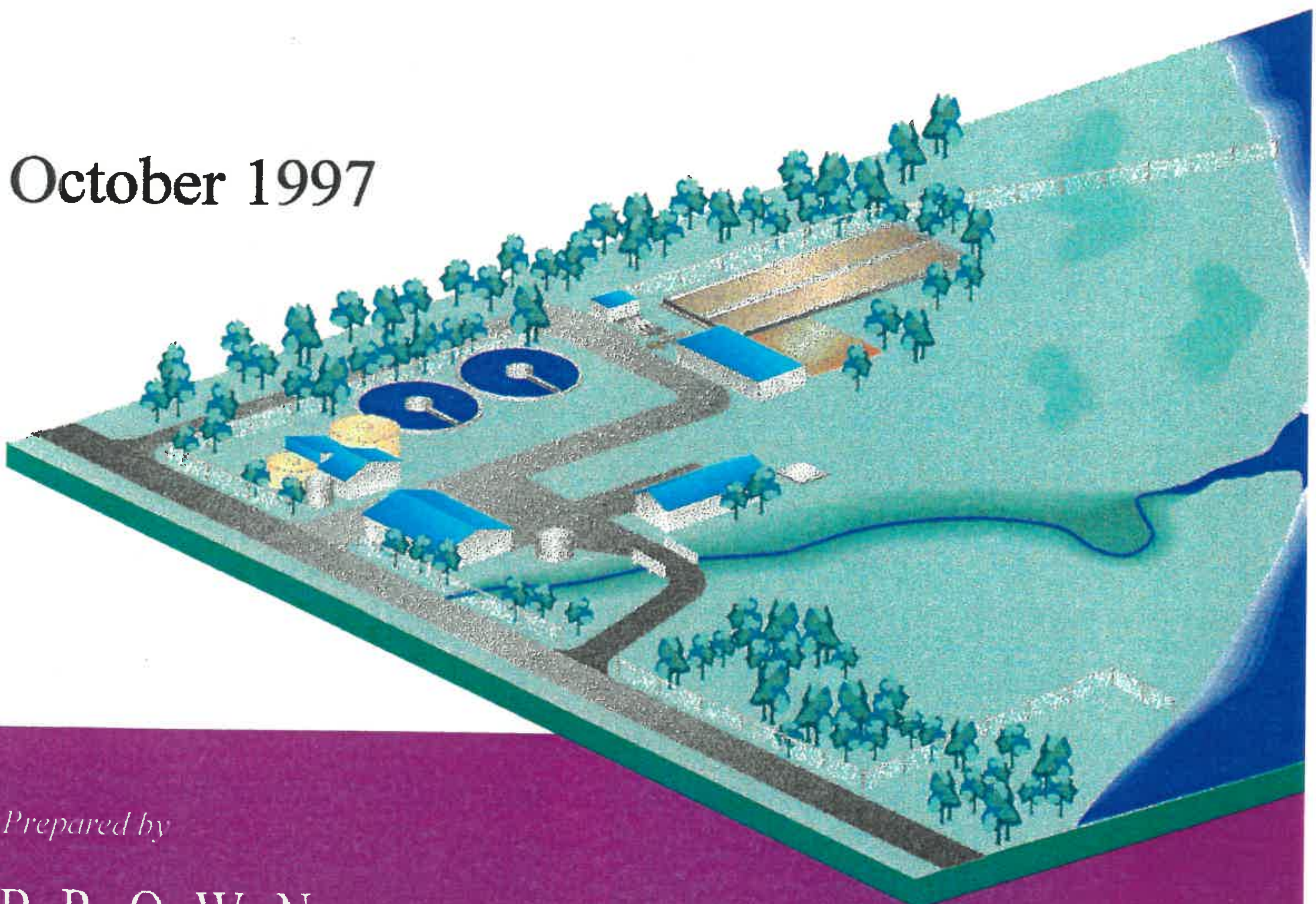
City of Florence

Wastewater Facilities Plan

October 1997



Prepared by

BROWN AND CALDWELL

City of Florence

Wastewater Facilities Plan

October 1997

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JONATHAN D. BEER
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BROWN AND CALDWELL

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September 30, 1997

Mr. Ken Lanfear
Director of Public Works
City of Florence
250 Highway 101
Florence, Oregon 97439

13-4141

Dear Mr. Lanfear:

We are pleased to present this final copy of the City of Florence Wastewater Facilities Plan. In August 1996, we began developing your plan to meet the city's wastewater collection and treatment needs through the year 2020. The city has long recognized its responsibility to protect the environment and provide efficient municipal services. New regulatory requirements and the end of the extended drought period have resulted in occasional discharge permit violations. This plan will allow the city to move forward quickly to implement the long-term improvements and avoid future violations.

The study begins with an evaluation of the study area characteristics and the existing treatment plant. Note that the study area limits have been modified since the draft report was published. Wastewater flow and loading characteristics are then developed, followed by a review of the regulatory requirements. Treatment alternatives are developed and evaluated on the basis of cost and non-cost criteria to a select a recommended plan.

The recommended plan focuses on meeting the current Department of Environmental Quality (DEQ) requirements while making provisions for future system expansion. The technology selected offers a high level of reliable treatment as well as economical and flexible operation. Innovative refinements to proven technology will allow this facility to accommodate more stringent future treatment limits.

A planning effort of this magnitude requires the cooperation of many participants. We acknowledge the considerable efforts of the City of Florence Public Works Department, the City Council, and other city staff. Many hours were spent providing us with background data, brainstorming alternatives, and reviewing draft report sections. We recognize the DEQ's efforts in providing guidance and support for this project.

We congratulate the city on the successful completion of this important planning document. Our entire team is eager to begin implementing your final, approved plan.

Very truly yours,

BROWN AND CALDWELL

John Holroyd
Project Manager

Jon Beer
Project Engineer

JEH:ps

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CHAPTER 1

EXECUTIVE SUMMARY

This facilities plan outlines how the City of Florence will meet the community's wastewater treatment needs for the next 20 years. The development of planning information is provided first. The plan then presents a range of alternatives to meet the wastewater treatment requirements resulting from growth within the study area and more stringent regulations. The alternatives are evaluated in detail to determine the best plan for the city. A recommended plan is developed based on the evaluation of economic and non economic comparisons. Costs and construction phasing opportunities for the recommended plan are discussed.

CURRENT WASTEWATER SYSTEM

The City of Florence's existing treatment plant uses a conventional activated sludge process. The process includes preliminary screening and grit removal, secondary treatment, disinfection by chlorine, and sludge handling to treat biological solids created during the process. Secondary treatment is a biological process that uses bacteria to consume dissolved organic material from the wastewater. Solids are created during the secondary process. The residual solids, known as sludge, then are collected for digestion, which breaks the sludge down further and makes it more stable. The digested sludge is then disposed of through beneficial agricultural use on land. The treated liquid stream, or effluent, is disinfected and discharged to the Siuslaw River about 4 miles upstream of the mouth of the river.

The existing plant has limitations in both the amount of wastewater it can treat and the degree of treatment which can be achieved. Reliability is low due to the age of some of the components and the lack of redundancy in major process units. As a result, several violations of the discharge permit have occurred in recent years, leading to concerns about the adequacy of the plant.

The collection system includes gravity sewers, pump stations, and pressure mains. The entire system is divided into a number of basins. The gravity sewers within a basin collect wastewater from users and convey it to a pump station. Each pump station pumps the wastewater through a pressure main to the treatment plant.

Many of the sewers in the older parts of town are in poor condition, allowing stormwater to enter the system, causing excessively high flows during the winter. Furthermore, the largest pump station is unable to handle the high flows during rainstorms, resulting in bypasses of untreated wastewater to the river.

FUTURE WASTEWATER TREATMENT NEEDS

Future needs for wastewater treatment are determined in part by growth projections for the study area. Growth projections are used to predict future wastewater flows, which in turn, govern the sizing of treatment processes.

STUDY AREA

The study area encompasses the proposed Florence Urban Growth Boundary (UGB). The proposed UGB is the same as the existing UGB with the exception of two additions on the east side. Comprehensive wastewater planning involves consideration of both physical and socioeconomic characteristics within the study area. Physical elements include climate, topography, soils, and local water resources. Socioeconomic characteristics include population, employment outlook, and growth projections.

Climate

The coastal climate is characterized by mild temperatures and frequent precipitation. The average monthly temperatures vary from about 44 degrees in January to about 60 degrees in August. Precipitation averages about 72 inches, mostly in the form of rain. About 70 percent of the rainfall occurs in November through March.

Topography

The study area is in a coastal terrace area characterized by gently rolling terrain dominated by sand dune formations. Slopes range from 0 to 10 percent, flat enough for development in most of the study area. Land elevations range from about 10 feet to 100 feet above sea level.

Soils

Most of the Florence area soils are derived from sand dunes. Formations include active dunes, stabilized dunes, and deflation plains. The active and stabilized dunes are composed of sandy, well drained soil. Deflation plains, consisting of interdune areas eroded by wind down to the level of the summer groundwater table, are poorly drained and subject to high groundwater. Sewers in these areas are subject to corrosion and infiltration of groundwater. Much of the geology of the Florence area comprises unconsolidated sand and layers of compressible organic materials. The unconsolidated sand could cause instability during an earthquake. The presence of compressible organic materials may require that special foundations be used for treatment plant structures to prevent settling.

