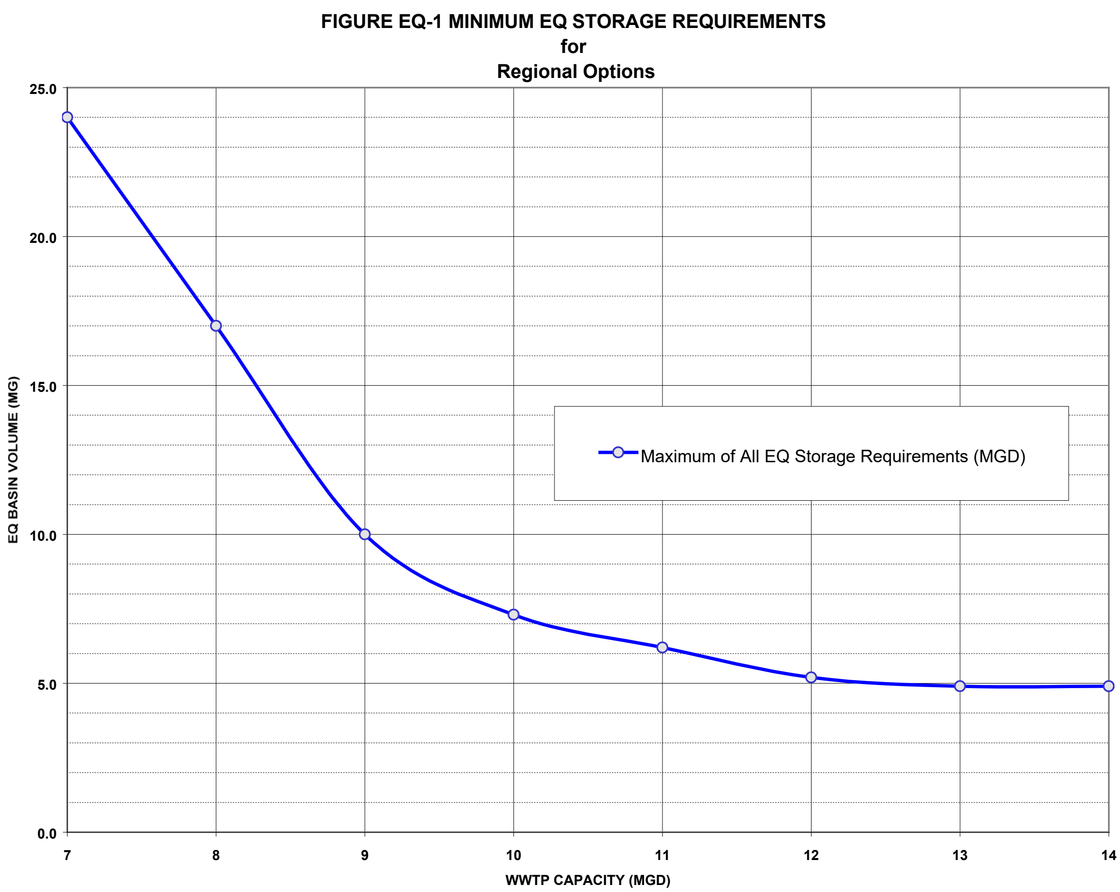
* Not dependent on WWTP capacity. Based on dry weather discharge limits.

Dry weather EQ storage is dependent upon the maximum discharge limits of 2.5 (when flow in the Centralia Reach is less than 200 cfs) and 3.0 MGD (when flow in the Centralia Reach is from 200 to 1,000 cfs). Therefore, Dry weather EQ storage requirements are independent of WWTP capacity when WWTP capacity is greater than 2.5 MGD.

Wet weather EQ storage requirements are highly dependent upon WWTP capacity and range from a high of 24.0 MG at 7.0 MGD of WWTP capacity to a low of 0.0 MG at 14.0 MGD of WWTP capacity. However, the wet weather discharge limit in the Consent Decree is 13.0 MGD. Therefore, the minimum wet weather storage required under regional options is 0.1 MGD if WWTP capacity is provided up to the maximum permitted discharge limit.

The dry weather to wet weather transitional storage requirements shown in the table are based on detailed review of model output for transitional events. During the transitional events, a major storm event increases WWTP flows, which fill the dry weather EQ storage before the river flows increase to a level that will allow the WWTP discharge to be increased. In all of the transitional periods, it is possible to empty the majority of the dry weather EQ storage upon initiation of wet weather discharge limits. However, in consideration of practical operation and control, the more conservative assumption is to utilize the combination of the two storage volumes during the transition period for design purposes.

Figure EQ-1 shows the minimum EQ storage requiredments for regional options based on values in Table EQ-1. The EQ storage values in Figure EQ-1 represent the maximum of the EQ storage required for either wet, dry, or transitional periods for each specific WWTP capacity shown.



Reuse Alternatives

Under all of the reuse alternatives, dry weather EQ storage capacity is dependent on the reuse and is not limited by river flow. Wet weather EQ storage capacity is dependent on the final permit conditions for wet weather discharge up to 13.0 MGD.

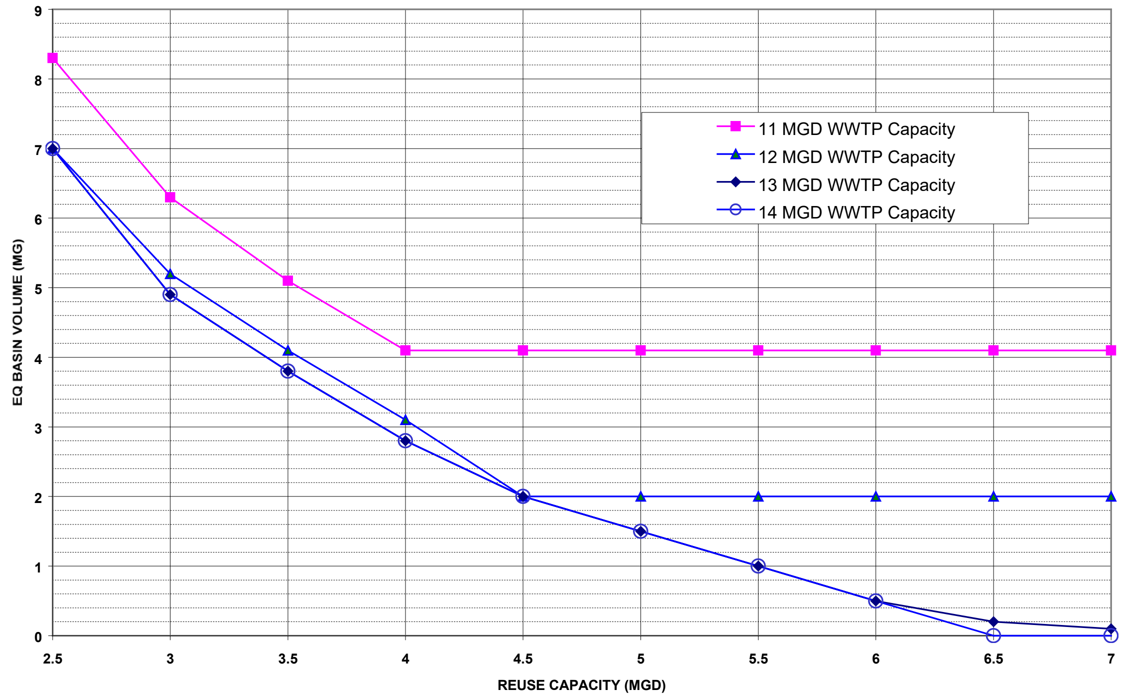
Table EQ-2 shows the EQ storage requirements for reuse options. The EQ storage requirements in the Table are based on the minimum EQ storage required for either dry, wet or transitional periods as discussed in the analysis of the regional alternatives above. The most significant dry weather to wet weather transition event in the model occurred in November 1995.

Table EQ-2 EQ Storage Requirements for Reuse Alternatives

	EQ Storage Requirement (MG)			
Reuse Capacity (MGD)	WWTP Wet Weather Capacity			
	11 MGD	12 MGD	13 MGD	14 MGD
2.5	8.3	7.0	7.0	7.0
3.0	6.3	5.2	4.9	4.9
3.5	5.1	4.1	3.8	3.8
4.0	4.1	3.1	2.8	2.8
4.5	4.1	2.0	2.0	2.0
5.0	4.1	2.0	1.5	1.5
6.0	4.1	2.0	0.5	0.5
7.0	4.1	2.0	0.1	0.0

The lower range of reuse capacity was selected to evaluate the relative cost differences and feasibility of constructing equalizing storage in lieu of peak reuse treatment and disposal capacity. For reuse capacities at or below 2.0 MGD, EQ storage requirements are in excess of 25 MG. The high range of reuse capacity (7.0 MGD) is based on the minimum peak reuse capacity that will result in no required dry weather EQ storage. Wet weather discharge permit limits must be modified to just above 13.0 MGD (13.1 MGD) to allow for the provision of no EQ storage. The decision to increase the permit limits should be based on the cost of reuse capacity relative to EQ storage capacity. Figure EQ-2 shows EQ storage requirements for the range of reuse and WWTP capacities shown in Table EQ-2.

FIGURE EQ-2 EQ STORAGE REQUIREMENTS
for
Reuse Options



River Enhancement Alternatives

DOE has not committed to final flow limitations for river enhancement alternatives. It it assumed that river enhancement will result in mass loading limits, but not flow limits. If there are no flow limits, EQ storage requirements will be based entirely on WWTP capacity. If treatment is provided to address peak daily flows, the minimum EQ storage requirement is zero. The optimum EQ storage capacity for river enhancement alternatives is dependent upon the balance between the incremental costs of additional treatment capacity and additional EQ storage. Optimal EQ storage under these options will be discussed in detail in the analysis of alternatives in section VII of this report.

Storage of All Dry Weather Flows for Discharge during Wet Weather Periods

EQ storage requirements for retaining all dry weather period flows for treatment and discharge during wet weather periods has been estimated by setting the dry weather discharge limits to zero within the model. The wet weather limit is maintained at 13.0-MGD for the purpose of this specific analysis. The minimum EQ storage required under this scenario is approximately 440 MG using an EQ basin depth of 20 feet. The minimum area required for this size of storage facility is approximately 75 acres not including potential set backs, which may be required in certain areas.

EQ BASIN DESIGN CRITERIA AND IMPLEMENTATION

Design Criteria

The actual design of EQ storage is based on the treatment process utilized, projected size of the EQ storage and prospective site conditions. General criteria for aeration of raw wastewater, solids handling, and other issues are addressed in DOE guidance material (Orange Book). Criteria for construction of impoundment's greater than 10 acre-ft (3.25 MG) are addresses in DOE dam safety guidance material.

Additional capacity for diurnal fluctuations is not needed because the model evaluated daily flows which are delivered to the basin over a 24-hour period. Conveyance facilities to the EQ basin will include a peak diurnal factor where appropriate. Conveyance facilities from storage to treatment or disposal will not require adjustments for diurnal conditions because treatment and disposal capacities will be based on peak daily flow over a 24-hour period. The final basin design for each alternative will include minimum side water depth of two feet below the basin overflow.

Siting

EQ Storage facilities siting criteria is consistent with the siting criteria for WWTP facilities discussed above. Flood protection requirements for dry weather finished effluent EQ storage is not as stringent as the requirements for WWTP facilities in general since the EQ storage would not be used or needed during flood periods. However, wet weather EQ storage for routine or emergency raw wastewater equalization would require flood protection to DOE standards.

Potential sites for EQ storage include the current WWTP site and various sites within industrial zoned areas. The WWTP site will require minimal conveyance improvements and the industrial sites with the vicinity of Darigold will require relatively moderate conveyance costs, which are discussed under the analysis of alternatives in Section VII of this report The industrial sites South of the city limits are not considered for dry weather EQ storage because of the excessive conveyance costs. However, if wet weather EQ storage is needed, the industrial sites to the south may be feasible for upstream hydraulic relief. This is not anticipated for the planning period, but may be needed in the future due to WWTP site constraints and future peak flows.

Upstream EQ Storage

Upstream EQ storage for raw wastewater can be provided under several of the alternatives discussed above. However, due to operation and control issues, upstream equalizing storage is not recommended for storage related to dry weather flows. For dry weather flow equalization it is important to equalize the flow at the point in the system where a consistent level of flow is anticipated such as just prior to or just after treatment. This issue is not as critical for wet weather flow equalization because it is possible to intercept consistently high flows at several locations within the collection system. Therefore, the only practical application under the proposed end use alternatives is for providing upstream equalization for wet weather flows. Such is the case where adequate peak capacity for reuse treatment and disposal is provided to eliminate the need for dry weather equalization storage.

EQ Storage Costs

EQ storage costs are highly variable for each alternative due to different site constraints and WWTP capacity options. For that reason, EQ costs are determined independently for each alternative in section VII of this report.

Wastewater Treatment Plant Operation
with Extended Aeration and Complete Mix

The existing plant will be modified so that it will be operated as an extended aeration plant in low flow and as a complete mix conventional activated sludge plant during high flows. In addition, the two existing aeration basins that are used for nitrification during the summer and for equalization storage during the winter will now be used year-round as aeration basins. Both of the existing aeration basins will be used at all times, but the process will be different depending upon influent flows. New equalization storage basins will be built to provide needed capacity for both dry weather and wet weather conditions. The existing trickling filters will be demolished. The existing primary clarifiers will continue to be used and two new 65-foot diameter secondary clarifiers will be built to increase TSS removal capabilities.

In order to maximize nitrification and BOD_5 removal during low flow conditions, the plant will be operated in the extended aeration mode. This process is characterized by a very long hydraulic retention time (HRT) and low mixed liquor suspended solids (MLSS) concentration with consistent aeration to keep the dissolved oxygen (DO) above 2 mg/l. Overall this process will be used for flows up to 4.5 MGD. One basin will be adequate up to 2.0 MGD. Flows between 2.0 and 4.5 MGD will be handled by using both basins in the extended aeration mode.

As the inflow increases during the wet weather, the effluent limits for TSS, BOD_5 and ammonia are not as stringent as they are during the critical dry weather period. When this happens, the plant will switch over to the complete mix activated sludge process. This process is characterized by a short HRT and a higher MLSS than the extended aeration process. Inflows between 4.6 and 5.0 MGD will be treated in just one basin in the complete mix mode. When inflows are between 5.1 and 9.5 MGD, they will be treated using extended aeration in one basin and complete mix in the other basin. Flows in excess of 9.5 MGD will be treated in both basins using the complete mix process. The existing aeration basins will be able to treat up to 13.0 MGD to secondary standards.