Water Resources

The principle water resource for planning purposes is the Siuslaw River and its estuary. The river drains an area of about 770 square miles. The average annual discharge is 2,400 cubic feet per second (cfs). The minimum summertime flows are about 100 cfs.

The river and estuary are heavily used for recreation, mainly fishing and boating. Recreational crabbing and clamming are prevalent in the area.

As required by the Clean Water Act, the Oregon Department of Environmental Quality maintains a list of streams for which one or more water quality parameters exceed the standards set by the Act. The Siuslaw exceeds the standards for temperature in the summer. Consequently, no measurable increase in stream temperature is allowed as a result of treatment plant discharges.

Population

The current service population is estimated to be 6,401. Service is currently limited to areas within the city limits. The estimated population for the entire UGB is 7,856. An average annual growth rate of 3.5 percent was assumed for the study period. This is within the 2.3 to 3.7 percent range assumed for the Comprehensive Plan. The projected populations presented in Table 1-1 are based on applying the assumed growth rate to the current estimated populations. Because the wastewater service area is expected to expand to the entire UGB, the design population is selected as the projected UGB population of 17,937.

Table 1-1. Florence City and UGB Population Projections.

	Population[a]	
Year	City	UGB
1996	6,401	7,856
2020	14,617	17,937

Note: [a]Projections based on 3.5 percent annual growth rate.

WASTEWATER CHARACTERISTICS

The primary wastewater characteristics important to treatment plant design are flows and waste loads. Flow is a measure of the volume of liquid entering the plant, normally expressed in millions of gallons per day (mgd). Waste loads are measures of the strength of the waste. The two types of loads important for design are total suspended solids (TSS) and biochemical oxygen demand (BOD). TSS is a measure of the amount of particulate matter that settles if the wastewater is left undisturbed. BOD represents the amount of oxygen that would be depleted from the water if the waste were allowed to remain in the water and degrade. TSS and BOD are typically measured in milligrams per liter or pounds per day (ppd).

The existing flows and loads are estimated from the current treatment plant records. The design flows and loads are projected from the current values based on increase in population. Other factors are also considered, including sewer rehabilitation efforts and rainfall statistics. The current and design flows and loads are presented in Table 1-2.

Table 1-2. Wastewater Flows and Loads

Item	Current value	Design value
Flow		
Average dry weather, mgd	0.7	1.9
Peak wet weather, mgd	3.6	6.9
BOD		
Average, ppd	1,900	5,300
Maximum month, ppd	2,500	7,000
TSS		
Average, ppd	1,350	3,800
Maximum month, ppd	1,700	4,800

WASTEWATER TREATMENT ALTERNATIVES

Major improvements to the existing wastewater system are necessary to treat the flows and loads projected over the next 20 years. Several alternatives were developed and evaluated to develop the program best suited to the city's needs.

ALTERNATIVE DEVELOPMENT

Numerous alternatives were considered initially to ensure that all suitable processes would be evaluated. Many of the alternatives were screened out using such criteria as economics, reliability, public acceptance, and ease of implementation. The remaining alternatives were evaluated in detail.

Liquid Stream Alternatives

The three liquid stream alternatives evaluated in detail were:

- activated sludge
- trickling filter/solids contact (TF/SC)
- sequencing batch reactor (SBR).

Site plans, design data tables, and budgetary costs were developed for each alternative. All three alternatives would utilize similar preliminary treatment at the head end of the plant: screening of larger debris and removal of grit. Each alternative would also include ultraviolet disinfection of the effluent. The biological, or secondary, portion of the treatment differs for each alternative. The alternatives are described briefly below.

Activated Sludge. This alternative utilizes the same process currently used at the treatment plant. In this process, the wastewater is aerated in large tanks with a high concentration of microorganisms that break down the sewage constituents. It is the most commonly used process for wastewater treatment in the United States, owing to its simplicity, reliability, and flexibility.

TF/SC. In this alternative, the wastewater is pumped over a high structure filled with plastic honeycomb media. As the wastewater trickles down through the media, microorganisms attached to the media break down the sewage. Further treatment is provided by a small aeration basin similar to that in the activated sludge process, only much smaller. The TF/SC is the most stable and energy efficient of the alternatives. However, it is less flexible than activated sludge in that the entire filter, designed for peak loads 20 years in the future, must be used even at the lighter initial loads. Operating in this lightly loaded condition could result in less effective treatment.

SBR. This alternative is a batch process in which biological treatment and final clarification take place in one tank at separate times during a cycle. The biological treatment is similar to activated sludge; the wastewater is aerated in a large tank with diffused air. The clarified effluent is decanted from the top of the same tank in a batches during a quiescent part of the cycle. This process has not been widely used until recently because it requires a complex control system to time the events that occur during each cycle. With the advent of computers, the control systems have become more practical. The SBR process consumes the most energy of the alternatives because the wastewater is aerated for a longer period. The process is flexible in operation because the cycle timing can be adjusted to meet differing conditions. However, it is inflexible with respect to future expansion because an additional full-sized tank must be added when expansion is required.

Solids Handling Options

The solids removed from the wastewater stream form sludge that must be treated further before disposal. Treated sludge is often referred to as biosolids. Biosolids can be classified as Class A or Class B. Class B biosolids receive some treatment and stabilization, but still contain substantial numbers of pathogens. The existing treatment plant currently produces Class B biosolids. Class A biosolids receive additional treatment that reduces the number of pathogens to a safe level. Class A biosolids provide greater flexibility in disposal because there is little restriction on their use or disposal. However, Class A biosolids are more costly to produce than are Class B biosolids. Options were developed for producing both Class A and Class B biosolids.

Four options were developed for detailed evaluation. These are:

- Autothermal thermophilic aerobic digestion (ATAD) with dewatering of digested sludge.
- Anaerobic digestion with dewatering of digested sludge.
- Anaerobic digestion with dewatering and composting of digested sludge.
- Anaerobic digestion with facultative sludge lagoon (FSL) storage.

Major equipment requirements, land requirements, and budgetary costs were developed for each option. The options are described briefly below.

ATAD With Dewatering. This process produces Class A biosolids. In this process, sludge is digested aerobically (in the presence of oxygen) at high temperatures (about 140 degrees F). No external heating is required; the biological process produces enough heat to maintain the required temperature in the insulated tanks. A large amount of energy is consumed in providing the air for the process. There is a high potential for odor from the digestion process and from the finished product. Dewatering the digested sludge would increase the solids content from about 2 percent to 20 percent, as well as reducing the odor potential of the final product.

Anaerobic Digestion With Dewatering. This process produces Class B biosolids. In this process, sludge is digested anaerobically (in the absence of oxygen) at medium temperature (about 100 degrees F). External heating is required; however, no external energy is consumed because the methane gas produced by the process is used to generate the heat. Anaerobic digestion is the process currently used for sludge treatment at the plant. Dewatering increases the solids content to about 20 percent, reducing the volume of sludge to haul.

Anaerobic Digestion With Dewatering And Composting. This process is similar to that above, except that the final product is composted to produce Class A biosolids. This process would provide the highest quality product. Composting produces substantial odors, but would take place at a remote site with sufficient buffer.

Anaerobic Digestion With FSL. In this option, the sludge is digested anaerobically and then hauled as a liquid to an FSL for long term storage. An FSL is a lagoon in which the top portion is maintained aerobic from oxygen produced by algae while the lower portion containing the sludge is anaerobic. The aerobic cap prevents odors from developing. The FSL provides long term storage of sludge, allowing flexibility in disposal. A dredge is used to remove sludge from the FSL for application on land.

Collection System

A computer simulation was performed using a model of the collection system under current and future flow conditions. From the results of the simulation, one alternative was developed for improving the system to meet the needs in the design year. The improvements include a new interceptor from the north end of the study area to the treatment plant. The northern section of the pipeline would include two new pump stations. A new pump station at the treatment plant would also be required to lift the flow from the gravity interceptor into the headworks of the plant. Existing sections of piping that are structurally deficient would be replaced to reduce the amount of infiltration into the system.

ALTERNATIVE EVALUATION

The treatment alternatives were evaluated and ranked on the basis of economic and non-economic criteria. The ranking was used in selecting the recommended plan.

Liquid Stream Alternatives

The capital, annual, and present worth costs of the three alternatives are summarized in Table 1-3. These costs include a 15 percent contingency, but do not include costs for engineering and contract administration. As the table shows, SBR has the lowest capital cost and total present worth, but the cost differences among the alternatives are nearly insignificant within the accuracy of the cost estimate. Because the costs are so similar, non-economic factors play a more significant role in alternative selection.

Table 1-3. Cost Comparison of Treatment Alternatives

Cost item, $1,000	Alternative		
	Activated sludge	TF/SC	SBR
Capital cost	9,963	10,069	9,365
Annual cost	448	383	435
Present worth of annual cost[a]	4,394	3,755	4,271
Total present worth[a]	14,357	13,824	13,636

Notes: [a]Present worth based on discount rate of 8 percent and study period of 20 years.

The ranking of the alternatives based on economic and non-economic factors is summarized in Table 1-4. From this table it is clear that the activated sludge alternative is favored in most respects. Therefore, the activated sludge alternative was selected for the recommended plan.

Table 1-4. Summary of Treatment Alternative Rankings

Criteria	Alternative Ranking		
	A/S	TF/SC	SBR
Environmental impact	1	2	3
Reliability	1	1	2
Flexibility in expansion	1	3	2
Flexibility in operation	1	2	1
Aesthetics	1	2	3
Economics	2	1	1

Solids Handling Options

The capital, annual, and present worth costs of the four options are summarized in Table 1-5. The costs of land for sludge processing and land application are included. These costs also include a 15 percent contingency, but do not include costs for engineering and contract administration. As the table shows, the option of anaerobic digestion with FSL storage has the lowest cost. Dewatering could be competitive if sludge hauling distances were greater than 60 miles. It is assumed that sludge application sites will be found within 10 to 20 miles.

Table 1-5. Cost Comparison of Solids Handling Options

Cost item, $1,000	Option			
	ATAD	Dewatering	Composting	FSL
Construction cost	3,839	4,087	4,006	3,029
Annual cost	116	88	137	110
Present worth of annual cost[a]	1,140	865	1,341	1,082
Total present worth[a]	4,979	4,952	5,347	4,111

Notes: [a]Present worth based on discount rate of 8 percent and study period of 20 years.

The ranking of the options based on economic and non-economic factors is summarized in Table 1-6. The FSL option is ranked the highest in most categories. Because the future conditions and land availability are unclear, flexibility is a particularly important factor. The FSL is the highest ranked option in this category. The FSL option was selected for the recommended plan.

Table 1-6. Summary of Solids Handling Option Rankings

Criteria	Alternative ranking			
	ATAD	Dewatering	Composting	FSL
Environmental impact	2	1	1	2
Ease of implementation	3	2	2	1
Reliability	3	1	2	1
Flexibility	3	2	1	1
Aesthetics	3	1	2	2
Economics	3	2	4	1

RECOMMENDED PLAN

The recommended plan includes major improvements to the treatment plant and the collection system. These improvements are summarized below.

TREATMENT PLANT

The treatment plant requires improvements to both the liquid stream treatment and solids handling processes. Most of the existing unit processes will be entirely replaced.

Liquid Stream Treatment

As discussed in the alternative evaluation above, the recommended alternative for the new treatment plant utilizes the activated sludge process. This alternative provides a reliable system using a process familiar to the operators. The process provides the flexibility needed to handle the wide variation in flows and loads expected over the next 20 years. It also provides flexibility in future expansion.

A site plan of the recommended treatment plant improvements is shown in Figure 1-1. An artist's rendition of the recommended treatment plant is shown on Figure 1-2. Units for future expansion are shown in dashed lines on the site plan. Not all the processes shown for the future would be used; they are shown to illustrate the flexibility in expansion. For example, the aeration basins could be expanded by adding another full-sized tank. Alternatively, primary clarifiers could be added, reducing the load on the aeration basins by about 30 percent. Or primary clarifiers and a trickling filter could be added, possibly eliminating the need for more aeration basins.

Solids Handling

The recommended plan includes upgrading the existing anaerobic digester, adding a second digester, and constructing an FSL at a remote site. Figure 1-3 shows a typical FSL site. Liquid digested sludge can be hauled directly from the digester for land application, or hauled to the FSL for storage until conditions are appropriate for land application. About 150 acres of land will be required to implement this plan. The ideal site would be large enough to accommodate both the FSL and the sludge application on one site. The city should pursue land acquisition as soon as possible.

COLLECTION SYSTEM

Collection system improvements include a new 5.5-mile-long interceptor. At the upper end, along Heceta Beach Road, the pipeline would consist of a 12-inch diameter pressure main. Two pump stations would be required. The pipeline would consist of an 18-inch diameter gravity sewer running south along Oak Street to the airport property. At 31st Street, flow would be diverted from the existing collection system to relieve the overloaded Ivy Street pump station. A 24-inch diameter gravity sewer would carry the flow from this point to the treatment plant.

CAPITAL COSTS

The estimated capital costs for the complete recommended project are summarized in Table 1-7. The table includes the costs for the recommended land acquisition, collection system improvements, and allied costs for engineering and contract administration.

SCHEDULE

The Mutual Agreement and Order (MAO) with the Oregon Department of Environmental Quality (DEQ) stipulates a schedule for the completion of the wastewater system improvements. Based on that schedule, it is expected that plans and specifications for the improvements will be complete by the first of 1999. Construction is expected to take about 16 months, resulting in completion in the spring of 2000. Compliance should be achieved by the late summer of 2000. Most of the recommended improvements will be completed in this first phase. Some portions, particularly in the collection system, can be deferred as discussed in the next section.

Table 1-7. Estimated Capital Costs for Recommended Plan

Item	Costs, $1,000
Liquid stream treatment	
Contractor indirects	469
Influent pumping	368
Yard development	384
Headworks	773
Odor control	237
Aeration basins	866
Blower building	628
Secondary clarifiers	1,381
Yard piping	341
Electrical/instrumentation	1,680
Disinfection	692
Outfall	558
Operations building	287
Subtotal, treatment plant	8,664
Solids Handling	
Anaerobic digestion	1,483
Tank truck	100
FSL	460
Dredge	50
Access road	100
Supernatant irrigation system	50
Subtotal, solids handling	2,243
Collection system	
Gravity interceptor	1,497
Force mains	493
Pump stations	150
Subtotal, collection system	2,140
Subtotal, total project	13,047
Bond at 1 percent	130
Contingency at 15 percent	1,957
Subtotal	15,135
Engineering, admin. at 20 percent	3,027
Subtotal	18,161
Land	450
Total project cost	18,611

PHASING OPPORTUNITIES

Phasing the construction could allow some costs to be deferred to the future. Because phasing incurs costs associated with multiple design and construction contracts, additional mobilization, and loss of economy of scale, an item should be deferred about 10 years to make phasing worthwhile. An exception would be individual pieces of mechanical equipment such as a blower or pump; these items would be worth deferring even a few years.

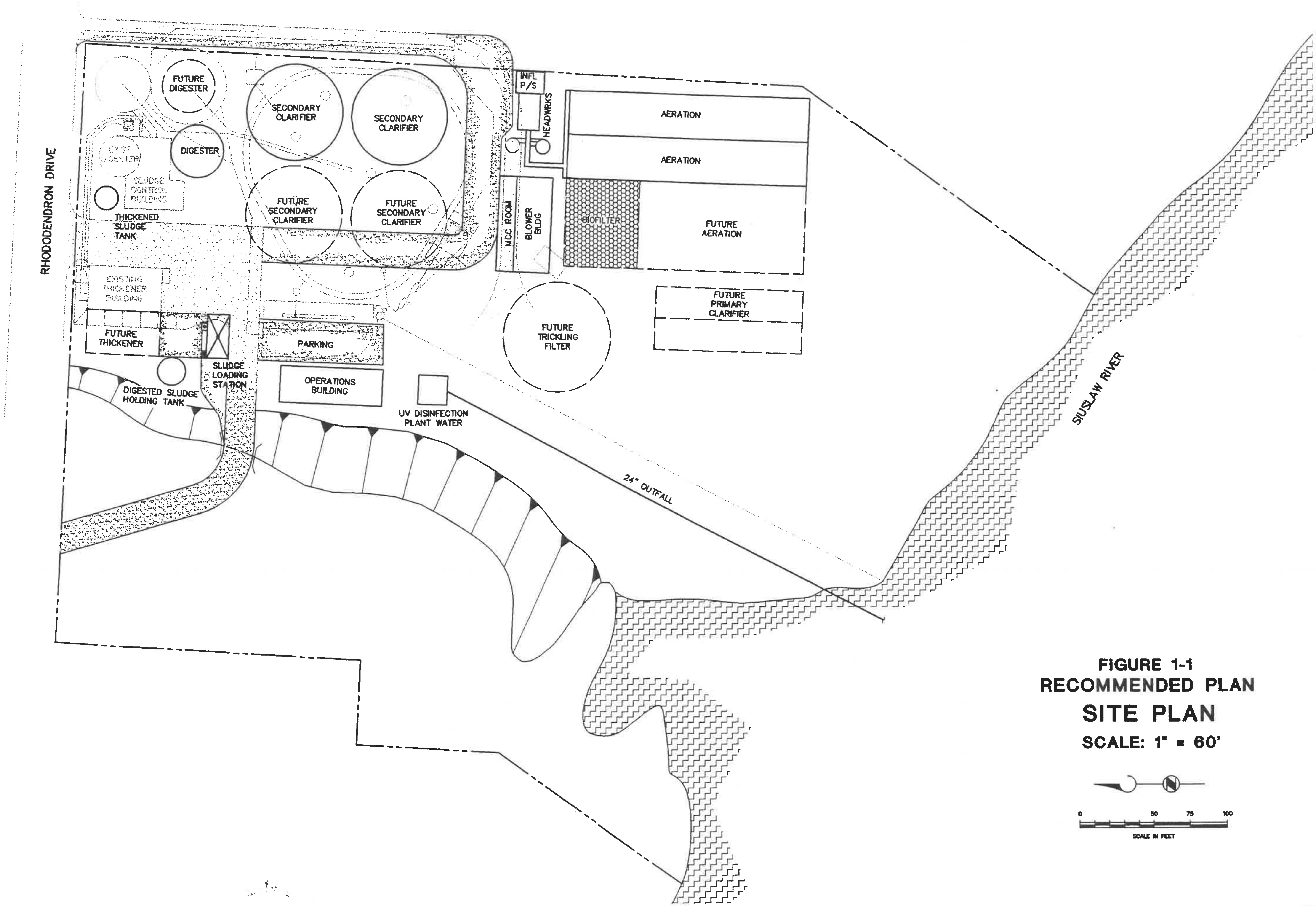

FIGURE 1-1
RECOMMENDED PLAN
SITE PLAN
SCALE: 1" = 60'