The DOE design criteria for the extended aeration process calls for an HRT of 10 - 24 hours and 3 - 5 hours for the complete mix process. The following table shows how the plant will be operated with varying inflows:

	Extended Aeration (HRT=10-24 Hrs)		Complete Mix (HRT=3-5 Hrs)	
Inflow	**Flow**	**HRT**	**Flow**	**HRT**
1	1	*23	0	--
1.5	1.5	*15	0	--
2	2	*11.5	0	--
2.5	2.5	18.3	0	--
3	3	15.3	0	--
3.5	3.5	13.1	0	--
4	4	11.5	0	--
4.5	0	--	4.5	*5
5	0	--	5	*4.6
5.5	1	23	4.5	5.1
6	1.5	15	4.5	5.1
6.5	2	11.5	4.5	5.1
7	2	11.5	5	4.6
7.5	2	11.5	5.5	4.2
8	2	11.5	6	3.8
8.5	2	11.5	6.5	3.5
9	2	11.5	7	3.3
9.5	2	11.5	7.5	3.1
10	0	--	10	4.6
10.5	0	--	10.5	4.4
11	0	--	11	4.2
11.5	0	--	11.5	4.0
12	0	--	12	3.8
12.5	0	--	12.5	3.7
13	0	--	13	3.5

** Using 1 Basin Only*

The modified plant must be capable of handling the anticipated BOD_5 loading for the year 2025 which is 5,500 ppd based on the maximum month. The primary clarifiers remove approximately 25% of the influent BOD_5. After primary clarification, this leaves 4,125 ppd (5,500 - 25%) of BOD_5 that needs to be treated. The DOE design criteria calls for BOD_5 loading to be 10 -25 lbs/1,000 cf of aeration basin capacity for extended aeration and 20 - 120 lbs/1,000 cf for complete mix. The combined volume of both aeration basins is 255,000 cf. For extended aeration, the basins have a capacity of 2,553 to 6,383 ppd of BOD_5. They are therefore adequate for BOD_5 loading with, or without, the primary clarifiers. For complete mix, the BOD_5 capacity is 5,107 to 30,640 ppd. The existing aeration basins are more than adequate for either mode of operation. The disadvantage of this option is the increased level of operation and control needed to operate in both extended aeration and complete mix mode simultaneously.

Since the primary clarifiers are old and not as efficient as new center feed types, they will only be used for flows up to 6 MGD which is the future maximum monthly average flow rate. Flows greater than that will go directly to the aeration basins after passing through the headworks. The existing primary clarifiers are 50-foot diameter each with a total area of 3,056 sf. The DOE design criteria calls for an overflow rate of 800 -1,200 gpd/sf at average flow and 2,000 - 3,000 gpd/sf at peak flow. At 4 MGD which is the future average wet weather flow rate, the overflow rate is 1,020 gpd/sf which meets the design criteria. The overflow rate at 6 MGD is 1,530 gpd/sf which is less than the allowable rate of 2,000 gpd/sf for peak flows. The primary clarifiers will be used as they are now with only one unit in service during the summer and both units in operation during the winter for flows up to 6 MGD.

The two existing secondary clarifiers are not adequate to provide for TSS removal in the future. The plant currently has numerous permit violations for effluent TSS concentration and percent removal due to high flows. One of the secondary clarifiers is the old spiraflow type and is not very efficient. It is also shallow and does not meet the depth requirements prescribed in the DOE design criteria. The second clarifier was built in 1987 and is 18-feet deep with center feed. It has excellent TSS removal capacity as long as the overflow rates are kept within design standards. It is recommended that two new 65-foot diameter secondary clarifiers be built. The DOE design criteria calls for an overflow rate of 400 - 600 gpd/sf for the extended aeration process and 600 - 800 gpd/sf for all other activated sludge processes with a peak overflow rate of 1,200 gpd/sf. With two new clarifiers, the total area will be 13,268 sf. At the future maximum monthly average flow of 6 MGD, the overflow rate is 450 gpd/sf which is near the low end of the range for the extended aeration process and is more than adequate for the complete mix process. At the 13 MGD peak flow, the overflow rate is 980 gpd/sf which is less that the required value of 1,200. During the summer months two clarifiers will be used. As flow increases, more will be brought on line with the oldest one put into service last. A new flow splitter box will be built to proportion out flow between the clarifiers.

APPENDIX E

Gibbs & Olson, Inc.

Town of Pe Ell

File : 626.13

GENERAL SEWER PLAN

c:\g&o\626.13\appendix-E.xls

COST ESTIMATE FOR OPTION 1 : Oxidation Ditch

ITEM	DESCRIPTION	QUANTITY		UNIT PRICE	AMOUNT
1	**Oxidation Ditch Basin & Equipment**				
	a. Oxidation Ditch Equipment (1 MGD)	1	L.S.	$48,500.00	$48,500.00
	b. Oxidation Ditch Installation	1	L.S.	$24,250.00	$24,250.00
	c. Oxidation Ditch Concrete	310	C.Y.	$400.00	$124,000.00
	Subtotal Oxidation Basin & Equipment				$196,750.00
2	**Clarifier Basin & Equipment**				
	a. Clarifier Equipment	1	L.S.	$100,000.00	$100,000.00
	b. Clarifier Equipment Installation	1	L.S.	$70,000.00	$70,000.00
	c. Clarifier Concrete	250	C.Y.	$400.00	$100,000.00
	Subtotal Clarifier Basin & Equipment				$270,000.00
3	Earthwork	1	L.S.	$5,000.00	$5,000.00
4	Piping	1	L.S.	$50,000.00	$50,000.00
5	Electrical	1	L.S.	$20,000.00	$20,000.00
6	Bar Screen Equipment	1	L.S.	$50,000.00	$50,000.00
7	Bar Screen Structure & Installation	1	L.S.	$30,000.00	$30,000.00
8	Grit Chamber	1	L.S.	$20,000.00	$20,000.00
9	Flow Diversion	1	L.S.	$15,000.00	$15,000.00
10	UV System (1.0 MGD Peak)	1	L.S.	$52,000.00	$52,000.00
11	Equalization Storage	5,000,000	Gal.	$0.03	$150,000.00
12	Solids Process Modifications	1	L.S.	$20,000.00	$20,000.00
13	Standby Generator (50 K.W.)	1	L.S.	$13,500.00	$13,500.00
14	Effluent Main	260	L.F.	$50.00	$13,000.00
15	Lab/Control Building	750	S.F.	$50.00	$37,500.00
16	Control Building Furnishings & Equipment	1	L.S.	$10,000.00	$10,000.00
17	Solids Application Area	5	Acre	$10,000.00	$50,000.00
				SUBTOTAL	$1,002,750.00

Note: These are bare costs that do not include contingency, sales tax, design, administration, etc.

COST ESTIMATE FOR OPTION 2: D-Ditch

ITEM	DESCRIPTION	QUANTITY		UNIT PRICE	AMOUNT
1	**D-Ditch Basin & Equipment**				
	a. D-Ditch Equipment	1	L.S.	$259,500.00	$259,500.00
	b. D-Ditch Installation	1	L.S.	$129,750.00	$129,750.00
	c. D-Ditch Concrete	342	C.Y.	$400.00	$136,800.00
	Subtotal D-Ditch Basin & Equipment				$526,050.00
2	Earthwork	1	L.S.	$8,000.00	$8,000.00
3	Piping	1	L.S.	$50,000.00	$50,000.00
4	Electrical	1	L.S.	$20,000.00	$20,000.00
5	Bar Screen Equipment	1	L.S.	$50,000.00	$50,000.00
6	Bar Screen Structure & Installation	1	L.S.	$30,000.00	$30,000.00
7	Grit Chamber	1	L.S.	$20,000.00	$20,000.00
8	Flow Diversion	1	L.S.	$15,000.00	$15,000.00
9	UV System (1.0 MGD Peak)	1	L.S.	$52,000.00	$52,000.00
10	Equalization Storage	5,000,000	Gal.	$0.03	$150,000.00
11	Solids Process Modifications	1	L.S.	$20,000.00	$20,000.00
12	Standby Generator (50 K.W.)	1	L.S.	$13,500.00	$13,500.00
13	Effluent Main	260	L.F.	$50.00	$13,000.00
14	Lab/Control Building	750	S.F.	$50.00	$37,500.00
15	Control Building Furnishings & Equipment	1	L.S.	$10,000.00	$10,000.00
16	Solids Application Area	5	Acre	$10,000.00	$50,000.00
				SUBTOTAL	$1,065,050.00

COST ESTIMATE FOR OPTION 3 : SBR

Note: These are bare costs that do not include contingency, sales tax, design, administration, etc.

ITEM	DESCRIPTION	QUANTITY		UNIT PRICE	AMOUNT
1	**SBR Basin & Equipment**				
	a. SBR Equipment (1.0 MGD)	1	L.S.	$200,000.00	$200,000.00
	b. SBR Equipment Installation	1	L.S.	$100,000.00	$100,000.00
	c. SBR Basin Concrete	418	C.Y.	$400.00	$167,200.00
	Subtotal SBR Basin & Equipment				$467,200.00
2	Earthwork	1	L.S.	$10,000.00	$10,000.00
3	Piping (10% of subtotal SBR Basin & Equip)	1	L.S.	$50,000.00	$50,000.00
4	Electrical (5% of subtotal SBR Basin & Equip)	1	L.S.	$20,000.00	$20,000.00
5	Bar Screen Equipment	1	L.S.	$50,000.00	$50,000.00
6	Bar Screen Structure & Installation	1	L.S.	$30,000.00	$30,000.00
7	Grit Chamber	1	L.S.	$20,000.00	$20,000.00
8	Flow Diversion	1	L.S.	$15,000.00	$15,000.00
9	UV System (1.0 MGD Peak)	1	L.S.	$52,000.00	$52,000.00
10	Blower Building	100	S.F.	$30.00	$3,000.00
11	Equalization Storage	5,000,000	Gal.	$0.03	$150,000.00
12	Solids Process Modifications	1	L.S.	$20,000.00	$20,000.00
13	Standby Generator (50 K.W.)	1	L.S.	$13,500.00	$13,500.00
14	Effluent Main	260	L.F.	$50.00	$13,000.00
15	Lab/Control Building	750	S.F.	$50.00	$37,500.00
16	Control Building Furnishings & Equipment	1	L.S.	$10,000.00	$10,000.00
17	Solids Application Area	5	Acre	$10,000.00	$50,000.00
				SUBTOTAL	$1,011,200.00

COST ESTIMATE FOR OPTION 4 : Oversized SBR

ITEM	DESCRIPTION	QUANTITY	UNIT PRICE	AMOUNT
1	**SBR Basin & Equipment**			

Note: These are bare costs that do not include contingency, sales tax, design, administration, etc.

	a. SBR Equipment (2.9 MGD Peak Day)	1	L.S.	$255,000.00	$255,000.00
	b. SBR Equipment Installation	1	L.S.	$127,000.00	$127,000.00
	c. SBR Basin Concrete	650	C.Y.	$400.00	$260,000.00
	Subtotal SBR Basin & Equipment				$642,000.00
2	Earthwork	1	L.S.	$20,000.00	$20,000.00
3	Piping (8% of subtotal SBR Basin & Equip)	1	L.S.	$50,000.00	$50,000.00
4	Electrical (3% of subtotal SBR Basin & Equip)	1	L.S.	$20,000.00	$20,000.00
5	Bar Screen Equipment	1	L.S.	$50,000.00	$50,000.00
6	Bar Screen Structure & Installation	1	L.S.	$30,000.00	$30,000.00
7	Grit Chamber	1	L.S.	$20,000.00	$20,000.00
8	UV System (2.9 MGD Peak)	1	L.S.	$110,000.00	$110,000.00
9	Blower Building	100	S.F.	$30.00	$3,000.00
10	Solids Process Modifications	1	L.S.	$20,000.00	$20,000.00
11	Standby Generator (50 K.W.)	1	L.S.	$13,500.00	$13,500.00
12	Effluent Main	260	L.F.	$50.00	$13,000.00
13	Lab/Control Building	750	S.F.	$50.00	$37,500.00
14	Control Building Furnishings & Equipment	1	L.S.	$10,000.00	$10,000.00
15	Solids Application Area	1	L.S.	$10,000.00	$10,000.00
				SUBTOTAL	$1,049,000.00

Note: These are bare costs that do not include contingency, sales tax, design, administration, etc.

APPENDIX F

The summary is meant to provide a quick overview of the regulations and does not contain all of the requirements, exceptions or details of the regulations. The 503 Regulations are organized into the following subparts:

- Subpart A - General Provisions (Sections 503.1 through 503.9);
- Subpart B - Land Application (Sections 503.10 through 503.18);
- Subpart C - Surface Disposal (Sections 503.20 through 503.28);
- Subpart D - Pathogens & Vector Attraction Reduction (Sections 503.30 through 503.33); and
- Subpart E - Incineration (Sections 503.40 through 503.48).

The standards for each sludge use or disposal category (land application, surface disposal, and incineration) consist of the following:

- Applicability and Definitions
- General Requirements
- Pollutant Limits
- Pathogen Reduction Requirements
- Management Practices
- Monitoring
- Record Keeping and Reporting Requirements.

The following discussion summarizes Subparts A, B and D of the 503 Regulations. Surface disposal and incineration are not included because they are not deemed suitable for the Town of Pe Ell.

503 REGULATIONS - GENERAL PROVISIONS

The 503 Regulations apply to sewage sludge generated from the treatment of domestic sewage and domestic septate. The regulations DO NOT establish requirements for processes used to treat domestic sewage or processes used to treat sewage sludge prior to final use or disposal, except for; 1) determining the sludge classification with respect to pathogens, and 2) to ensure adequate vector attraction reduction is achieved. The regulations also DO NOT require the selection of a sewage sludge use or disposal method. The regulations specifically state that it is a local determination regarding the manner in which sewage sludge is used or disposed of.

Compliance with the 503 Regulations was required within 12 months of enactment, or by February 19, 1994. If the ability to achieve compliance required the construction of capital facilities, compliance was required by February 19, 1995. The regulations are written so that they are self-implementing, meaning that EPA can enforce the regulations even before a specific permit is issued to a facility. Compliance with monitoring, record keeping and reporting requirements was required by July 20, 1993 (150 days after publication in the Federal Register).

Currently the 503 Regulations are implemented and enforced by EPA The new State Rule for Biosolids Management being prepared by the Washington State Department of Ecology (DOE) is anticipated to be very similar to the federal regulations, and will be implemented and enforced by DOE. At some point in the future, DOE also anticipates being delegated by EPA as the federal permitting and enforcement agency for biosolids used beneficially on the land within the State of Washington. It is anticipated that EPA will continue to be the permitting and enforcement agency for biosolids disposed of in landfills or by incineration. Once DOE assumes implementation and enforcement responsibilities, permits may be issued to individual facilities to implement the requirements. Where possible permit requirements will be issued through, or associated with, the National Pollution Discharge Elimination System (NPDES) permits issued to treatment facilities.

PART 503 REGULATIONS - LAND APPLICATION

Land application generally includes the application of bulk or bagged biosolids to land at agronomic nitrogen rates. The application rate varies based on the nitrogen demand of the crop/vegetation being grown and the nitrogen content of the biosolids. Typical land application uses include; 1) agricultural land, 2) non-agricultural land, 3) public contact sites such as parks and golf courses, 4) disturbed land reclamation such as mines and gravel pits, and 5) home lawns and gardens.

The treatment facility is required to provide biosolids users with the information needed to comply with the regulations. The regulations prefer high quality biosolids meeting "clean" biosolids pollutant concentration limits and Class A pathogen reduction requirements through elimination of both information requirements and management practices and by reducing record keeping requirements.

Pollutant Limits for Land Application

Land application pollutant limits vary with the final use of the biosolids. Pollutant limits are listed in Tables 1 through 4. Biosolids used in the following manners shall meet the given pollutant limit criteria:

- All biosolids that are land applied shall not exceed the Pollutant Ceiling Concentrations listed in Table 1, (refer to Section 503.13(b)(1) of the regulations).

- Bulk biosolids applied to agricultural or non-agricultural land, a public contact site, or a reclamation site shall not exceed either the cumulative pollutant loading rates shown in Table 2, or the pollutant concentrations shown in Table 3, (refer to Section 503.13(b)(2) and (b)(3) of the regulations).