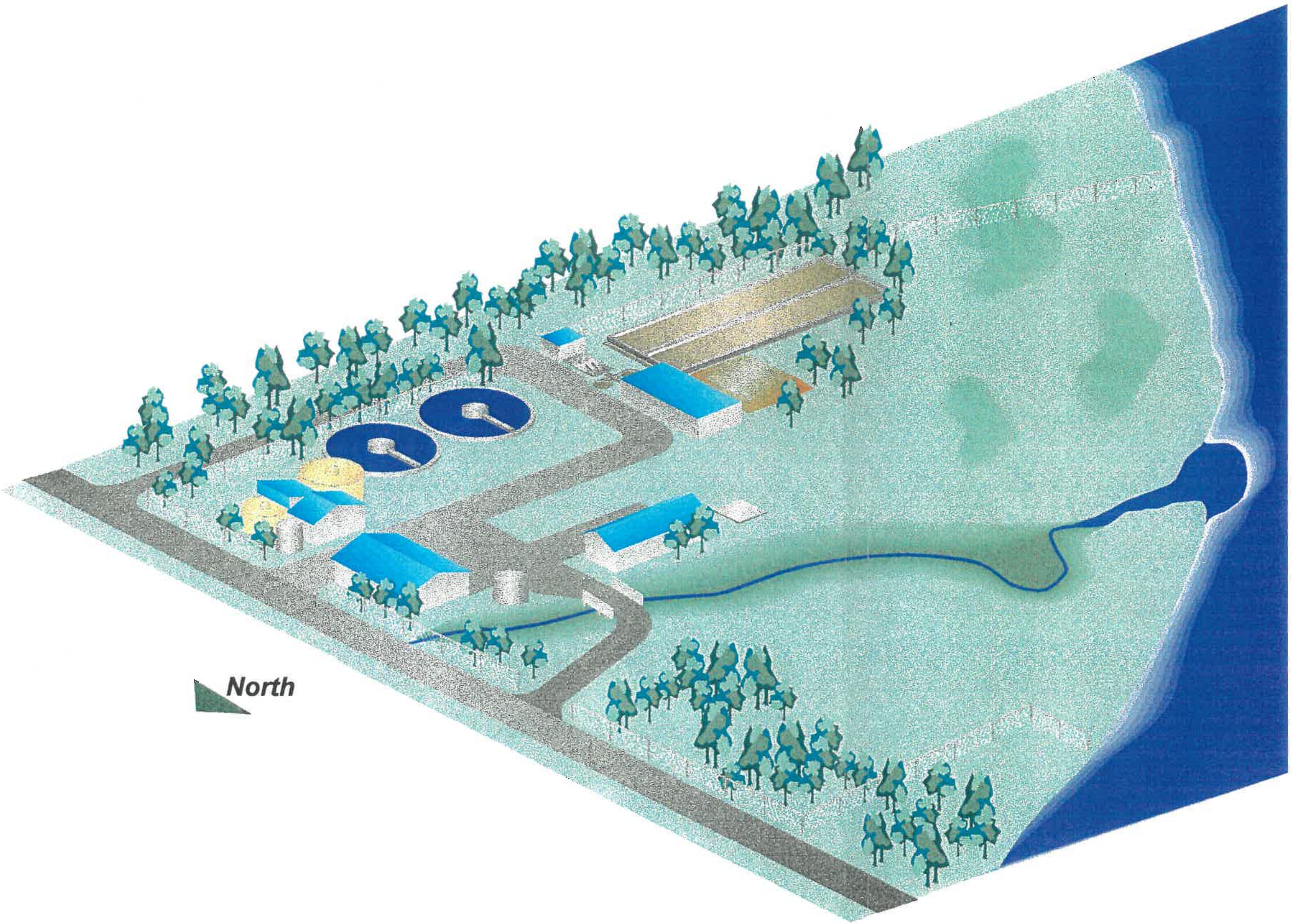

FIGURE 1-2
ARTIST'S RENDITION OF RECOMMENDED PLAN

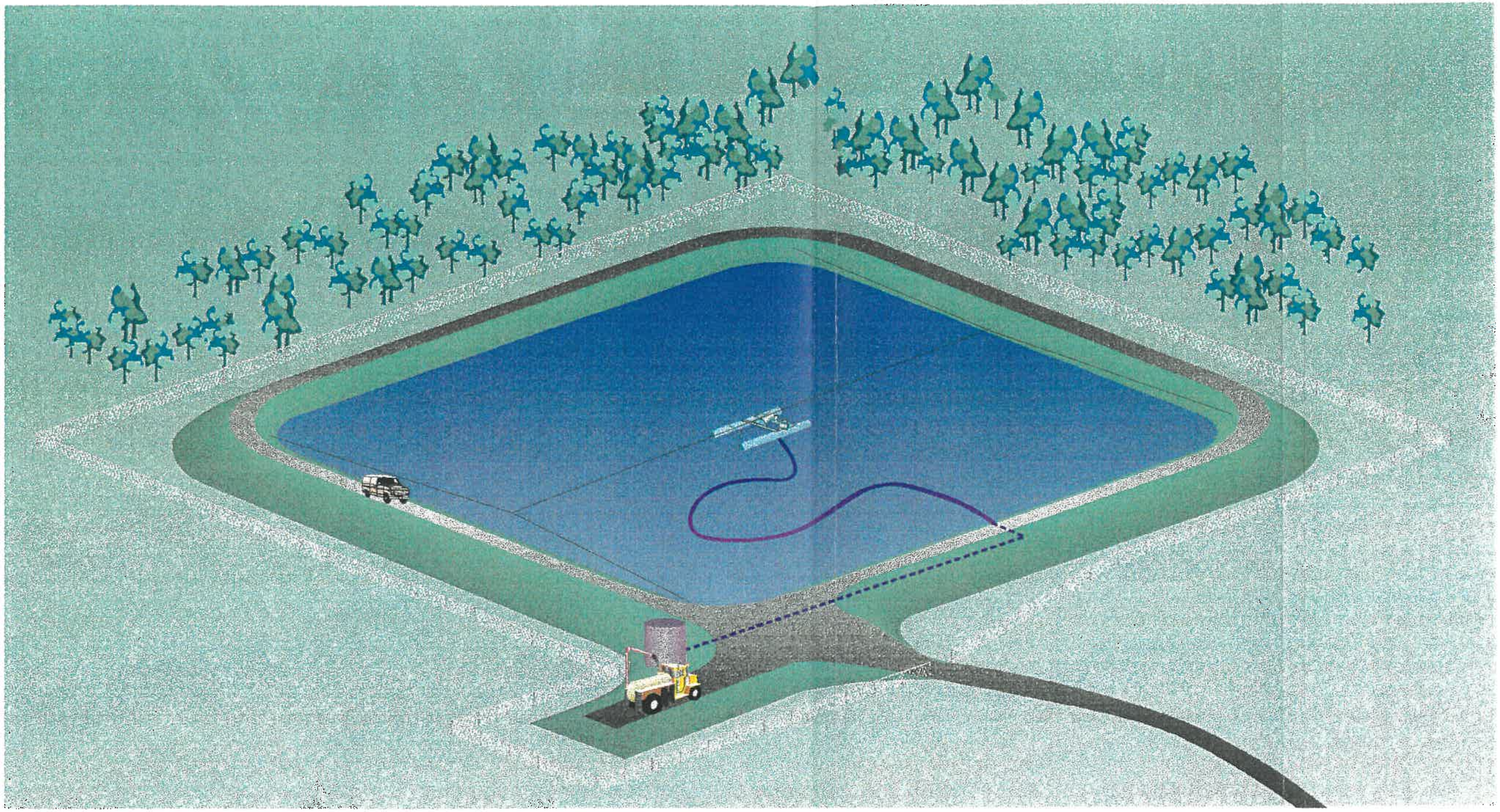

FIGURE 1-3
ARTIST'S RENDITION OF TYPICAL FSL SITE

Components of this project that may have phasing potential are discussed briefly below.

- Collection system. The upper portions of the new interceptor, including the pumping stations and force mains and portions of the gravity sewer will not be necessary until those areas are developed. At this time, only the portion between the treatment plant and 8th Street, which provides relief to the Ivy Street pump station, is necessary.
- Influent pumping. It may be possible to provide two pumps now and add the third later.
- Aeration. Four blowers will not be necessary for several years. Two or three would be sufficient at first. Likewise, some of the diffusers can be installed in the future. Although the aeration basins would have excess capacity at first, it is unlikely that phasing the construction of the basins would be worthwhile. Adding on to the basins would represent a major project with significant mobilization costs and potential disruption to plant operation.
- Disinfection. Although the entire structure would be built initially, some of the actual UV modules could be installed later.