- Bulk biosolids applied to a lawn or a home garden shall not exceed the pollutant concentrations shown in Table 3, (refer to Section 503.13(b)(3) of the regulations).
- Biosolids sold or given away in a bag or other container for application to the land shall not exceed the pollutant concentrations listed in Table 3, or the product of the concentration of each pollutant in the biosolids and the annual whole biosolids application rate for the biosolids shall not exceed the annual pollutant loading rates listed in Table 4, (refer to Section 503.13(b)(4) of the regulations).

For general land application, biosolids must meet both the Pollutant Ceiling Concentrations in Table 1 and the Cumulative Pollutant Loadings in Table 2 or the Pollutant Concentrations for "clean" biosolids in Table 3. All concentrations in Tables 1 and 3 are on a dry weight basis.

TABLE 1 POLLUTANT CEILING CONCENTRATIONS	
Pollutant	Ceiling Concentration (mg/Kg)
Arsenic	75
Cadmium	85
Chromium	3,000
Copper	4,300
Lead	840
Mercury	57
Molybdenum	75
Nickel	420
Selenium	100
Zinc	7,500

TABLE 2 CUMULATIVE POLLUTANT LOADING RATES		
Pollutant	Cumulative Pollutant Loading Rate (Kg/Hectare)	Cumulative Pollutant Loading Rate (Lbs/Acre)
Arsenic	41	36
Cadmium	39	34
Chromium	3,000	2,676
Copper	1,500	1,338
Lead	300	268
Mercury	17	15
Molybdenum	18	16
Nickel	420	374
Selenium	100	89
Zinc	2,800	2,498

TABLE 3 POLLUTANT CONCENTRATIONS	
Pollutant	Monthly Average Concentrations (mg/Kg)
Arsenic	41
Cadmium	39
Chromium	1,200
Copper	1,500
Lead	300
Mercury	17
Molybdenum	18
Nickel	420
Selenium	36
Zinc	2,800

TABLE 4 ANNUAL POLLUTANT LOADING RATES		
Pollutant	Annual Pollutant Loading Rate (Kg/Hectare/Year)	Annual Pollutant Loading Rate (Lbs/Acre/Year)
Arsenic	2.0	1.8
Cadmium	1.9	1.7
Chromium	150	134
Copper	75	67
Lead	15	13
Mercury	0.85	0.76
Molybdenum	0.90	0.80
Nickel	21	19
Selenium	5.0	4.5
Zinc	140	125

Note: The conversion factors used in Tables 2 and 4 are,
1 Kg = 2.2046 Pounds and 1 Hectare = 2.471 Acres.

Pathogen and Vector Attraction Reduction Requirements for Land Application

The level of pathogen reduction required varies with the end use of the sludge as summarized below:

- Bulk biosolids applied to agricultural land and non-agricultural land, a public contact site, or a reclamation site shall meet Class B pathogen requirements and site restrictions.

- Bulk biosolids applied to lawns and home gardens shall meet Class A pathogen requirements.

- Biosolids sold or given away in bags shall meet Class A pathogen requirements.

Pathogen reduction requirements for biosolids to be Class A or Class B are described later in this section.

Land application of biosolids will require one of the vector attraction reduction options listed in Table 5. Vector attraction reduction for bulk biosolids applied to lawns and gardens and for bagged biosolids must be met with one of the first eight options listed in Table 5.

Management Practices for Land Application

The following management practices shall be followed when land applying stabilized biosolids:

- Biosolids shall not be applied to the land if it is likely to adversely affect a threatened or endangered species listed under Section 4 of the Endangered Species Act or its designated critical habitat.

- Biosolids shall not be applied to flooded, frozen, or snow-covered ground so that biosolids enter surface waters or wetlands.
- Biosolids shall not be applied to land within 10 meters (32.8 feet) of waters of the United States.

- Biosolids shall not be applied at rates above agronomic rates. Permitting authorities have some flexibility on agronomic rates at reclamation sites.

- Biosolids sold or given away in bags shall include a label or information sheet provided to the user with the following information included.

    a. Name and address of person selling/giving away biosolids.
    b. Prohibition on use except in accordance with instructions on label/sheet.
    c. Annual whole biosolids application rate for the particular biosolid.

Monitoring for Land Application

Monitoring for pollutants, pathogens and vector attraction reduction requirements shall be at a frequency determined by annual biosolids production volume listed in Table 5.

TABLE 5 MONITORING FREQUENCY		
Amount of Biosolids (Dry Metric Tons/Year)	Amount of Biosolids (Dry Tons/Year)	Monitoring Frequency
0 to < 290	0 to < 320	Annually
290 to < 1,500	320 to <1,653	Quarterly
1,500 to < 15,000	1,653 to < 16,535	Bi-Monthly
> 15,000	>16,535	Monthly

Note: 1 metric ton = 1.1 English tons; 1 English ton = 2,000 pounds

Record Keeping and Reporting for Land Application

Records are required to be developed and kept for five years. The types of records required depend on the quality of the biosolids and the end use. The requirements are more stringent for sludge which does not meet both Class A pathogen requirements and the "clean" biosolids pollutant concentration limits previously reviewed. Since the 503 Regulations are self-implementing and are intended to be self-enforcing, specific certification statements are required from treatment works officials as part of the record keeping. There are six categories of record keeping contained in the regulations for domestic biosolids. These are summarized as follows:

- Required records for land application of biosolids that meet the "clean" biosolids pollutant concentrations, Class A pathogen reduction requirements and vector attraction reduction are; a) pollutant concentrations, b) how Class A pathogen reduction is met, c) how vector attraction reduction is met, and d) a required certification statement from the biosolids generator or preparer that Class A and vector attraction requirements are met, (refer to Section 503.17(a)(1)(ii) of the regulations for the required statement).

- For a material or product derived from biosolids that do not meet "clean" biosolids pollutant concentrations, Class A pathogen requirements, or vector attraction reduction requirements, but the final product does meet all the requirements the following records are required; a) pollutant concentrations, b) how Class A pathogen reduction is met, c) how vector attraction reduction is met, and d) a required certification statement from the final product generator or preparer, (refer to Section 503.17(a)(2)(ii)).

- For biosolids that meet the "clean" biosolids pollutant concentrations, Class A pathogen reduction requirements and are subsurface injected or incorporated into the soil, (see Sections 503.33(b)(9) and 503.33(b)(10)), to achieve vector attraction reduction the required records are; a) pollutant concentrations, b) how Class A pathogen reduction is met, c) how vector attraction reduction is met, d) how management practices are met, and e) required certification statements from both the generator and applicator, (see Sections 503.17(a)(3)(i)(B) and 503.17(a)(3)(ii)(B)).

- Required records for land application of biosolids that meet the "clean" biosolids pollutant concentrations, Class B pathogen reduction requirements, and vector attraction reduction are; a) pollutant concentrations, b) how pathogen reduction is met, c) how vector attraction reduction is met, d) how management practices are met, e) how site restrictions for Class B biosolids are met, and f) certification statements from the generator and applicator, (refer to Sections 503.17(a)(4)(i)(B) and 503.17(a)(4)(ii)(B)).

- For land application of a biosolid that does not meet the "clean" biosolids pollutant concentrations, but does meet both the pollutant ceiling concentrations (Table 1) and the cumulative pollutant loadings (Table 3), Class B pathogen reduction requirements, and vector attraction reduction the following records are required; a) Location and size of application site, b) date and time biosolids is applied, c) amount of each pollutant applied to the site, d) amount of biosolids applied to the site, e) pollutant concentrations, f) how pathogen reduction is met, g) how management practices are met, h) how vector attraction reduction is met, i) how site restrictions for Class B biosolids are met, and j) certification statements from the generator and applicator, (refer to Sections 503.17(a)(5)(i)(B) and 503.17(a)(5)(ii)(F), 503.17(a)(5)(ii)(H) and 503.17(a)(5)(ii)(L)). Items a through d are to be kept indefinitely to document compliance with cumulative loading rate limits.

- For stabilized biosolids sold or given away in bags the following records are required; a) the annual biosolids application rate which avoids exceeding the annual pollutant loading, b) pollutant concentrations, c) how Class A pathogen reduction is met, d) how vector attraction reduction is met, e) a copy of the required label or information sheet, and f) a certification statement from the generator, (see Section 503.17(a)(6)(iii)).

Information contained in the required documentation shall be submitted to the permitting authority on February 19th of each year for sludge management facilities or publicly owned treatment works (POTW's) with a design flow rate equal to or greater than one million gallons per day (MGD), or that serve more than 10,000 people.

503 REGULATIONS - PATHOGEN AND VECTOR ATTRACTION REDUCTION REQUIREMENTS

Scope: This subpart contains the requirements for biosolids to be classified as Class A or B with respect to pathogens, site restrictions for land on which Class B sludge is applied, and alternative vector attraction reduction requirements for biosolids applied to the land or placed on a surface disposal site.

Pathogen Class A Requirements

A biosolid is classified as Class A with respect to pathogen control if the following criteria is met. Either the density of fecal coliform in the biosolids shall be less than 1,000 Most Probable Number (MPN) per gram of total solids (dry weight basis), or the density of salmonella bacteria in the biosolids shall be less than 3 MPN per 4 grams of total solids (dry weight basis) at the time the biosolids are used or disposed of. In addition, one of the following six alternatives must be met.

- Various time/temperature relationships. An example of these relationships is the relationship for biosolids with more than seven percent solids concentration, a temperature of 50 degrees C or higher and a time period of 20 minutes or longer. For biosolids in this category the temperature and time period shall be determined using the following equation:

  $D = 131{,}700{,}000/10^{0.1400t}$; D = time in days, t = temp in degrees C.

- The pH of the biosolids shall be raised to above 12 for at least 72 hours, with the temperature above 52 degrees C for at least 12 hours, followed by air drying to achieve at least 50 percent solids.

- Density calculations of enteric viruses and viable helminth ova before and after treatment to determine process operating parameters. The parameters are then monitored and used to document compliance.

- Density calculations of enteric viruses and viable helminth ova at the time the biosolids are used or disposed of. The density of enteric viruses shall be less than 1 plague-forming unit per 4 grams of total solids (dry weight basis). The density of viable helminth ova shall be less than 1 per 4 grams of total solids (dry weight basis).

- Biosolids shall be treated in one of the Processes to Further Reduce Pathogens (PFRP) contained in Appendix B of the 503 Regulations.

- Biosolids shall be treated in a process that is equivalent to a PFRP, as determined by the permitting authority.

To minimize the regrowth potential of pathogens, the Class A pathogen reduction requirements shall be met either prior to meeting, or at the same time as the vector attraction reduction is met, if utilizing one of the biological stabilization techniques, such as volatile solids reduction, listed in Table 6.

Pathogen Class B Requirements

To be classified as Class B with respect to pathogen controls, biosolids must meet one of the following three options:

- The geometric mean of the density of fecal coliform of seven representative biosolid samples shall be less than either 2,000,000 MPN per gram of total solids (dry weight basis), or 2,000,000 Colony Forming Units per gram of total solids (dry weight basis).

- Biosolids shall be treated in one of the Processes to Significantly Reduce Pathogens (PSRP) contained in Appendix B of the 503 Regulations.

- Biosolids shall be treated with a process that is equivalent to a PSRP as determined by the permitting authority.

In addition to the pathogen reduction requirements for biosolids to be designated as Class B, there are site restrictions which also must be met. The site restrictions are summarized as follows:

- Food crops with harvested parts that touch the biosolids/soil mixture shall not be harvested for 14 months after application of the biosolids.
- Food crops with harvested parts below the surface of the land shall not be harvested for 20 months after biosolids application if biosolids remain on the land surface for four months or more prior to incorporation into the soil.

- Food crops with harvested parts below the surface of the land shall not be harvested for 38 months after biosolids application if biosolids remain on the land surface for less than four months before being incorporated into the soil.

- Food crops, feed crops and fiber crops shall not be harvested for 30 days after application of the biosolids.

- Animals shall not be allowed to graze on the land for 30 days after application of the biosolids.

- Turf grown on the biosolids applied land shall not be harvested for one year after biosolids application, if it will be placed on land with a high potential for public exposure or a lawn.

- Public access to land with high potential for public exposure shall be restricted for one year after application of biosolids.

- Public access to land with low potential for public exposure shall be restricted for one year after application of the biosolids.

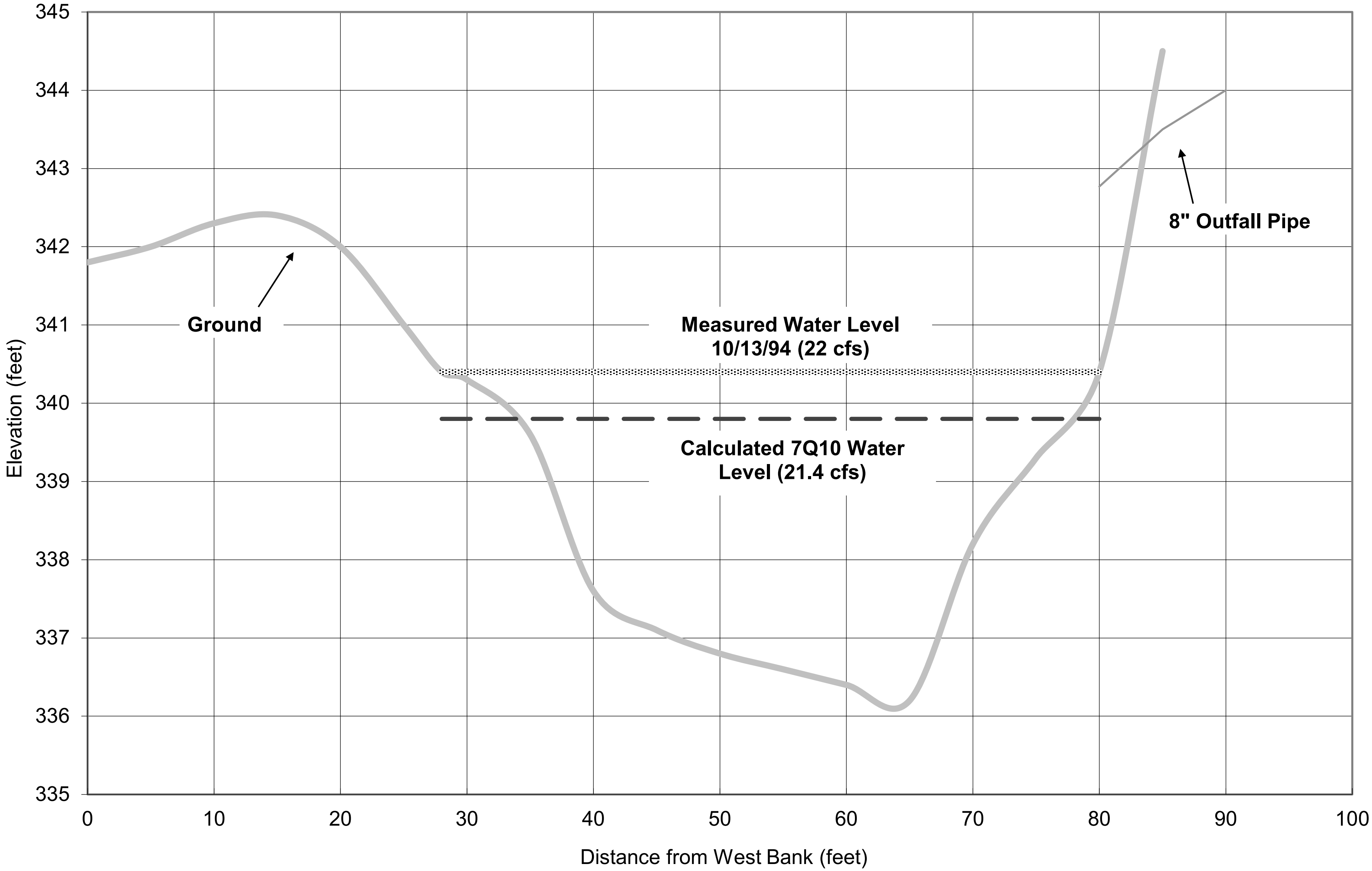
VECTOR ATTRACTION REDUCTION

Ten vector attraction reduction options are contained in Table 6 on the following page. For biosolids applied to agricultural land, non-agricultural land, a public contact site, or a reclamation site one of the ten options must be used. For biosolids applied to lawns and gardens and for bagged biosolids, one of the first eight options must be utilized.

TABLE 6 VECTOR ATTRACTION REDUCTION OPTIONS
1. The mass of volatile solids in the biosolids must be reduced a minimum of 38 percent.
2. If anaerobic digestion cannot meet the required 38 percent reduction, vector attraction

	reduction can be demonstrated in the laboratory for 40 additional days of digestion at 30 to 37 degrees C. If the additional volatile solids reduction after 40 days is less than 17 percent, vector attraction reduction is achieved.
3.	If aerobic digestion cannot meet the required 38 percent reduction, vector attraction can be demonstrated in the laboratory for 30 additional days of digestion at 20 degrees C. If the additional volatile solids reduction after 30 days is less than 15 percent, vector attraction reduction is achieved.
4.	The specific oxygen uptake rate (SOUR) for aerobically treated biosolids shall be equal to or less than 1.5 milligrams of oxygen per hour per gram of total solids (dry weight basis) at a temperature of 20 degrees C.
5.	Biosolids shall be treated in an aerobic process for 14 days or longer at greater than 40 degrees C. The average temperature of the biosolids shall be higher than 45 degrees C.
6.	The pH of the biosolids shall be raised to 12 or higher by alkali addition, and remain at 12 or higher for two hours and then at 11.5 or higher for an additional 22 hours, without the addition of more alkali.
7.	The percent solids of biosolids that does not contain unstabilized primary sludge shall be a minimum of 75 percent prior to mixing with other materials.
8.	The percent solids of biosolids that does not contain unstabilized primary sludge shall be a minimum of 90 percent prior to mixing with other materials.
9.	Biosolids shall be injected below the surface of the land. No biosolids shall be present on the land surface within one hour after the biosolids are injected. If the biosolids are Class A with respect to pathogens, they shall be injected below the land surface within eight hours after being discharged from the pathogen reduction process.
10.	Biosolids applied to land surface or placed on a surface disposal site shall be incorporated into the soil within six hours after application. If biosolids are Class A with respect to pathogens, they shall be injected below the land surface within eight hours after being discharged from the pathogen reduction process.

FIGURE III-1
PE ELL WWTP OUTFALL EVALUATION
Chehalis River Cross-Section @ Outfall



Cross-sectional data taken by Gibbs and Olson on 8/15/96

FIGURE III-2
Chehalis River Centerline Profile in the Vicinity of the Pe Ell WWTP Outfall Discharge Pipe
(labels on the River Bottom profile define the exact distance from the outfall).

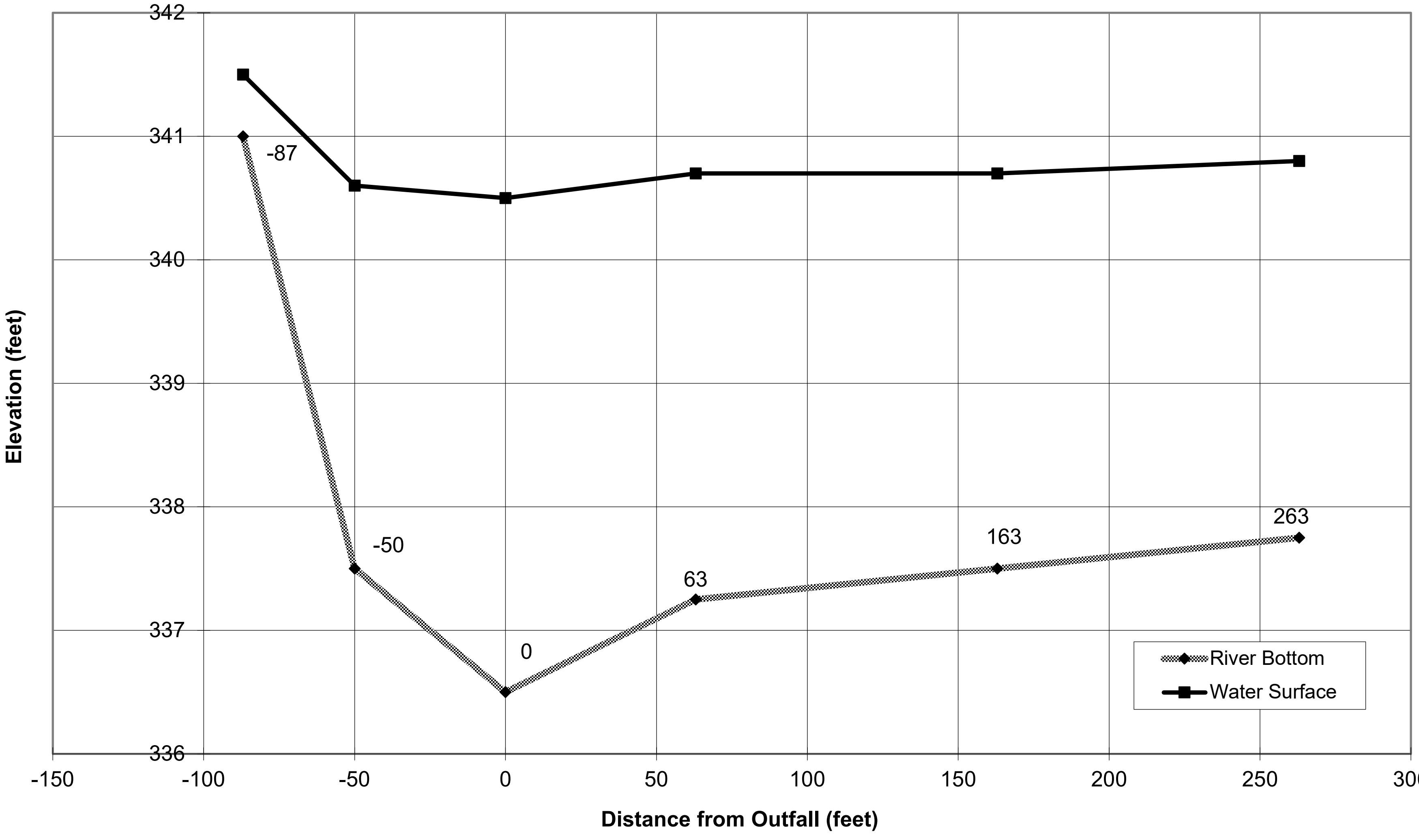


FIGURE VI-3
CHEHALIS/NAPAVINE/LCSD NO. 1 FLOW MONITORING STUDY

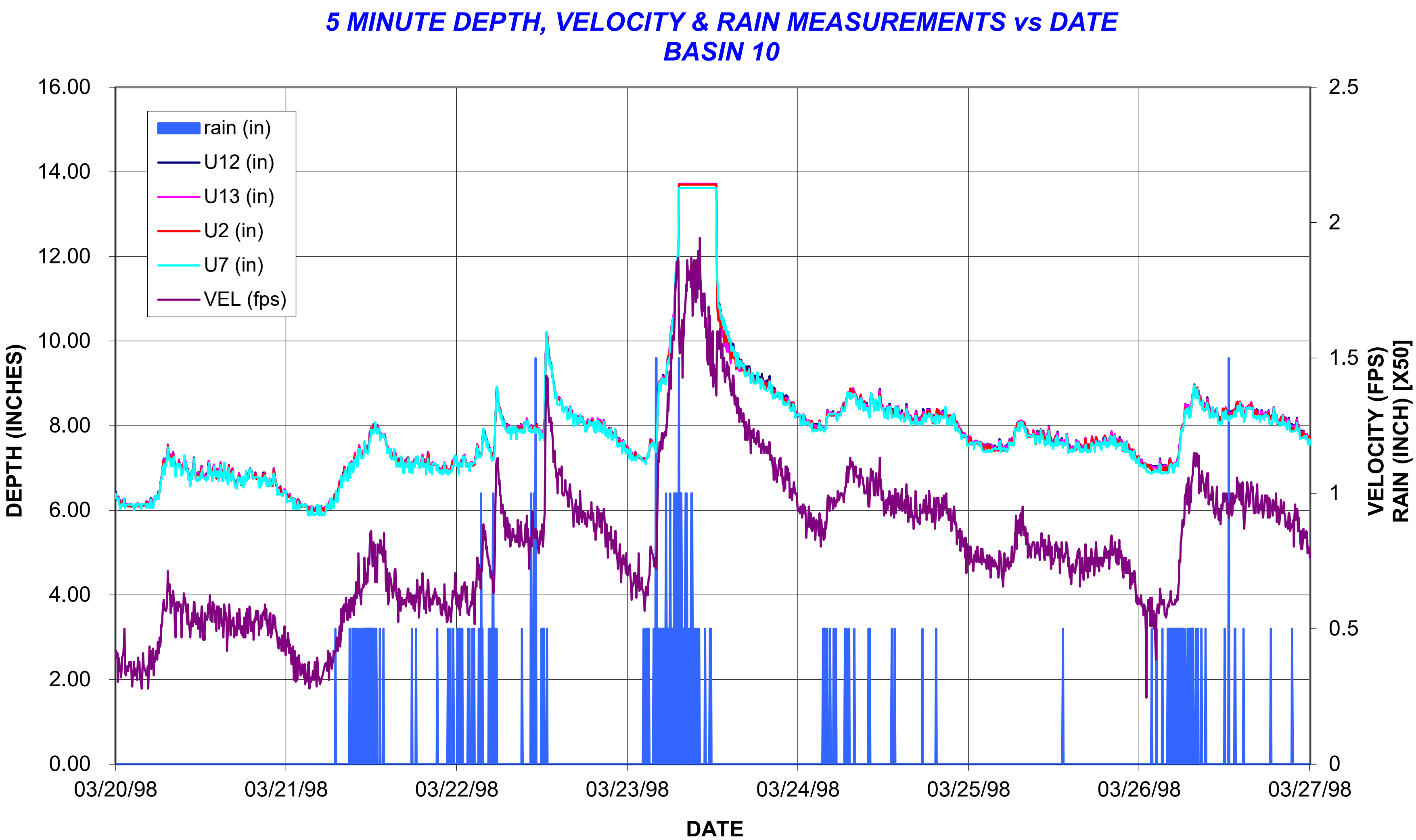


TABLE VI-4
CITY OF CHEHALIS GENERAL SEWER PLAN
BASIN FLOWS AND INCREASES ABOVE BASE FLOWS DURING MONITORED STORMS

BASIN NUMBER	WINTER BASE FLOW (MGD)	% OF TOTAL BASE FLOW	SUMMER BASE FLOW (MGD)	STORM 1 JAN 21-24 FLOW (MGD) % INCREASE	STORM 2 FEB 11-14 FLOW (MGD) % INCREASE	STORM 3 FEB 19-21 FLOW (MGD) % INCREASE	STORM 4 FEB 26-28 FLOW (MGD) % INCREASE	STORM 5 MAR 7-9 FLOW (MGD) % INCREASE	STORM 6 MAR 20-24 FLOW (MGD) % INCREASE	AVERAGE % INCREASE DURING STORMS	February 7, 1996 ESTIMATED I/I FLOW (MGD) % INCREASE
1	0.15	10.36%	0.12	0.84	0.30	0.45	0.57	0.39	0.71		2.09
				461%	103%	**202%**	**283%**	161%	**375%**	**264%**	***1402%***
2	0.54	37.55%	0.43	1.06	1.01	1.02	1.01	1.00	1.00		2.03
				97%	86%	89%	88%	84%	85%	88%	375%
3	0.04	2.60%	0.03	0.27	0.07	0.14	0.24	0.18	0.24		0.75
				631%	87%	**274%**	**528%**	**388%**	**548%**	**409%**	***2015%***
4	0.14	9.41%	0.11	*1.00*	0.63	0.76	0.58	0.43	0.78		2.39
				637%	**363%**	**463%**	**327%**	**220%**	**476%**	**414%**	***1759%***
5	0.05	3.69%	0.04	N/A	0.10	0.10	0.08	0.09	0.08		0.14
					87%	91%	56%	74%	52%	72%	255%
6	0.11	7.60%	0.09	0.18	0.13	0.14	0.12	0.12	0.15		0.24
				69%	17%	25%	11%	11%	41%	29%	215%
7	0.04	3.00%	0.03	0.22	0.08	0.12	0.09	0.08	0.10		0.23
				405%	93%	181%	114%	90%	132%	169%	539%
8	0.18	12.20%	0.14	0.49	0.33	0.37	0.28	0.24	0.29		0.53
				179%	89%	109%	61%	37%	65%	90%	301%
9	0.04	2.95%	0.03	0.10	N/A	N/A	N/A	N/A	*0.10*		0.23
				135%					235%	185%	552%
10	0.15	10.65%	0.12	*1.00*	0.53	0.72	0.56	0.41	0.75		2.22
				552%	**246%**	**372%**	**267%**	164%	**389%**	**332%**	***1450%***
TOTAL	1.44		1.15	5.17	3.18	3.83	3.55	2.94	4.11		10.85

ESTIMATED FLOWS SHOWN IN ITALICS

N/A NOT AVAILABLE

PERCENTAGES IN **BOLD** ARE GREATER THAN 200% INCREASES OVER WINTER BASE FLOWS

ESTIMATED PEAK I/I VALUES IN ***BOLD/ITALICS*** ARE FROM NON-REHABILITATED BASINS COMPRISED LARGELY OF OLD (GREATER THE 40 YEARS) PIPE.