CHAPTER 2

STUDY AREA CHARACTERISTICS

Development of a sound, long-range wastewater management plan for the Florence area requires consideration of both natural and socioeconomic environmental characteristics. The natural environment, including topography, geology, soils, climate, and water resources, affects any wastewater treatment alternative. Factors such as land use, population, and irrigation practices affect the area's natural resources and further affect the availability of land for wastewater treatment and disposal. In this chapter, the study area is defined and the characteristics of both its physical and economic environment are examined.

SEWERAGE STUDY AREA

The City of Florence is situated along the north bank of the Siuslaw River on the central Oregon coast. The city is located in the southern third of the western edge of Lane County. The study area for the facilities plan encompasses the proposed Florence Urban Growth Boundary (UGB). The UGB is currently under review for updating. It is expected to remain essentially unchanged except for minor modifications discussed later in this chapter. Figure 2-1 shows the wastewater facility study area (proposed UGB). The current UGB encompasses about 5,400 acres. As can be seen from the figure, geographic barriers (Siuslaw River and Pacific Ocean) preclude development to the south and west. Development is also somewhat limited to the east by lakes, the drinking water supply basin, and steep contours of the Coast Range foothills. Most development is expected to occur to the north.

PLANNING PERIOD

The facilities planning period must be long enough to allow the city to develop and pursue a long-range program. Key limits to the planning period are the precision of population projections and the ability to estimate industrial and commercial growth. A 20-year planning period is typically used in facilities plans. The start of the period is assumed to be the year that the facilities are first put into operation. For this study, the planning period is 20 years, from the year 2000 through 2020.

PHYSICAL ENVIRONMENT

The physical environment includes the topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items as they relate to the sewerage planning program.

TOPOGRAPHY, GEOLOGY, AND SOILS

The topography, geology, and soils of a region can have a significant effect on the design and construction requirements of sewage works. Topography can determine the route and slope of sewer lines as well as the need for and location of pump stations. The geology and soil conditions in an area can affect construction costs for pipelines and treatment units.

Topography

The city is in a coastal terrace area with gently rolling terrain dominated by dune formations. Slopes range from 0 to 10 percent.[1] The slopes are flat enough for any type of development throughout most of the study area. Land elevations vary from about 10 feet above sea level near the river and coastline to about 100 feet around dune formations. The land generally slopes upward as one proceeds north from the downtown area near the Siuslaw River. The elevation approaches 200 feet further east, along the foothills of the Coast Range mountains.

Geology and Soils

The Florence area is underlain by the Tyee Formation, a thick bed of sandstone and siltstone deposited during the middle Eocene epoch. Although this formation is the dominant outcropping throughout vast areas in the Coast Range, it is overlain by more recent deposits in the Florence area.[2]

The most recent deposits in the Florence area are sand dune formations from the Holocene epoch. These include active dunes, stabilized dunes, and deflation plains. Stabilized dunes are the most widespread geologic feature in developable areas. The resulting soil type on stabilized dunes is Waldport fine sand. It is a deep, excessively drained soil. Although most of the Florence area development has occurred on stabilized sand dunes, construction on this type of deposit requires several precautions. The stabilized dune deposits consist of unconsolidated sand with possible layers of compressible organic materials and peat. Because the deposits are compressible, precautions must be taken in foundation design for heavy structures. Additional geotechnical studies are required to determine the extent of compressibility and resulting foundation requirements. Unconsolidated sand is particularly unstable during earthquakes; liquefaction and major additional settling can occur. The permeability of the soil results in rapid movement of groundwater; hence, the use of septic drain fields is limited. During construction, wind erosion of unprotected exposed soils can easily occur.

Active dunes are structurally similar to stabilized dunes, except that they are still subject to movement due to blowing sand. In addition to all the limitations and potential problems associated with stabilized dunes, active dunes have blowing sand. Structures in these areas require frequent maintenance, including removal of sand and repair of sandblasted surfaces. These areas are generally unsuitable for most development unless stabilized.

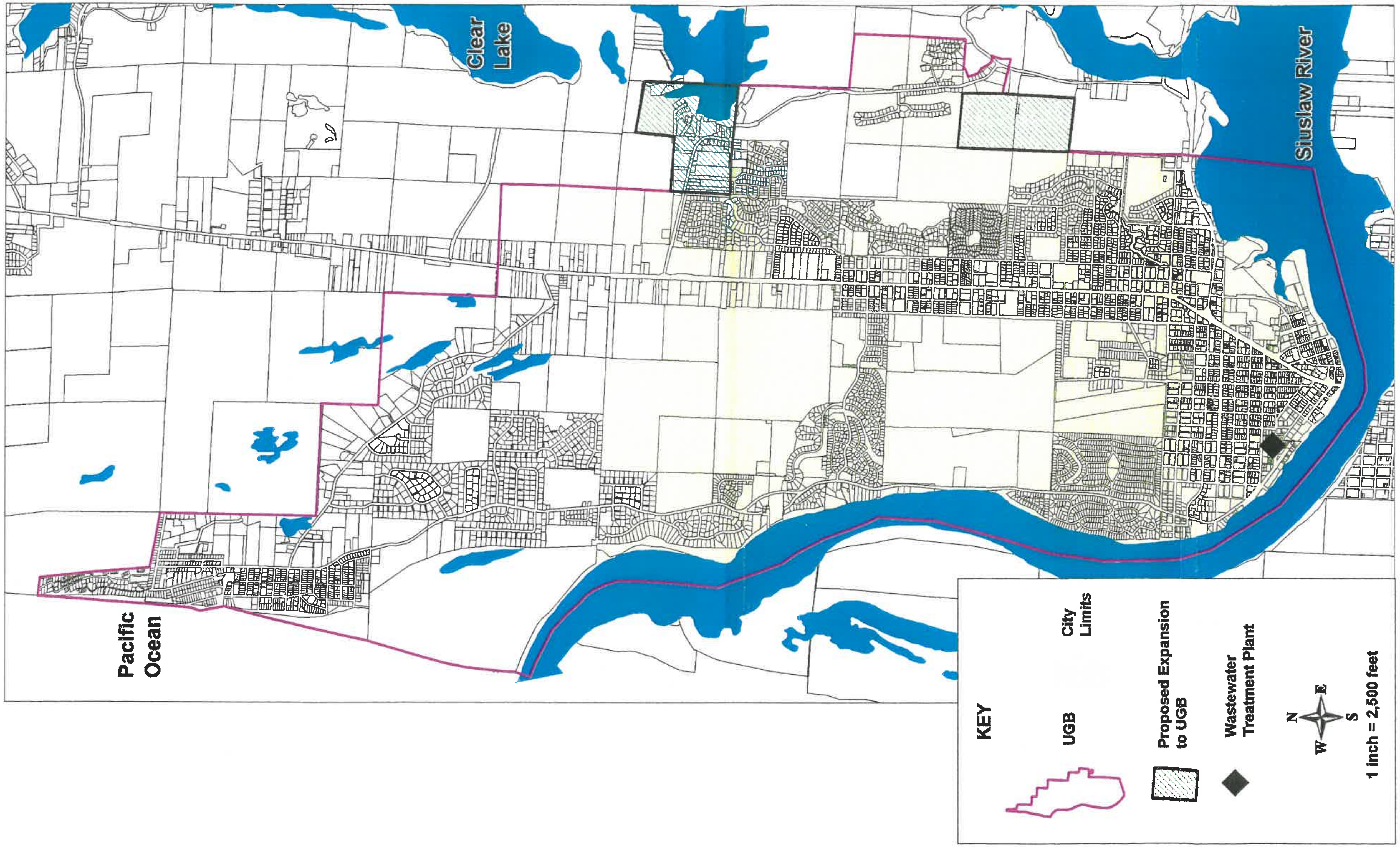

Figure 2-1. Florence Wastewater Study Area

Deflation plains are defined as the interdune areas that have been eroded by wind down to the summer groundwater table. T1hese areas have high groundwater in the summer, and are usually submerged in the winter. The soil type is Yaquina loamy fine sand. The soil tends to be corrosive to steel and concrete. Although the soil is rather permeable, it is poorly drained because of the high water table. Deflation plains that have been developed generally have been filled and drained. Sewers in deflation plains are subject to high groundwater and corrosive soils.

Estuarine deposits underlie the dune deposits in much of the Florence area. They are old Siuslaw River deposits from the Pleistocene epoch. They are usually found 35 feet or less below the dune deposits. Location and thickness can be estimated from well drilling records. These deposits may contain soft compressible clay, organic materials, and peat. They have poor engineering properties and would cause settling of heavy structures if foundations were not designed to account for the presence of this material.

A soil boring performed for a geotechnical study at the existing treatment plant site in June 1993 confirmed the presence of unconsolidated sand interspersed with organic materials, as described above for stabilized dunes.[3] The boring indicated that the depth to groundwater was about 5 feet. The groundwater is expected to be substantially higher in the winter. The report recommends that placement of structural fill and compaction can provide resistance against ground movement from moderate earthquakes. Pile-supported foundations would provide more protection. However, a major earthquake could cause widespread liquefaction, which would result in lateral ground spreading. Piles and compaction would not be effective in protecting against lateral ground spreading; protection from major earthquake damage would probably be prohibitively expensive.

CLIMATE

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of sewerage facilities. Rainfall is especially significant because it can directly or indirectly cause large flow increases in sewage collection systems. It also affects the amount of effluent that may be used for irrigation in a specified period. For example, stormwater runoff may directly enter the sewers at manholes or through illicitly connected roof drains. Accumulated rainfall may raise groundwater levels in many areas, particularly in deflation plains, as mentioned above in the discussion on geology.

Other climatic factors can also affect wastewater processes. Biological treatment processes depend on air and water temperature. Temperature, cloud cover, and the rate of evaporation are important factors to be considered in design of sludge drying beds, composting facilities, and sludge lagoons.

General Climatic Conditions

Florence generally has a mild marine climate. The maximum summertime temperature seldom exceeds 95 degrees F. The wintertime temperature seldom drops below 25 degrees F. Summer conditions are often foggy to sunny and cool; winter conditions include frequent heavy rain with occasional strong wind.

Precipitation

The average annual precipitation recorded at Honeyman State Park, about 3 miles south of Florence, is 72.09 inches. Essentially all of the precipitation is in the form of rain; snow rarely exceeds minor flurries. About 70 percent of the rainfall occurs in November through March. For a more detailed discussion of precipitation in Florence, refer to Chapter 4, *Wastewater Characteristics.*

Temperature

The mean and extreme temperatures recorded at Honeyman State Park are summarized in Table 2-1. The mean daily extremes are mild, as expected in a marine environment. The absolute extremes are also fairly mild, although wintertime temperatures occasionally fall well below freezing. Freezing temperatures have been experienced in October through April. Although the subfreezing temperatures may persist long enough to freeze water in aboveground facilities, they do not last long enough to be of concern for buried facilities. The highest summertime temperature recorded during the period of record was 99 degrees F.

Table 2-1. Florence Area Temperature Summary

						Mean number of days[b]			
	Means[a]			Extremes[a]		Maximum		Minimum	
Month	Daily Max	Daily Min	Monthly	Max	Min	90 or above	32 or below	32 or below	0 or below
Jan	50.3	37.4	43.8	65	14	0	0	8	0
Feb	52.9	38.6	45.8	71	13	0	0	4	0
Mar	55.5	39.5	47.5	78	23	0	0	5	0
Apr	58.6	40.5	49.6	83	29	0	0	2	0
May	62.7	44.0	53.4	85	33	0	0	0	0
Jun	66.0	47.8	56.9	92	36	0	0	0	0
Jul	68.8	50.2	59.5	95	40	0	0	0	0
Aug	69.1	51.0	60.0	91	39	0	0	0	0
Sep	69.3	49.1	59.2	99	32	0	0	0	0
Oct	63.1	45.5	54.3	88	26	0	0	0	0
Nov	54.1	41.6	47.8	69	20	0	0	3	0
Dec	49.9	37.5	43.7	63	9	0	0	5	0
Year	60.1	43.6	51.8	99	9	0	0	27	0

Notes: [a] Temperature mean and extreme data from Honeyman State Park, 1971 through 1990. From Oregon Climate Service.

[b] "Number of days exceeded" data from NOAA Climatological Summary for Reedsport, Oregon, 1951 through 1980.

Other Climatic Factors

Wind speed and direction are not measured and recorded for the Florence area. The nearest coastal location with wind data is North Bend. At that location, the prevailing wind in the wintertime is southeasterly at 7 knots. Discussions with plant staff indicate that the winds are similar in Florence. Several houses on the north side of Rhododendron Drive are downwind of the plant. The summertime prevailing wind is north-northwesterly. This wind would blow odors out over the Siuslaw River.

Evaporation data for the area are unavailable. Evapotranspiration data are available from an agricultural station near Bandon. For design of lagoons and effluent irrigation facilities, site-specific and crop-specific data are needed. If sludge drying beds or sludge lagoons are proposed, pan evaporation data should be collected.

WATER QUALITY ASSESSMENT

The upgrade of the Florence wastewater treatment plant requires an analysis of the impact of future plant discharges on water quality in the Siuslaw River. An understanding of the existing water quality in the river provides the basis for determining allowable pollutant loads while preserving the water quality in the river.

Water Resources

The most significant water resource with respect to wastewater planning is the Siuslaw River and its estuary. The treatment plant discharges into the tidal zone at River Mile 4.1. The river and estuary are heavily used for recreation. Fishing, boating, and other water-based activities provide valued recreation for local residents and seasonal visitors. It is reported that some crab and fish are harvested near the treatment plant, and that some clam beds are located within a few hundred feet of the plant. The most significant clam beds are reported to be more than one-half mile upstream.

Several freshwater lakes are found within the Florence area. Many are used for recreation. Clear Lake, one of the largest, is used as a drinking water source for the Heceta Water District, north of the city. The lake is under consideration as a potable water source for the city as well. The city currently obtains its drinking water from wells. Because the soil is highly permeable in this area, these lakes could be subject to contamination if septic tank drain fields are improperly sited or designed.

Siuslaw River Drainage

The Siuslaw River is in the mid-coast basin. The headwaters are near Lorane, Oregon. From its origin it flows 118 miles to the Pacific Ocean encompassing a watershed of 773 square miles. The river reaches sea level near Mapleton, about 20 miles above its mouth. It then flows across old marine terraces, past Florence, and to the Pacific Ocean. The flat aspect of this part of the drainage has caused flooding problems in the past. Mapleton is only about 60 feet above sea level and tidal influences extend a short distance upstream of the town. The mean tide range is 5.2 feet with an extreme of 11.0 feet.

The river is joined by numerous tributaries throughout its length. The largest of these are Lake Creek at Swisshome and the North Fork Siuslaw at Florence. In the Florence area, the river is an estuary, heavily influenced by saltwater and tides.

Siuslaw River Flows

The USGS maintained a monitoring station on the Siuslaw at River Mile (RM) 23.7 near Mapleton until 1994. Records from the station confirm, that like most rivers in Oregon unregulated by dams or diversions, over 70 percent of the flow in the Siuslaw occurs during the winter months from December through March (Table 2-2).

Table 2-2. Mean Daily Discharge of Siuslaw River at Mapleton (1968-1987)

	Discharge, cubic feet per second			
Month	**Minimum**	**Maximum**	**Mean**	**10th Percentile**
January	300	10,100	5,000	1,070
February	876	9,080	4,710	1,230
March	1,290	6,820	3,530	1,320
April	686	4,450	2,120	919
May	541	2,100	1,040	508
June	320	1,240	567	291
July	127	628	269	152
August	77	321	157	83
September	86	356	194	82
October	94	1,220	449	93
November	281	7,820	2,520	310
December	261	9,790	5,260	780
Annual average	576	3,720	2,140	136

The Mapleton station measures flow from a drainage area of 588 square miles, although a number of tributaries enter the Siuslaw below this point. The largest of these is the North Fork Siuslaw, which joins the main stem just east of Florence. Until 1985, the USGS had a monitoring station at RM 13 near Minerva. Flow statistics from that station represent an additional 41 square miles of drainage area (Table 2-3).

Table 2-3. Mean Daily Discharge of North Fork Siuslaw at Minerva (1967-1985)

Month	Discharge, cubic feet per second			
	Minimum	Maximum	Mean	10th Percentile
January	72	1080	636	136
February	171	901	578	147
March	150	809	450	153
April	134	560	274	124
May	76	384	145	71
June	41	295	105	45
July	26	135	48	24
August	15	78	28	16
September	16	99	39	16
October	15	279	99	16
November	66	1080	417	53
December	56	1300	758	133
Annual average	119	445	297	22

None of the other small tributaries towards the lower end of the Siuslaw have been gauged. Therefore, adding the flows from the two stations provides an incomplete summary of flows at Florence (Table 2-4), as they encompass only 629 of the 773 square miles of the total watershed. The normal river flow at the mouth, for instance, is estimated as 3,150 cubic feet per second (cfs), compared to the 2,437 cfs calculated in Table 2-4.

Table 2-4. Estimated Mean Daily Discharge of Siuslaw at Florence

Month	Discharge, cubic feet per second			
	Minimum	Maximum	Mean	10th Percentile
January	372	11,180	5,636	1,206
February	1,047	9,981	5,288	1,377
March	1,440	7,629	3,980	1,473
April	820	5,010	2,394	1,043
May	617	2,484	1,185	579
June	361	1,535	672	336
July	153	763	317	176
August	92	399	185	99
September	102	455	233	98
October	109	1,499	548	109
November	347	8,900	2,937	363
December	317	11,090	6,018	913
Annual average	695	4,165	2,437	158

Another statistic used for regulatory compliance is the 7Q10 flow. This is defined as the lowest flow during a consecutive 7-day period over 10 years. The 7Q10 value for the Mapleton station is 62 cfs and for the Minerva station it is 13 cfs, for a combined total of 75 cfs. These 7Q10 values, calculated by the USGS, are lower than the minimum flows reported during the same 18-year period, which appears inconsistent. However, using the lower values in the water quality evaluation is conservative.

Existing Water Quality

A large amount of sampling by various regulatory agencies has occurred along the Siuslaw River within the last 30 years. The most comprehensive was performed by the Oregon Department of Environmental Quality (DEQ) from 1968 through 1983 at 15 sites along the river and its tributaries. The monitoring station that has operated the longest is No. 402062 at Mapleton. It has provided water quality information dating from 1960 until the present. The USGS station, also at Mapleton, has monitored the most parameters, including metals, from 1977 until 1992. Several other sites were monitored briefly in 1971. Summary statistics from selected sites are shown in Table 2-5.