PEAK TO OBSERVED I/I CONVERSION FACTOR: 3.55

FIGURE VI-5
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #1

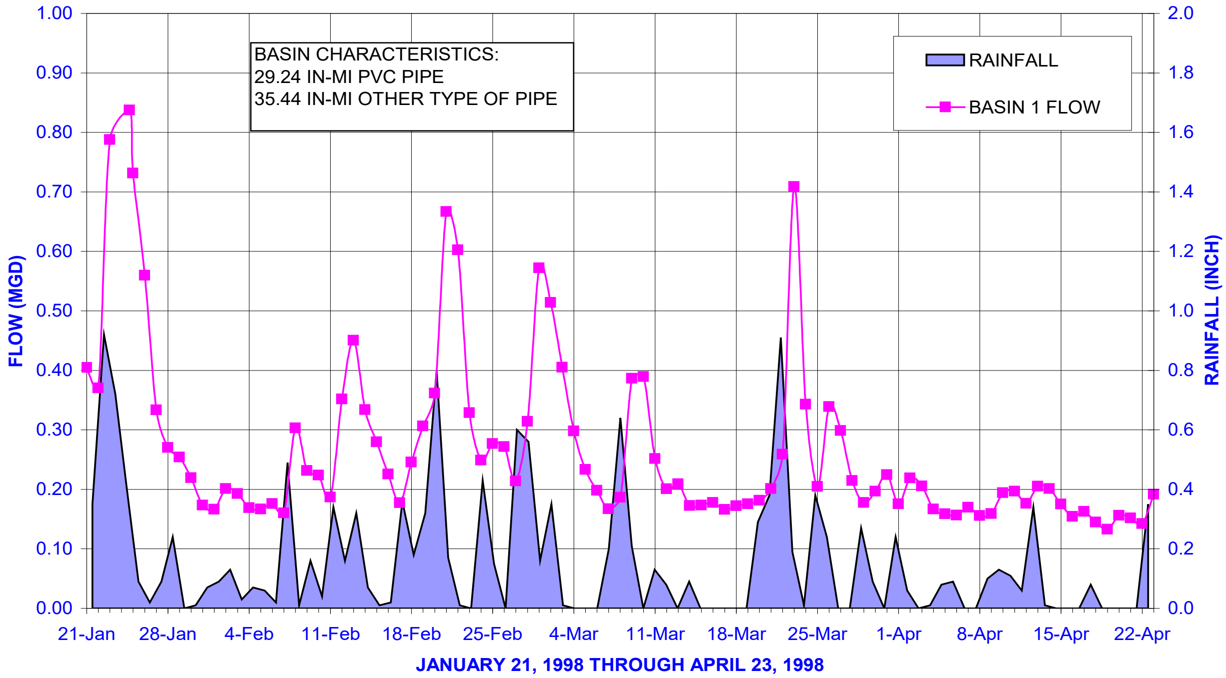


FIGURE VI-6
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & NET FLOW vs TIME
BASIN #2

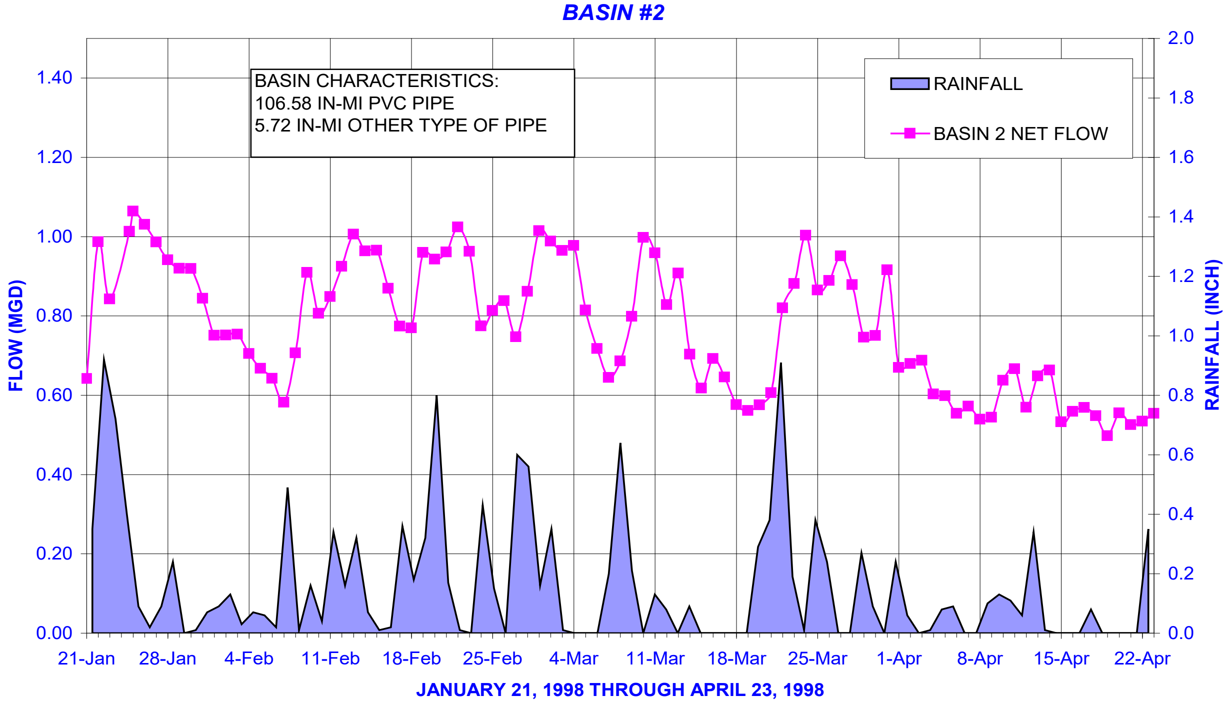


TABLE VI-6
CITY OF CHEHALIS
1988 SEWER REHABILITATION PROGRAM

1998 BASIN NUMBER	1988 BASIN NUMBER	LENGTH OF PIPE IN BASIN (IN-MI)	ESTIMATED TOTAL PEAK I/I BEFORE REHAB (GPD/IN-MI)	ESTIMATED TOTAL PEAK I/I BEFORE REHAB (GPD)	ESTIMATED TOTAL PEAK I/I REMOVED (GPD)	**(1)** CONSTRUCTION REHAB COST INCLUDING SALES TAX	TOTAL EST. COST INCLUDING ADMINISTRATION, LEGAL, DESIGN CONSTRUCTION MANAGEMENT, AND INSPECTION @ 30%
1	2024	8.44	95,464	805,713	794,741	$452,258	$587,935
9	**2012**	**13.94**	**95,343**	**1,329,082**	**1,310,960**	**$630,251**	**$819,326**
2	**4078**	**2.54**	**76,570**	**194,487**	**191,185**	**$296,829**	**$385,878**
2	**4026**	**3.26**	**72,513**	**236,392**	**232,154**	**$375,036**	**$487,547**
11	**5082**	**9.6**	**67,896**	**651,800**	**639,320**	**$344,709**	**$448,122**
2	**2004**	**11.52**	**54,125**	**623,515**	**608,539**	**$794,469**	**$1,032,810**
2	**3012**	**37.96**	**52,236**	**1,982,861**	**1,933,513**	**$2,893,823**	**$3,761,970**
2	**4082**	**19.04**	**46,483**	**885,029**	**860,277**	**$1,011,884**	**$1,315,449**
1	2041	4.49	43,327	194,538	188,701	$425,158	$552,705
1	3058	8.93	38,902	347,399	335,790	$546,517	$710,472
2	**4006**	**8.68**	**37,440**	**324,979**	**313,695**	**$918,894**	**$1,194,562**
1	2051	14.06	36,142	508,151	489,873	$347,894	$452,262
10	6058	39.94	30,504	1,218,322	1,166,400	$1,715,237	$2,229,808
1	2063	7.56	30,496	230,553	220,725	$434,742	$565,165
4	5076	42.1	24,538	1,033,069	978,339	$2,260,460	$2,938,598
3	4038	7.12	21,339	151,931	142,675	$747,331	$971,530
3	4050	7.07	13,995	98,942	89,751	$394,882	$513,347
5,6,7,8,11,12	8002	178.23	12,325	2,196,697	1,964,998	$240,980	$313,274
1	1022	10.37	4,884	50,646	37,165	$0	$0

Note: Basins in BOLD have been rehabilitated since 1988.

(1) Actual construction costs for rehabilitated basins are in bold and other costs are estimated costs from 1988 Sewer System Rehabilitation Project.

FIGURE VI-7
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #3

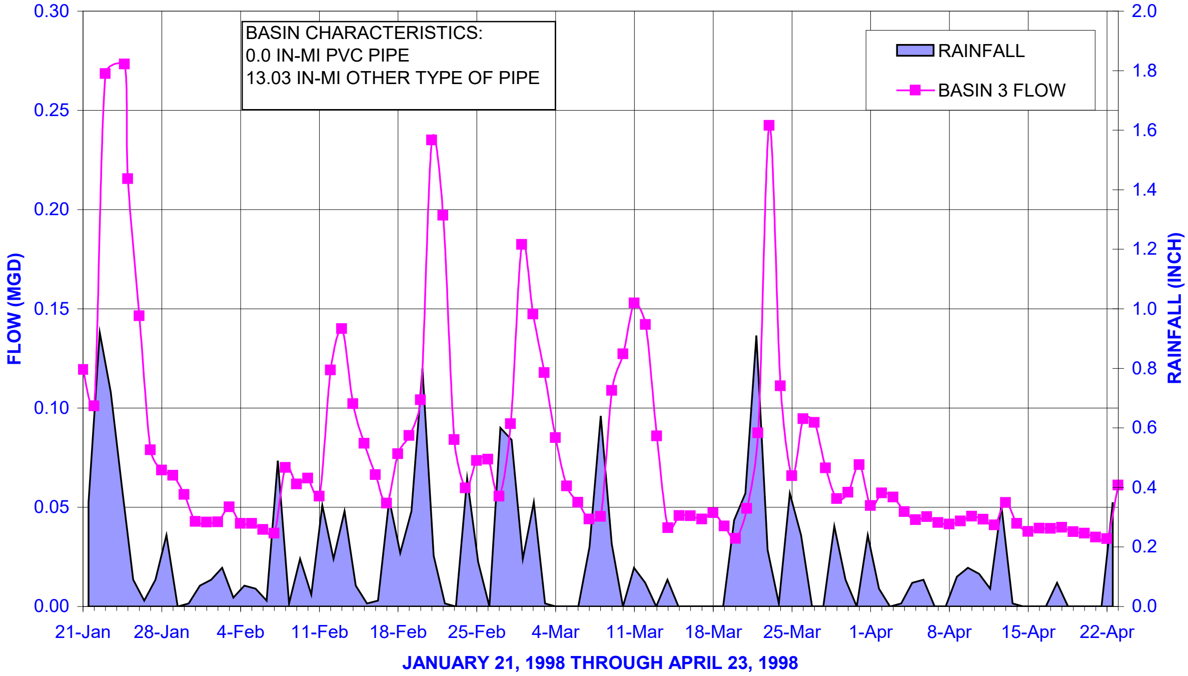


FIGURE VI-8
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & NET FLOW vs TIME
BASIN #4