Dissolved Oxygen. The dissolved oxygen (DO) standard for the Mid Coast Basin is contained in the Oregon Administrative Rules 340-41-245: *"For estuarine water, the dissolved oxygen concentrations shall not be less than 6.5 milligrams per liter (mg/L) (for coastal waterbodies)."*

The mean DO values at all eight sites listed are above the 6.5 mg/L limit. The minimum values, however, did fall below this level in 12 of 281 samples (< 5%). Most of these low values occurred in June of 1968. Only isolated instances of DO less than 6.5 mg/L have been reported since then.

Temperature. The Siuslaw has been listed on the 303-d list as water quality limited due to excessive summer temperatures. The temperature standard for the Mid Coast Basin is contained in the Oregon Administrative Rules 340-41-245: *"Marine and estuarine waters: No significant increase above natural background temperatures shall be allowed, and water temperatures shall not be altered to a degree which creates or can reasonably be expected to create an adverse effect on fish or other aquatic life."*

The mean temperatures at the eight Siuslaw sites range from 53 to 57 degrees F, well below the 64 degrees F that is considered to adversely affect salmonid fish rearing. However, the 90th percentile values at most of the sites are above this level. The 90th percentile values are often used in mixing zone analysis of outfalls.

pH. The pH standard for the Mid Coast Basin is contained in the Oregon Administrative Rules 340-41-245. The pH values shall not fall outside the following range: *"Estuarine and fresh waters: 6.5 - 8.5."*

The maximum pH values at all eight sites are below the permissible upper limit. Three of the sites have minimum values that fall below the lower end of the range.

Table 2-5. Water Quality Summary Statistics for Selected Sites

Site	Temperature, degrees F				pH				Dissolved Oxygen, mg/L				Total Ammonia			
	min	max	mean	90^{th} per-centile	min	max	mean	90th per-centile	min	max	mean	10th per-centile	min	max	mean	90th per-centile
A	37	75	54	68	6.3	7.8	7.2	7.6	8.1	13.4	11.0	8.9	.01	.09	.022	.048
B	45	62	53	59	7.1	8.4	8.0	8.3	5.4	11.3	8.9	7.4				
C	44	65	55	62	6.5	8.4	7.8	8.3	5.5	13.4	9.4	7.5				
D	43	64	55	63	6.6	8.4	7.7	8.3	5.6	11.4	8.9	7.2				
E	43	66	57	66	6.7	8.3	7.6	8.2	4.8	12.7	9.4	7.5				
F	42	68	57	66	6.6	8.3	7.6	8.1	5.6	11.7	8.7	6.5				
G	42	70	56	65	6.4	8.2	7.4	8.0	5.6	11.6	9.0	6.7				
H	41	77	57	68	6.4	7.9	7.0	7.3	4.3	13.2	10.1	8.0	.01	.87	.105	.235

Site A - Station 14307620, USGS station near Mapleton, OR
Site B - Station 412065, DEQ at Marker 16
Site C - Station 412066, DEQ at Marker 32
Site D - Station 412067, DEQ at Marker 47
Site E - Station 412068, DEQ 0.5 miles upstream of Hwy 101
Site F - Station 412069, DEQ at Marker 52
Site G - Station 412070, DEQ North Fork Siuslaw at Hwy 126
Site H - Station 402062, DEQ, Hwy 126 at Mapleton

Bacteria. The bacterial standard for the Mid Coast Basin is contained in the Oregon Administrative Rules 340-41-245. After discussions with the DEQ, it was determined that clamming and other activities in the Siuslaw qualified it as a shellfish growing water. Accordingly, the applicable regulation is *Marine Waters and Estuarine Shellfish Growing Waters: "A fecal coliform median concentration of 14 organisms per 100 milliliters, with not more than ten percent of the samples exceeding 43 organisms per 100 ml."*

The eight sites included in Table 2-5 had more than 150 samples taken for total fecal coliforms. All of the sites routinely exceeded both the median and the 10 percent requirements except for sites 412065 and 412066. Note that site 412065 is the only site in the lower part of the estuary below Florence.

Ammonia. Toxicity is caused by the un-ionized form of ammonia. The amount of un-ionized ammonia is dependent on many factors, including temperature, pH, and salinity. The methodology for determining freshwater toxicity is well established; however, a similar method does not exist for saltwater.

Ammonia concentrations were measured only at the Mapleton sites, Stations A and H. Toxicity calculations for freshwater show that the maximum un-ionized ammonia present is less than 4 percent of that required for acute toxicity and less than half that required for chronic toxicity. Whether organisms would actually be exposed for the 4-day duration assumed for chronic toxicity is uncertain, as well as the effects of salinity in the Siuslaw's lower reaches.

Metals. Metals toxicity is affected mainly by water hardness. Typically, the harder the water, the lower the toxicity. The Siuslaw River has very low hardness, averaging 12 mg/L. However, metals do not appear to be a significant problem at the only site to monitor them, Station 14307620.

The dissolved form of the metal is the toxic form, and therefore was the one used for calculations. Dissolved cadmium values were frequently reported as exceeding the 0.4 mg/L acute toxicity limit, but many of these values appeared to be sample detection limits and not very reliable. Dissolved zinc appears to have exceeded the acute toxicity guidelines several times during the late 1970s and early 1980s, but no recent excursions have been reported. Dissolved copper is the only metal that continues to be measured frequently at concentrations exceeding those believed to cause acute toxicity.

Sediments

The Army Corps of Engineers maintains a dredged channel from the Siuslaw entrance to RM 16.5 and have tested sediments prior to disposal since the early 1960s. Sediments from the dredged channel are fine to medium sands low in organic content. Therefore, the potential for significant chemical concentrations is low. The Corps does not routinely run chemical analyses on sediments of this nature. The one analysis reported was run in 1991. It showed cadmium, copper, and mercury at less than detection limits and only small amounts of arsenic, chromium, lead, nickel, and zinc. No organochlorine pesticides, PCBs, polynuclear aromatic hydrocarbons, or phenols were detected. Sediments in the Siuslaw appear relatively free of contaminants.

SOCIOECONOMIC ENVIRONMENT

Wastewater treatment system demands and design capacities are determined by population, land use patterns, and economic growth within the UGB. This section presents population projections based on historical data for the city. Land use information was obtained from Lane Council of Governments (LCOG) land use and zoning maps.

POPULATION

Existing and projected population of the service area are key elements in projecting sewage flows. Population projections in this report are based on projections developed by LCOG and the city planning department in the process of updating the comprehensive plan. The comprehensive plan update is currently under development.

The Florence population projections were developed by LCOG using several approaches, resulting in a range of projections. The low end projection was based on the lowest historical growth rate experienced out of the last 5, 15, and 25 years. The lowest annual average growth rate (AAGR) occurred between 1980 and 1995. This rate was 2.3 percent and is assumed as the low end projection.

The high-end projection is based on the most recent growth rate of Florence. The rate from 1990 to 1995 was 3.7 percent. Although the rate of 3.7 percent is not as high as the rate of 7 percent experienced during the 1970s, it is assumed as the maximum sustainable rate based on the decline of resource-based industries.

The two rates presented above (2.3 percent and 3.7 percent) are the expected minimum and maximum bounds on the AAGR for Florence over the next 20 years as determined by LCOG. The updated comprehensive plan will assume a growth rate between these bounds. City planners expect that the assumed rate will be toward the lower end of the range. For the wastewater facilities plan, a growth rate of 3.5 percent is assumed. Selecting a rather high rate within the planning range is based on the following observations:

- Demographics. Although population growth in Florence may be limited by the lack of resource-based industries, other factors point toward a continued high growth rate. Growth in the nearby Eugene-Springfield area is projected to be strong. New industries (computer and electronics-based) have moved into the area, reducing the dependence on timber. Growth in the Eugene area will probably result in a substantial increase in tourism in the Florence area. The attendant increases in services in Florence will make the area even more attractive to new residents. Demographics indicate that a large proportion of retired people are moving to Florence and other parts of the Oregon coast. The average age of the population in Florence has been increasing since 1960. This trend will probably continue, particularly as the age of the United States population as a whole increases. Although the increase in retirement-age population results in a lower birthrate, it also provides growth that is relatively independent of the availability of jobs. Influx of more retired people will actually create more jobs.

- Economies of scale. The marginal cost of constructing a plant with a slightly higher capacity is relatively small. Constructing a unit process with a greater capacity generally does not cost as much per unit capacity. Also, there are many project-wide fixed costs, including design, mobilization, and construction management, which are not heavily affected by small changes in plant size.
- Uncertainties in projections. Because there are many uncertainties in wastewater planning, it is generally advantageous to take a conservative approach in sizing facilities. If the facilities are oversized, they will be adequate for a longer period than the 20-year planning horizon. If they are undersized, capacity problems could develop in the near future.

To determine the design population, the assumed AAGR of 3.5 percent is applied to the current UGB population over the 20-year design period. The entire UGB population is used because it expected that the service area will expand to include the entire UGB. According to the Center for Population And Census at Portland State University, the 1995 populations for the city and the UGB were 6,185 and 7,590, respectively. The projected populations for each year of the design period are calculated based on these populations and the assumed AAGR of 3.5 percent. These populations are summarized in Table 2-6.