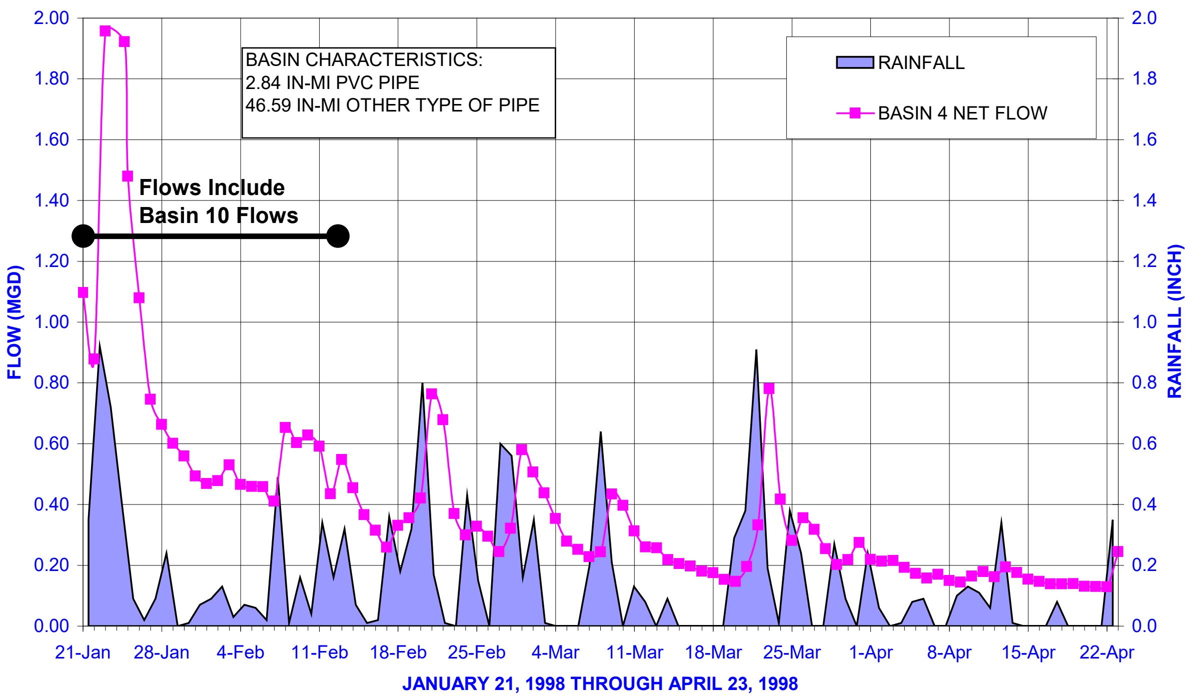


FIGURE VI-10
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #6

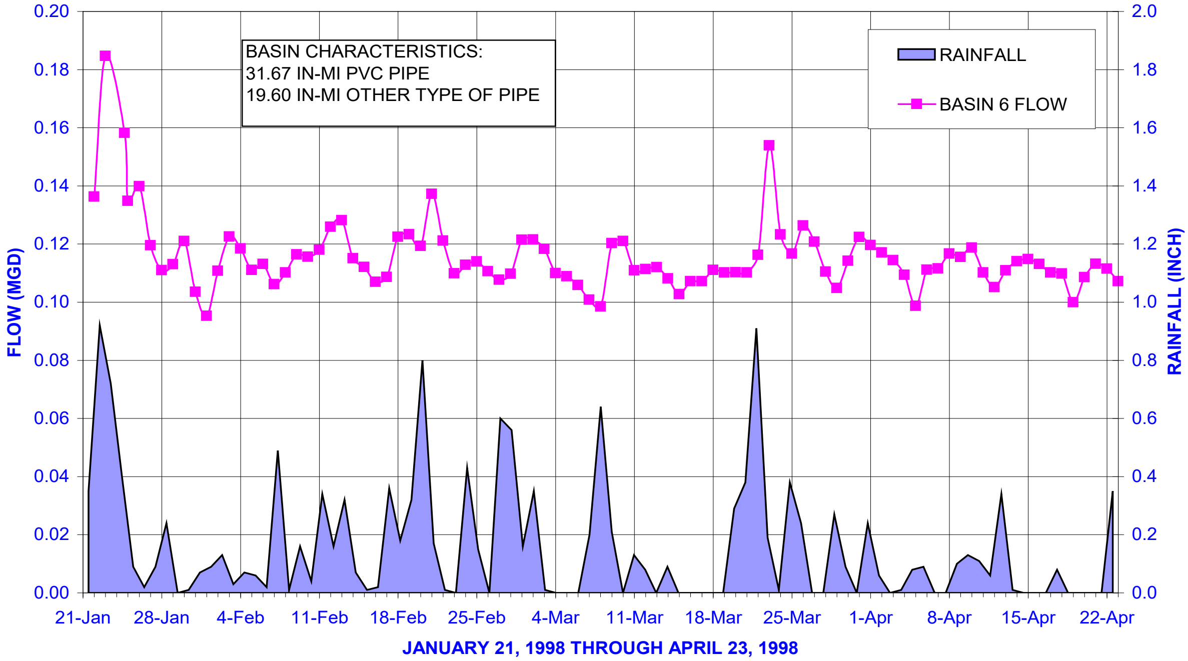


FIGURE VI-11
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #7

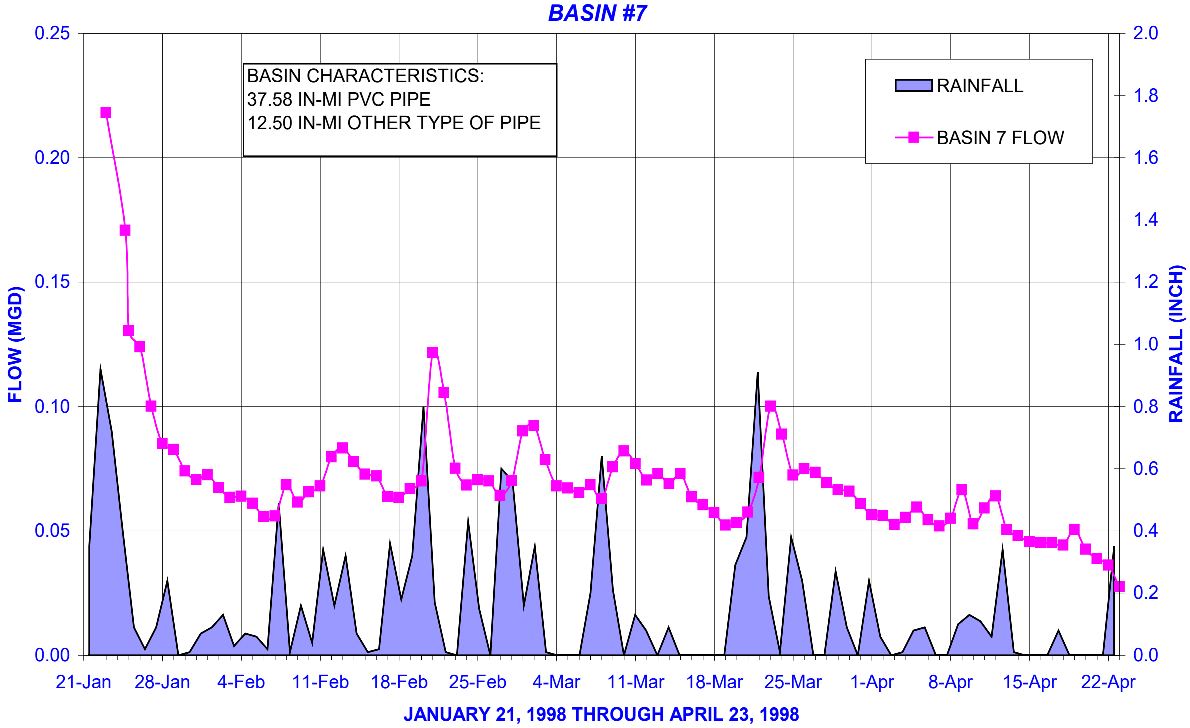


FIGURE VI-12
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #8

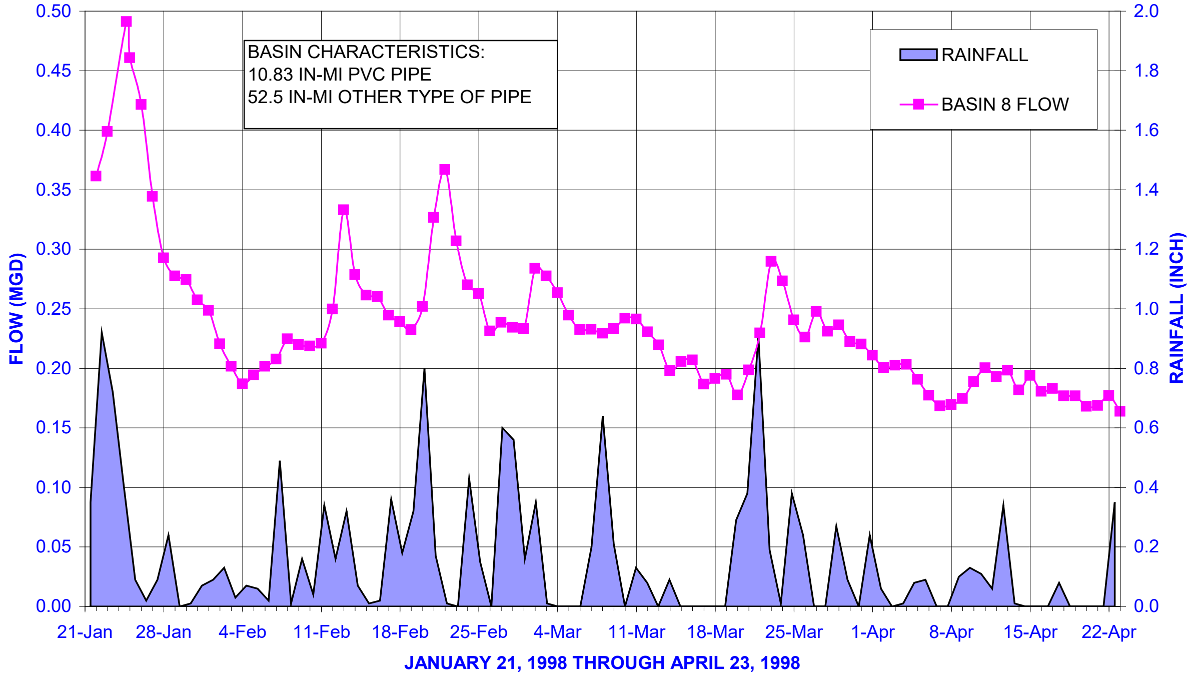


FIGURE VI-13
CHEHALIS/NAPAVINE/LCSD No. 1 FLOW MONITORING STUDY
RAINFALL & FLOW vs TIME
BASIN #9

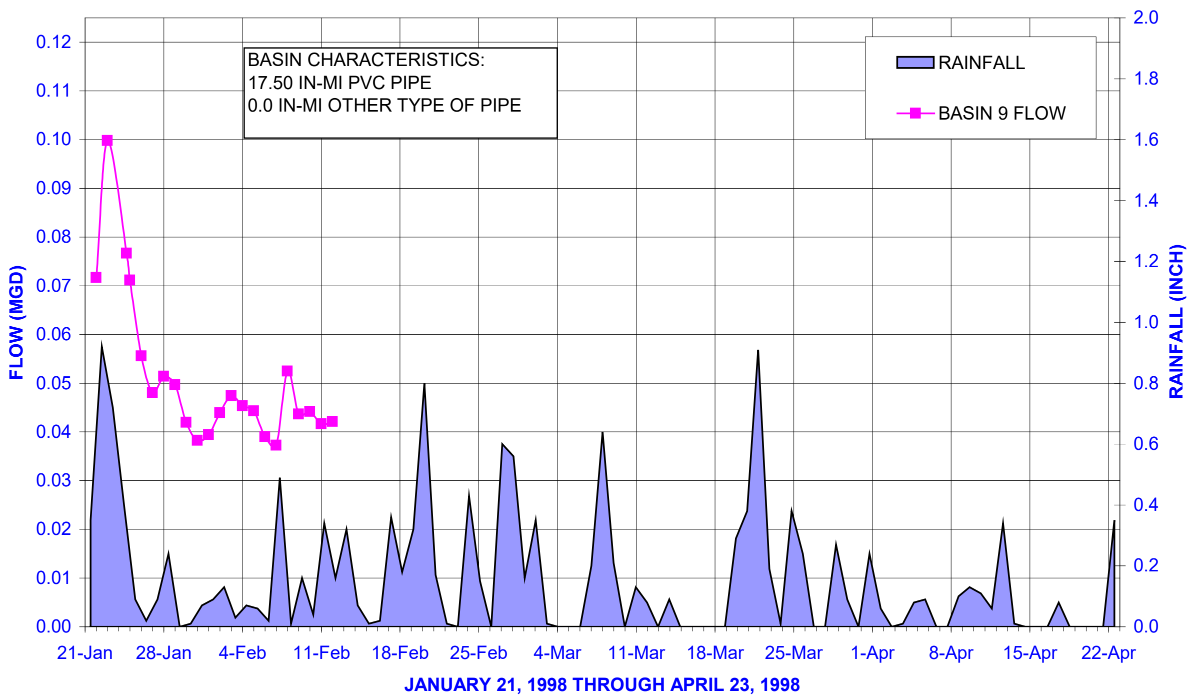


TABLE VII-1
SOIL AQUIFER TREATMENT
CHEMICAL REMOVAL DURING ARTIFICIAL RECHARGE BY RAPID INFILTRATION

Analyte	Amount Removal (1)	Removal Mechanism	Comments
Dissolved solids	Minimal (0%)	Equilibrium	Groundwater eventually is similar to source water.
Suspended solids	Complete (100%)	Filtering	Soils can't be too permeable (coarse) or suspended solids will be carried to subsurface; intermittent drying periods help to decompose suspended solids; fine to medium texture is ideal.
Nitrogen	Minimal to substantial (20% to 75%)		
Ammonium (NH_4)	Minimal to substantial (up to 95%)	Nitrification	Needs oxygen, thus needs drying cycle; needs some clay to absorb ammonium.
Nitrate (NO_3)	Minimal to substantial (20% to 75%) (expect low end of range for the NW)	Denitrification	Needs anaerobic conditions, thus occurs during wetting cycle; needs organic carbon to support bacterial population; facilitated by wetting schedule; removal decreases with decreasing temperature.
Phosphorus	Substantial (30% to 85%)	Adsorption and subsequent precipitation to amorphous or crystalline compounds	Precipitation is very slow; rate is proportional to P loading rate; P continuously removed as water moves in subsurface.
Metals	Substantial (up to 50%) for: Zn, Cu, Hg Negligible (0%) for: Cd Slightly (20%) for: Pb	Cation exchange with clays and organics; sorption, chelation; physical filtration of large molecules containing metals	Metals can accumulate, thus overall removal could decrease with time.
Boron	Variable	Absorbed to Fe & Al hydroxide coatings on clay or Fe and Al oxides, or coating on ferromag minerals	Removal depends on lithology.
Bacterial	Complete (80% to 100%)	Adsorbed, physical straining, non-native dieoff	Needs to move a certain distance (100m) laterally, within the subsurface for complete removal.
Viruses	Complete, subject to debate (80% to 100%)	Adsorbed, immobilized	Dominant process is pH dependent; concentration decreases with distance from source; removal complete with chlorination.
TOC (Total Organic Carbon)	Substantial (60%)	Biodegradation	Residual organic carbon may indicate that some of the TOC is not biodegradable and may be anthropogenic compounds (organic pollutants).
BOD	Complete (100%)		Complete regardless of the wetting/drying cycles (anaerobic or aerobic).
COD	Substantial (40% to 60%)	Various chemical reactions	Some of the COD may be due to reduced inorganic compounds (i.e., Fe and Mn).

Note: LOTT Inflow and Infiltration Study and Capital Improvement Plan, Parametrix (1994)
(1) Percentages are based on information in Bouwer (1985)

TABLE VII-2 SOIL AQUIFER TREATMENT CHEMICAL REMOVAL DURING ARTIFICIAL RECHARGE BY RAPID INFILTRATION			
Analyte	**Amount Removal** (1)	**Removal Mechanism**	**Comments**
Dissolved solids	Minimal (0%)	Equilibrium	Groundwater eventually is similar to source water.
Suspended solids	Complete (100%)	Filtering	Soils can't be too permeable (coarse) or suspended solids will be carried to subsurface; intermittent drying periods help to decompose suspended solids; fine to medium texture is ideal.
Nitrogen	Minimal to substantial (20% to 75%)		
Ammonium (NH_4)	Minimal to substantial (up to 95%)	Nitrification	Needs oxygen, thus needs drying cycle; needs some clay to absorb ammonium.
Nitrate (NO_3)	Minimal to substantial (20% to 75%) (expect low end of range for the NW)	Denitrification	Needs anaerobic conditions, thus occurs during wetting cycle; needs organic carbon to support bacterial population; facilitated by wetting schedule; removal decreases with decreasing temperature.
Phosphorus	Substantial (30% to 85%)	Adsorption and subsequent precipitation to amorphous or crystalline compounds	Precipitation is very slow; rate is proportional to P loading rate; P continuously removed as water moves in subsurface.
Metals	Substantial (up to 50%) for: Zn, Cu, Hg Negligible (0%) for: Cd Slightly (20%) for: Pb	Cation exchange with clays and organics; sorption, chelation; physical filtration of large molecules containing metals	Metals can accumulate, thus overall removal could decrease with time.
Boron	Variable	Absorbed to Fe & Al hydroxide coatings on clay or Fe and Al oxides, or coating on ferromag minerals	Removal depends on lithology.
Bacterial	Complete (80% to 100%)	Adsorbed, physical straining, non-native dieoff	Needs to move a certain distance (100m) laterally, within the subsurface for complete removal.
Viruses	Complete, subject to debate (80% to 100%)	Adsorbed, immobilized	Dominant process is pH dependent; concentration decreases with distance from source; removal complete with chlorination.
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Note: LOTT Inflow and Infiltration Study and Capital Improvement Plan, Parametrix (1994)
(1) Percentages are based on information in Bouwer (1985)

TABLE VII-19
CHEHALIS WWTP CAPITAL COST SUMMARY

End Use Alternatives	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge
Plant Alternative	Capital Cost Required to Meet Permit				Capital Cost for Capital Improvements				Capital Cost for Operational Enhancement				Total Capital Cost		
Modify Existing Plant	$ 20.0	$ 13.5	$ 17.5	$ 13.9	$ 3.7	$ 4.1	$ 3.7	$ 4.1	$ 1.7	$ 1.7	$ 1.7	$ 1.7	$ 25.4	$ 19.3	$ 22.9
SBR at Existing Site	$ 21.5	$ 16.0	$ 19.5	$ 16.0	$ 3.1	$ 3.5	$ 3.1	$ 3.5	$ 0.6	$ 0.6	$ 0.6	$ 0.6	$ 25.2	$ 20.1	$ 23.2
SBR at New Site	$ 27.6	$ 22.2	$ 26.0	$ 22.4	$ 3.0	$ 3.4	$ 3.0	$ 3.4	$ 0.6	$ 0.5	$ 0.6	$ 0.5	$ 31.1	$ 26.1	$ 29.5

All Costs in Million Dollars

6 Streamflow Augmentation
$ 19.7
$ 20.1
$ 26.4

TABLE VII-19
CHEHALIS WWTP PRESENT WORTH COST SUMMARY

End Use Alternatives	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation
Plant Alternative	Total Capital Cost				Annual O & M Cost				Present Worth Cost			
Modify Existing Plant	$ 25.3	$ 18.9	$ 22.9		$ -	$ -	$ -		$ -	$ -	$ -	
SBR at Existing Site	$ 25.1	$ 18.5	$ 23.2		$ -	$ -	$ -		$ -	$ -	$ -	
SBR at New Site	$ 31.4	$ 25.0	$ -		$ -	$ -	$ -		$ -	$ -	$ -	

All Costs in Million Dollars

TABLE VII-1
SOIL AQUIFER TREATMENT
CHEMICAL REMOVAL DURING ARTIFICIAL RECHARGE BY RAPID INFILTRATION

Analyte	Amount Removal (1)	Removal Mechanism	Comments
Dissolved solids	Minimal (0%)	Equilibrium	Groundwater eventually is similar to source water.
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Nitrogen	Minimal to substantial (20% to 75%)		
Ammonium (NH_4)	Minimal to substantial (up to 95%)	Nitrification	Needs oxygen, thus needs drying cycle; needs some clay to absorb ammonium.
Nitrate (NO_3)	Minimal to substantial (20% to 75%) (expect low end of range for the NW)	Denitrification	Needs anaerobic conditions, thus occurs during wetting cycle; needs organic carbon to support bacterial population; facilitated by wetting schedule; removal decreases with decreasing temperature.
Phosphorus	Substantial (30% to 85%)	Adsorption and subsequent precipitation to amorphous or crystalline compounds	Precipitation is very slow; rate is proportional to P loading rate; P continuously removed as water moves in subsurface.
Metals	Substantial (up to 50%) for: Zn, Cu, Hg Negligible (0%) for: Cd Slightly (20%) for: Pb	Cation exchange with clays and organics; sorption, chelation; physical filtration of large molecules containing metals	Metals can accumulate, thus overall removal could decrease with time.
Boron	Variable	Absorbed to Fe & Al hydroxide coatings on clay or Fe and Al oxides, or coating on ferromag minerals	Removal depends on lithology.
Bacterial	Complete (80% to 100%)	Adsorbed, physical straining, non-native dieoff	Needs to move a certain distance (100m) laterally, within the subsurface for complete removal.
Viruses	Complete, subject to debate (80% to 100%)	Adsorbed, immobilized	Dominant process is pH dependent; concentration decreases with distance from source; removal complete with chlorination.
TOC (Total Organic Carbon)	Substantial (60%)	Biodegradation	Residual organic carbon may indicate that some of the TOC is not biodegradable and may be anthropogenic compounds (organic pollutants).
BOD	Complete (100%)		Complete regardless of the wetting/drying cycles (anaerobic or aerobic).
COD	Substantial (40% to 60%)	Various chemical reactions	Some of the COD may be due to reduced inorganic compounds (i.e., Fe and Mn).

Note: LOTT Inflow and Infiltration Study and Capital Improvement Plan, Parametrix (1994)
(1) Percentages are based on information in Bouwer (1985)

TABLE VII-2
SOIL AQUIFER TREATMENT
CHEMICAL REMOVAL DURING ARTIFICIAL RECHARGE BY RAPID INFILTRATION

Analyte	Amount Removal (1)	Removal Mechanism	Comments
Dissolved solids	Minimal (0%)	Equilibrium	Groundwater eventually is similar to source water.
Suspended solids	Complete (100%)	Filtering	Soils can't be too permeable (coarse) or suspended solids will be carried to subsurface; intermittent drying periods help to decompose suspended solids; fine to medium texture is ideal.
Nitrogen	Minimal to substantial (20% to 75%)		
Ammonium (NH_4)	Minimal to substantial (up to 95%)	Nitrification	Needs oxygen, thus needs drying cycle; needs some clay to absorb ammonium.
Nitrate (NO_3)	Minimal to substantial (20% to 75%) (expect low end of range for the NW)	Denitrification	Needs anaerobic conditions, thus occurs during wetting cycle; needs organic carbon to support bacterial population; facilitated by wetting schedule; removal decreases with decreasing temperature.
Phosphorus	Substantial (30% to 85%)	Adsorption and subsequent precipitation to amorphous or crystalline compounds	Precipitation is very slow; rate is proportional to P loading rate; P continuously removed as water moves in subsurface.
Metals	Substantial (up to 50%) for: Zn, Cu, Hg Negligible (0%) for: Cd Slightly (20%) for: Pb	Cation exchange with clays and organics; sorption, chelation; physical filtration of large molecules containing metals	Metals can accumulate, thus overall removal could decrease with time.
Boron	Variable	Absorbed to Fe & Al hydroxide coatings on clay or Fe and Al oxides, or coating on ferromag minerals	Removal depends on lithology.
Bacterial	Complete (80% to 100%)	Adsorbed, physical straining, non-native dieoff	Needs to move a certain distance (100m) laterally, within the subsurface for complete removal.
Viruses	Complete, subject to debate (80% to 100%)	Adsorbed, immobilized	Dominant process is pH dependent; concentration decreases with distance from source; removal complete with chlorination.
TOC (Total Organic Carbon)	Substantial (60%)	Biodegradation	Residual organic carbon may indicate that some of the TOC is not biodegradable and may be anthropogenic compounds (organic pollutants).
BOD	Complete (100%)		Complete regardless of the wetting/drying cycles (anaerobic or aerobic).
COD	Substantial (40% to 60%)	Various chemical reactions	Some of the COD may be due to reduced inorganic compounds (i.e., Fe and Mn).

Note: LOTT Inflow and Infiltration Study and Capital Improvement Plan, Parametrix (1994)
(1) Percentages are based on information in Bouwer (1985)

TABLE VII-19
CHEHALIS WWTP CAPITAL COST SUMMARY

End Use Alternatives	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge
Plant Alternative	Capital Cost Required to Meet Permit				Capital Cost for Capital Improvements				Capital Cost for Operational Enhancement				Total Capital Cost		
Modify Existing Plant	$ 20.0	$ 13.5	$ 17.5	$ 13.9	$ 3.7	$ 4.1	$ 3.7	$ 4.1	$ 1.7	$ 1.7	$ 1.7	$ 1.7	$ 25.4	$ 19.3	$ 22.9
SBR at Existing Site	$ 21.5	$ 16.0	$ 19.5	$ 16.0	$ 3.1	$ 3.5	$ 3.1	$ 3.5	$ 0.6	$ 0.6	$ 0.6	$ 0.6	$ 25.2	$ 20.1	$ 23.2
SBR at New Site	$ 27.6	$ 22.2	$ 26.0	$ 22.4	$ 3.0	$ 3.4	$ 3.0	$ 3.4	$ 0.6	$ 0.5	$ 0.6	$ 0.5	$ 31.1	$ 26.1	$ 29.5

All Costs in Million Dollars

6 Streamflow Augmentation
$ 19.7
$ 20.1
$ 26.4

TABLE VII-19
CHEHALIS WWTP PRESENT WORTH COST SUMMARY

End Use Alternatives	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation	2 Discharge Downstream	3 River Enhancement	4Aii Class A RW to Poplars w/ GW Recharge	6 Streamflow Augmentation
Plant Alternative	Total Capital Cost				Annual O & M Cost				Present Worth Cost			
Modify Existing Plant	$ 25.3	$ 18.9	$ 22.9		$ -	$ -	$ -		$ -	$ -	$ -	
SBR at Existing Site	$ 25.1	$ 18.5	$ 23.2		$ -	$ -	$ -		$ -	$ -	$ -	
SBR at New Site	$ 31.4	$ 25.0	$ -		$ -	$ -	$ -		$ -	$ -	$ -	

All Costs in Million Dollars

TABLE IV-2
Influent BOD (mg/L)

Month	average	min	max	Month	average	min	max
Jan-89	--	--	--	Jan-93	28.50	28.50	28.50
Feb-89	--	--	--	Feb-93	61.50	58.50	64.50
Mar-89	18.00	7.00	29.00	Mar-93	81.75	54.00	109.50
Apr-89	--	--	--	Apr-93	--	--	--
May-89	46.90	26.00	67.80	May-93	54.00	54.00	54.00
Jun-89	--	--	--	Jun-93	75.00	75.00	75.00
Jul-89	--	--	--	Jul-93	57.25	55.50	59.00
Aug-89	--	--	--	Aug-93	53.00	40.00	66.00
Sep-89	86.00	22.00	150.00	Sep-93	63.50	50.00	77.00
Oct-89	40.20	40.20	40.20	Oct-93	82.00	80.00	84.00
Nov-89	12.36	6.12	18.60	Nov-93	61.00	42.00	80.00
Dec-89	18.40	18.40	18.40	Dec-93	--	--	--
Jan-90	--	--	--	Jan-94	34.00	20.00	48.00
Feb-90	--	--	--	Feb-94	--	--	--
Mar-90	14.47	11.10	17.83	Mar-94	--	--	--
Apr-90	36.00	27.00	45.00	Apr-94	63.60	63.60	63.60
May-90	41.10	28.50	53.70	May-94	71.00	56.00	86.00
Jun-90	28.90	19.50	38.30	Jun-94	71.37	59.00	81.60
Jul-90	42.00	42.00	42.00	Jul-94	66.73	53.20	75.50
Aug-90	105.00	76.00	134.00	Aug-94	57.67	44.00	82.00
Sep-90	81.25	79.00	83.50	Sep-94	40.83	36.50	46.00
Oct-90	61.30	61.30	61.30	Oct-94	--	--	--
Nov-90	88.50	88.50	88.50	Nov-94	--	--	--
Dec-90	10.50	10.50	10.50	Dec-94	--	--	--
Jan-91	31.35	27.00	35.70	Jan-95	--	--	--
Feb-91	30.45	23.90	37.00	Feb-95	--	--	--
Mar-91	--	--	--	Mar-95	--	--	--
Apr-91	50.75	39.10	62.40	Apr-95	--	--	--
May-91	32.65	31.30	34.00	May-95	--	--	--
Jun-91	45.80	31.60	60.00	Jun-95	--	--	--
Jul-91	72.00	72.00	72.00	Jul-95	--	--	--
Aug-91	73.33	56.00	100.00	Aug-95	--	--	--
Sep-91	122.00	122.00	122.00	Sep-95	--	--	--
Oct-91	105.00	105.00	105.00	Oct-95	--	--	--
Nov-91	63.00	63.00	63.00	Nov-95	--	--	--
Dec-91	79.50	45.00	114.00	Dec-95	--	--	--
Jan-92	--	--	--	Jan-96	8.16	3.00	18.00
Feb-92	--	--	--	Feb-96	11.41	6.60	17.70
Mar-92	81.00	68.00	94.00	Mar-96	41.58	11.60	89.30
Apr-92	55.00	48.00	62.00	Apr-96	21.19	4.00	39.00
May-92	57.00	38.00	76.00	May-96	45.20	28.00	120.00
Jun-92	82.00	78.00	86.00	Jun-96	118.63	57.00	228.00
Jul-92	105.00	105.00	105.00	Jul-96	130.13	70.50	189.00
Aug-92	129.75	111.00	148.50	Aug-96	131.35	37.00	227.00
Sep-92	168.00	168.00	168.00	Sep-96	90.06	57.50	137.50
Oct-92	72.00	33.00	111.00	Oct-96	71.50	53.00	102.50
Nov-92	73.50	73.50	73.50	Nov-96	57.22	40.00	87.00
Dec-92	--	--	--	Dec-96	40.31	24.00	78.00
				Jan-97	18.98	6.25	42.00
				Feb-97	29.81	20.00	55.00
				Mar-97	25.81	9.00	40.00
				Apr-97	37.94	21.00	72.00
				May-97	56.67	34.50	96.00
				Jun-97	65.50	30.00	90.00

TABLE IV-3
Influent TSS (mg/L)

Month	Influent TSS (mg/L)			Month	Influent TSS (mg/L)		
	average	min	max		average	min	max
Jan-89	--	--	--	Jan-93	59.00	59.00	59.00
Feb-89	--	--	--	Feb-93	41.00	33.00	49.00
Mar-89	16.50	9.00	24.00	Mar-93	22.50	11.00	34.00
Apr-89	--	--	--	Apr-93	34.00	31.00	37.00
May-89	68.75	29.00	108.50	May-93	24.00	12.00	36.00
Jun-89	--	--	--	Jun-93	26.00	26.00	26.00
Jul-89	--	--	--	Jul-93	59.00	25.00	93.00
Aug-89	--	--	--	Aug-93	59.50	57.00	62.00
Sep-89	105.00	54.00	156.00	Sep-93	79.00	70.00	88.00
Oct-89	57.00	57.00	57.00	Oct-93	26.00	26.00	26.00
Nov-89	17.75	12.50	23.00	Nov-93	30.00	25.00	35.00
Dec-89	76.50	66.00	87.00	Dec-93	16.50	16.50	16.50
Jan-90	--	--	--	Jan-94	49.50	35.00	64.00
Feb-90	--	--	--	Feb-94	15.00	15.00	15.00
Mar-90	29.00	28.00	30.00	Mar-94	37.00	37.00	37.00
Apr-90	122.00	81.00	163.00	Apr-94	40.00	32.00	48.00
May-90	58.50	43.00	74.00	May-94	34.50	25.00	44.00
Jun-90	76.00	71.00	81.00	Jun-94	89.50	56.00	152.00
Jul-90	344.50	28.00	661.00	Jul-94	50.33	44.00	54.00
Aug-90	233.00	203.00	263.00	Aug-94	59.75	39.00	86.00
Sep-90	77.00	71.00	83.00	Sep-94	92.25	46.00	126.00
Oct-90	81.00	81.00	81.00	Oct-94	--	--	--
Nov-90	68.00	68.00	68.00	Nov-94	--	--	--
Dec-90	31.00	31.00	31.00	Dec-94	--	--	--
Jan-91	23.50	22.00	25.00	Jan-95	--	--	--
Feb-91	27.50	24.00	31.00	Feb-95	--	--	--
Mar-91	23.00	23.00	23.00	Mar-95	--	--	--
Apr-91	39.50	25.00	54.00	Apr-95	--	--	--
May-91	61.50	30.00	93.00	May-95	--	--	--
Jun-91	41.50	29.00	54.00	Jun-95	--	--	--
Jul-91	155.00	100.00	210.00	Jul-95	--	--	--
Aug-91	59.67	56.00	63.00	Aug-95	--	--	--
Sep-91	91.00	91.00	91.00	Sep-95	--	--	--
Oct-91	83.00	83.00	83.00	Oct-95	--	--	--
Nov-91	43.50	29.00	58.00	Nov-95	--	--	--
Dec-91	52.00	24.00	80.00	Dec-95	--	--	--
Jan-92	--	--	--	Jan-96	19.44	4.00	34.00
Feb-92	--	--	--	Feb-96	30.63	4.00	40.00
Mar-92	112.50	78.00	147.00	Mar-96	40.67	24.00	62.00
Apr-92	46.00	17.00	75.00	Apr-96	14.63	4.00	23.00
May-92	67.00	63.00	71.00	May-96	24.90	11.00	39.00
Jun-92	163.00	134.00	192.00	Jun-96	58.38	3.00	154.00
Jul-92	59.00	30.00	88.00	Jul-96	73.13	22.00	156.00
Aug-92	32.00	29.00	35.00	Aug-96	60.10	21.00	165.00
Sep-92	81.50	79.00	84.00	Sep-96	29.88	22.00	37.00
Oct-92	90.00	64.00	116.00	Oct-96	29.22	14.00	43.00
Nov-92	41.00	41.00	41.00	Nov-96	41.44	19.00	85.00
Dec-92	--	--	--	Dec-96	10.00	1.00	20.00
				Jan-97	6.90	0.00	19.00
				Feb-97	16.50	7.00	28.50
				Mar-97	13.50	3.00	30.50
				Apr-97	15.50	3.00	42.00
				May-97	40.00	14.50	69.50
				Jun-97	37.31	19.50	64.50

TABLE IV-4
Effluent Flow (MDG)

Month	Flow (MGD) average	min	max
Jan-89	--	--	--
Feb-89	--	--	--
Mar-89	--	--	--
Apr-89	--	--	--
May-89	0.18	0.13	0.23
Jun-89	--	--	--
Jul-89	--	--	--
Aug-89	--	--	--
Sep-89	0.11	0.08	0.16
Oct-89	0.16	0.09	0.30
Nov-89	0.49	0.17	1.32
Dec-89	0.61	0.13	1.32
Jan-90	0.81	0.24	2.15
Feb-90	0.99	0.50	1.91
Mar-90	0.47	0.22	0.93
Apr-90	0.26	0.17	0.50
May-90	0.29	0.23	0.40
Jun-90	0.25	0.15	0.32
Jul-90	0.13	0.10	0.17
Aug-90	0.15	0.11	0.61
Sep-90	0.15	0.10	0.31
Oct-90	0.25	0.10	0.67
Nov-90	0.70	0.30	1.22
Dec-90	0.91	0.52	1.98
Jan-91	0.73	0.39	1.44
Feb-91	0.80	0.45	1.99
Mar-91	0.61	0.31	1.33
Apr-91	0.79	0.14	2.12
May-91	0.21	0.12	0.34
Jun-91	0.14	0.12	0.20
Jul-91	0.11	0.09	0.14
Aug-91	0.13	0.10	0.20
Sep-91	0.11	0.09	0.20
Oct-91	0.12	0.10	0.21
Nov-91	0.24	0.10	0.52
Dec-91	0.36	0.21	0.65
Jan-92	0.36	0.21	0.75
Feb-92	0.42	0.22	0.61
Mar-92	0.18	0.10	0.28
Apr-92	0.24	0.10	0.70
May-92	0.14	0.10	0.22
Jun-92	0.09	0.08	0.10
Jul-92	0.09	0.07	0.12
Aug-92	0.09	0.07	0.13
Sep-92	0.10	0.08	0.15
Oct-92	0.10	0.08	0.24
Nov-92	0.24	0.15	0.45
Dec-92	0.41	0.14	0.75
Jan-93	0.36	0.16	0.69
Feb-93	0.20	0.12	0.32
Mar-93	0.27	0.13	0.79
Apr-93	0.38	0.22	0.76
May-93	0.19	0.12	0.26
Jun-93	0.14	0.10	0.20
Jul-93	0.10	0.07	0.16
Aug-93	0.09	0.06	0.14
Sep-93	0.10	0.06	0.12
Oct-93	0.11	0.07	0.14
Nov-93	0.12	0.08	0.14
Dec-93	0.35	0.16	0.92
Jan-94	0.36	0.18	0.71
Feb-94	0.44	0.13	1.22
Mar-94	0.41	0.15	0.80
Apr-94	0.18	0.10	0.28
May-94	0.10	0.06	0.18
Jun-94	0.10	0.08	0.20
Jul-94	0.07	0.05	0.10
Aug-94	0.07	0.05	0.11
Sep-94	0.06	0.04	0.08
Oct-94	0.12	0.05	0.49
Nov-94	0.37	0.15	1.27
Dec-94	0.74	0.40	1.26
Jan-95	0.33	0.21	0.67
Feb-95	0.37	0.17	0.82
Mar-95	0.35	0.15	0.67
Apr-95	0.18	0.11	0.42
May-95	0.11	0.06	0.21
Jun-95	0.07	0.05	0.11
Jul-95	0.06	0.05	0.08
Aug-95	0.05	0.01	0.06
Sep-95	0.07	0.05	0.11
Oct-95	0.12	0.08	0.18
Nov-95	0.39	0.08	0.66
Dec-95	0.76	0.16	2.78
Jan-96	0.36	0.18	0.62
Feb-96	0.52	0.13	1.01
Mar-96	0.13	0.02	0.28
Apr-96	0.28	0.03	0.95
May-96	0.25	0.13	0.43
Jun-96	0.09	0.03	0.17
Jul-96	0.06	0.02	0.07
Aug-96	0.07	0.05	0.11
Sep-96	0.07	0.02	0.13
Oct-96	0.12	0.04	0.20
Nov-96	0.26	0.11	0.77
Dec-96	0.53	0.30	0.73
Jan-97	0.49	0.30	0.72
Feb-97	0.38	0.25	0.62
Mar-97	0.44	0.35	0.51
Apr-97	0.30	0.11	0.43
May-97	0.20	0.12	0.32
Jun-97	0.12	0.04	0.19

Figure IV-9
Influent BOD (mg/l)

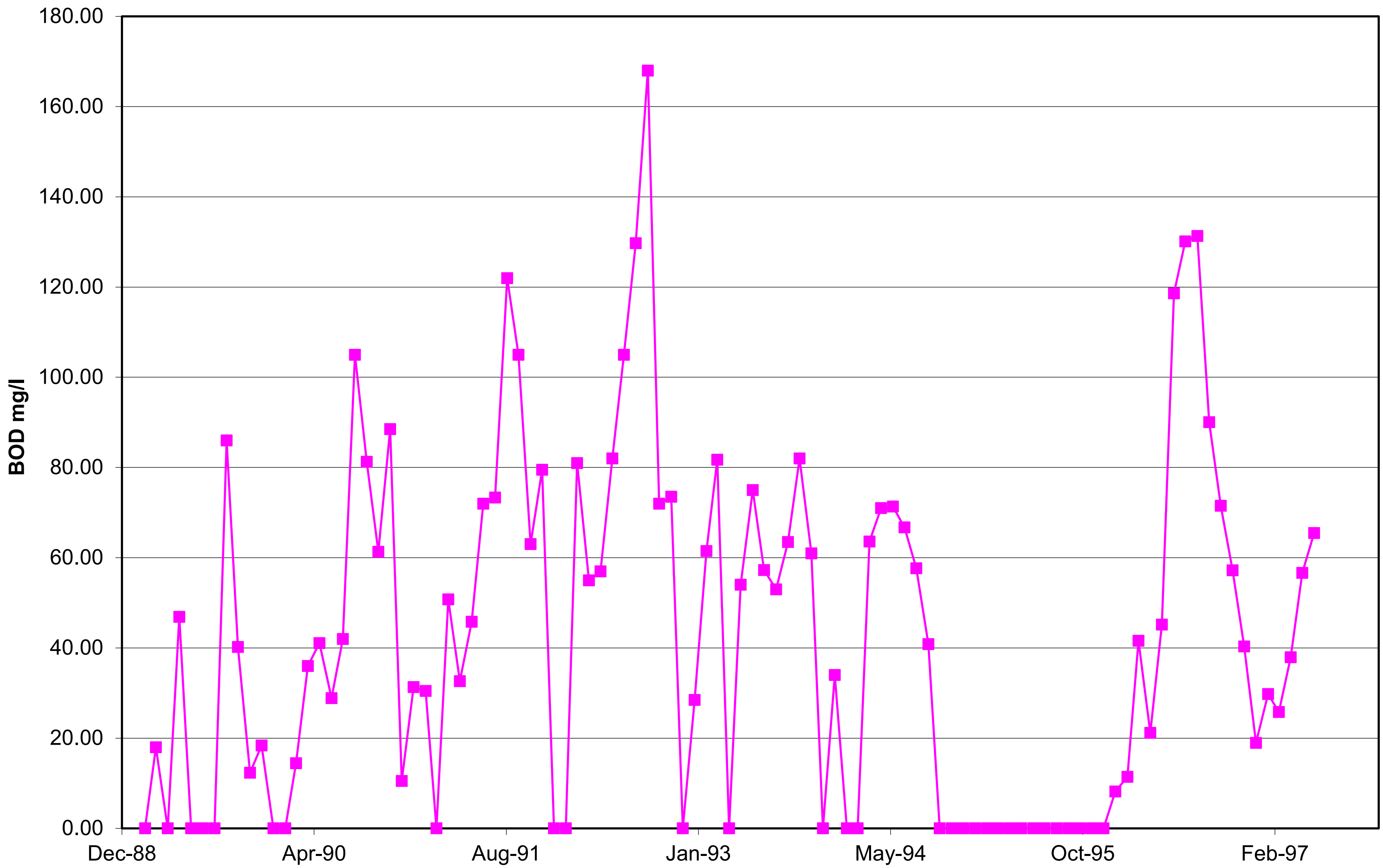
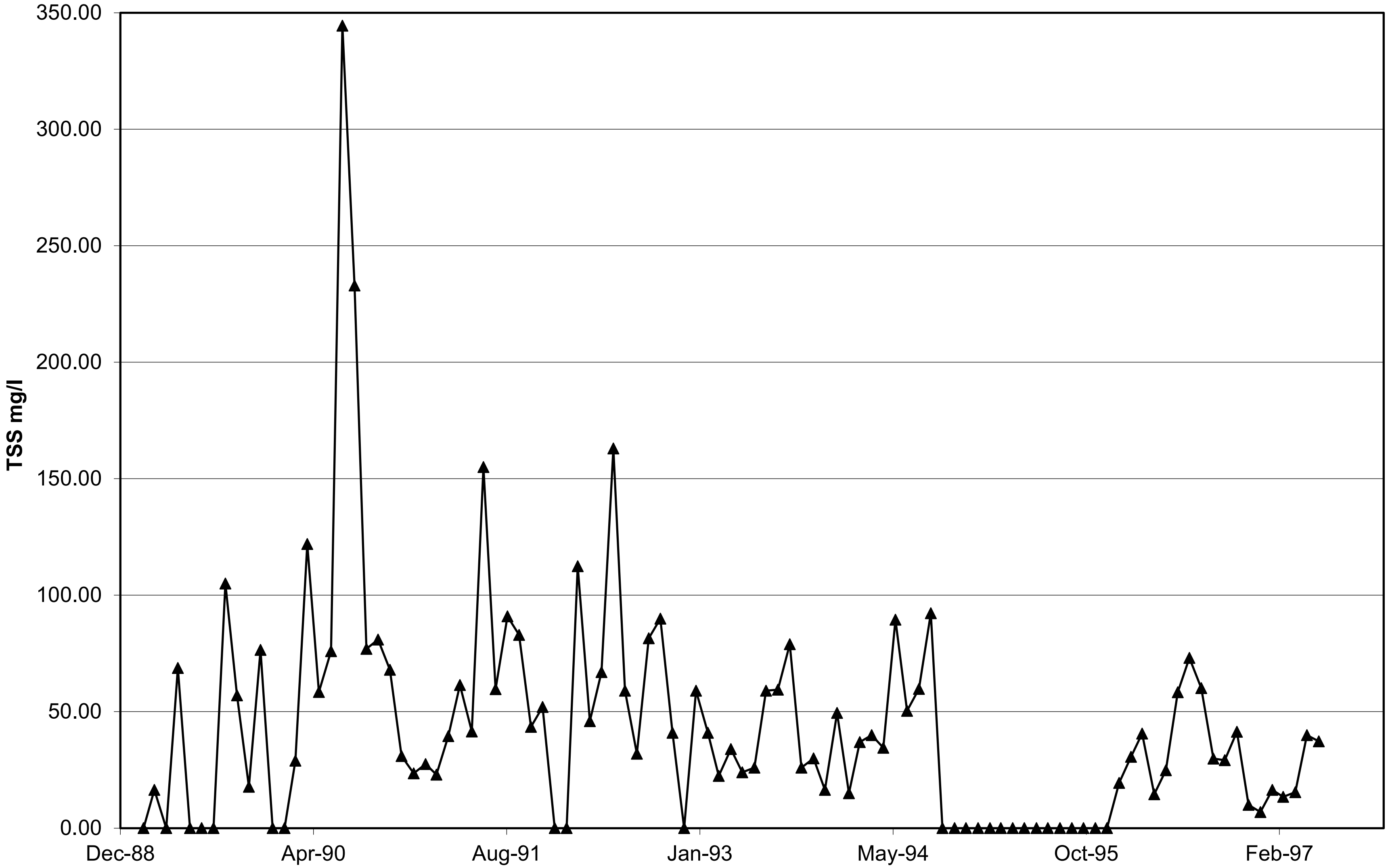


Figure IV-10
Influent TSS (mg/l)



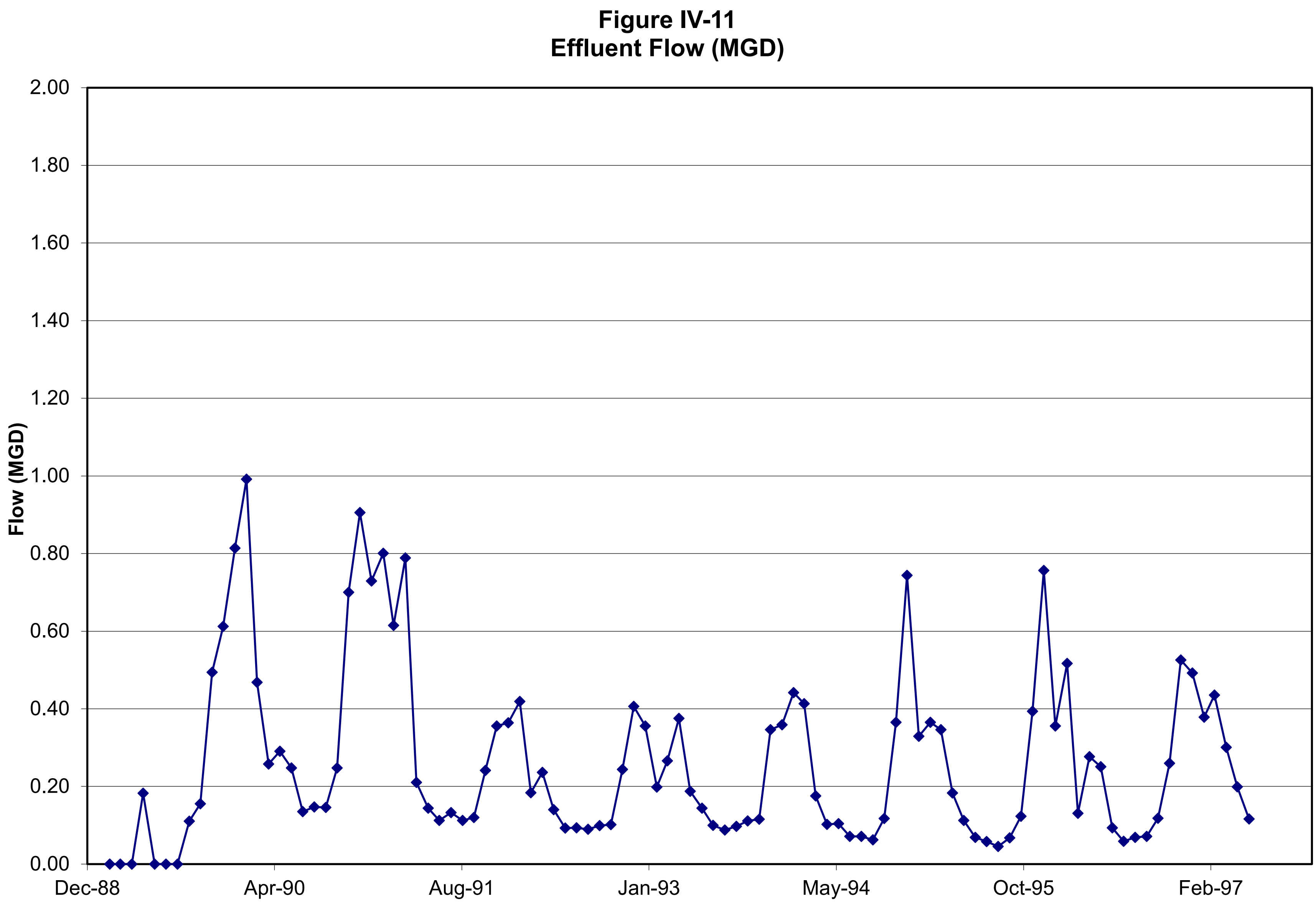


Figure IV-11
Effluent Flow (MGD)
Flow (MGD)
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
Dec-88
Apr-90
Aug-91
Jan-93
May-94
Oct-95
Feb-97