Table 2-6. Florence City and UGB Population Projections

	Population	
Year	City	UGB
1995	6,185	7,590
1996	6,401	7,856
2000	7,346	9,015
2005	8,725	10,706
2010	10,362	12,716
2015	12,307	15,102
2020	14,617	17,937

Note: Shaded area represents actual data from the Center for Population And Census. Unshaded area represents extrapolations based on 3.5 percent AAGR.

The current service population is assumed as 6,401, the estimated city population in the year 1996. The design service population is calculated as 17,937, the estimated entire UGB population for the design year 2020. The service population increase over the design period is 280 percent, or a factor of 2.8.

LAND USE

Land use within the Florence UGB is largely determined through the city's comprehensive plan and zoning. Historical development patterns in many cases have simply been reflected by these efforts. For this study, the city's 1988 Comprehensive Plan land use plan categories can be used to reflect general land use distribution throughout the UGB. These categories include residential, commercial, highway area, waterfront, industrial, marine, open space, and public. Figure 2-2 presents the city's current land use plan. The corresponding plan categories reflect the recommended use of those lands, even though the current use may be quite different. For example, residential use of commercially planned land occurs in many instances. For facility planning purposes, the planned use governs the assumptions made in this report.

The city is in the process of updating this land use plan (Periodic Review) and may change some of the land use recommendations presented in Figure 2-2. Details are presented in the city's urban growth boundary amendment reports. The significant anticipated changes are discussed below. One area of potential land use change includes about 60 acres near the intersection of Munsel Lake Road and Highway 101 in north Florence. This area is planned and zoned mostly for commercial use, but is being considered for large-scale, regional commercial uses with a planned commercial activity node. This would change the use expectations from small-scale light and heavy commercial uses to large retail and supporting commercial uses such as a hotel and full-service restaurants. Another land use change is anticipated along 9th Street west of Kingwood Street. This area is experiencing professional office and institutional development rather than the planned residential development. The city expects to continue this transition to professional office space, while still encouraging higher density residential uses on the periphery.

Two 80-acre areas are being considered as part of an expanded UGB, as shown on Figure 2-1. One area lies on the southeastern edge of Florence. Currently, the Ocean Dunes Golf Course lies partially within the city and UGB, and partially outside. The Ocean Dunes residential planned unit development lies within city limits, and the golf course developer proposes to expand the UGB to bring the entire Ocean Dunes Golf Course into the UGB, and ultimately city limits. This will increase the residential yield opportunities through the availability of public sewer to this area.

The second area lies on the northeastern edge of Florence, near Munsel Lake. Suburban densities have already been established through property dividing in this area. Including this area in the UGB, and perhaps ultimately within the city, would permit the extension of sanitary sewer service along Munsel Lake Road.

A wetlands inventory completed in November 1996 by Pacific Habitat Services indicates that about 570 acres within the wetlands study area are covered by wetlands. The area of wetlands within the UGB would be less than this. About one third of the wetland acreage falls within areas that would otherwise be available for development. This represents a 3.5 percent reduction in available land area. For planning purposes, it is assumed that this reduction in available land will not affect growth. A slight increase in population density could compensate for the loss in land area.

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1. U.S.D.A. Soil Conservation Service. *Soil Survey of Lane County Area, Oregon.* 1987.
2. Schlicker, Herbert G., et al. Environmental Geology of Coastal Lane County, Oregon. 1974.
3. Applied Geotechnology, Inc. Geotechnical Investigation and Report Proposed Sludge Thickening Facility Wastewater Treatment Plant Florence Beach, Oregon. August 1993.

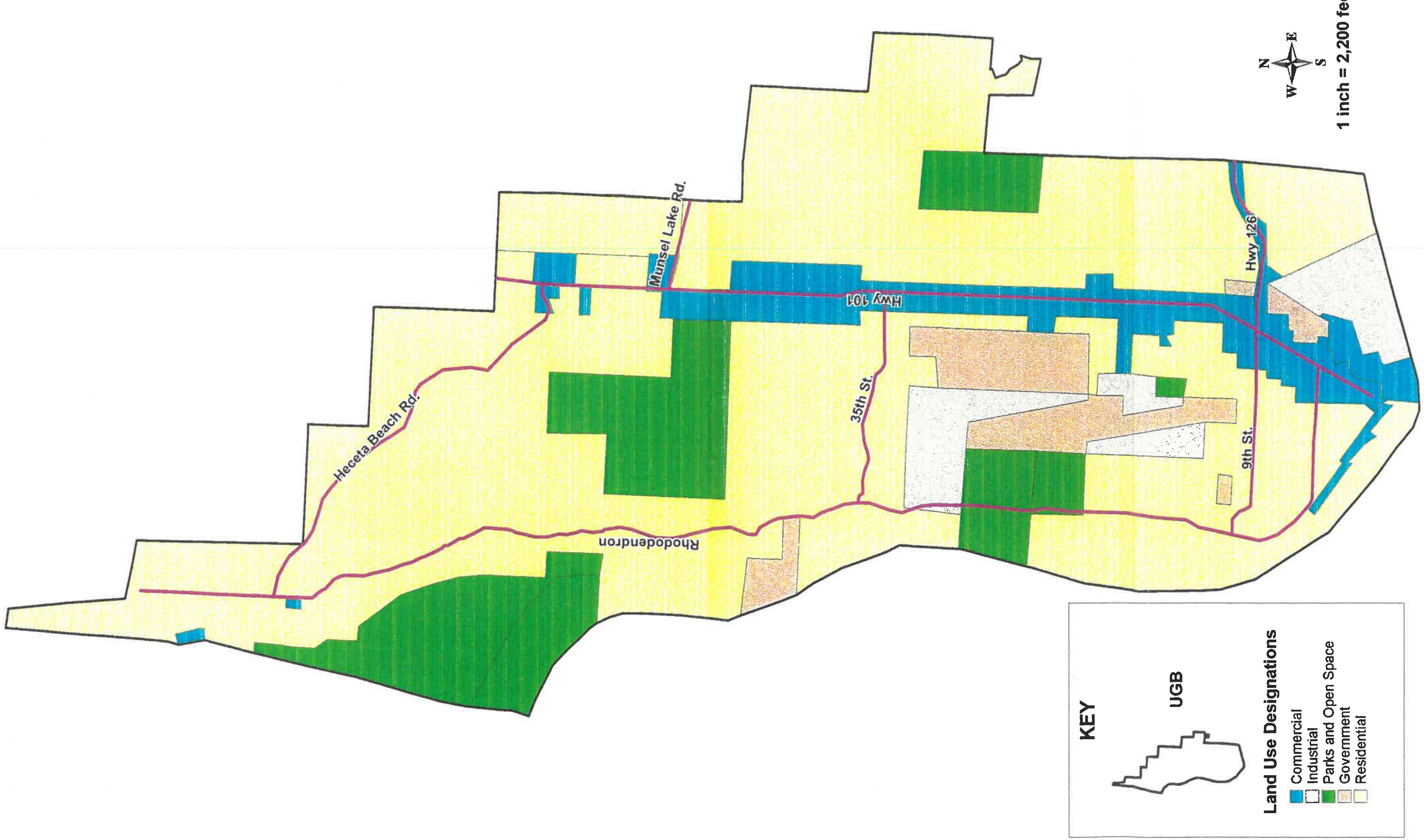

Figure 2-2. Land Use Designations in Florence Area

CHAPTER 3

EXISTING WASTEWATER SYSTEM

The Florence wastewater system includes the collection system and the wastewater treatment plant. The collection system is divided into basins, from which the wastewater is pumped to the treatment plant. The plant is located on the north bank of the Siuslaw River. In this chapter, the plant and collection system are described and their condition and performance are assessed.

WASTEWATER TREATMENT PLANT

The existing treatment plant utilizes a complete mix activated sludge process for secondary treatment. Preliminary treatment includes fine mesh screens and vortex grit removal tanks. There is no primary sedimentation. Aeration takes place in a single basin with mechanical surface aerators. Secondary sedimentation is accomplished in two circular secondary clarifiers. The secondary effluent is disinfected with chlorine. The outfall discharges to the north shore of the Siuslaw River at River Mile 4.1.

One anaerobic digester provides sludge stabilization. Waste activated sludge is thickened on a gravity belt thickener before it is pumped to the digester. Digested sludge is hauled in liquid form for land application.

The original plant was built on the current site in the early 1960s. It consisted of a primary clarifier and anaerobic digester, with sludge drying beds. The digester and associated control building continue to be in use. In 1971, the plant was upgraded to provide secondary treatment by adding the aeration basin and converting the primary clarifier to a secondary clarifier. The chlorine contact tank was also constructed at that time. In 1978 the second clarifier was constructed. In 1982 the headworks was constructed. A third screen and grit tank was added in 1990. In 1994 a sludge thickening building housing a 1-meter belt thickener was added.

PLANT DESIGN

The treatment plant layout is shown in Figure 3-1, and the design data are presented in Table 3-1. The plant flow schematic is shown in Figure 3-2. Raw wastewater is conveyed to the plant site through two force mains. An 8-inch force main, fed by several pump stations, conveys wastewater into the plant from Rhododendron Drive. About one-fourth of the current flow to the plant is conveyed by this pipeline. The remaining flow is pumped from the Ivy Street pump station through an 8-inch force main, entering the plant at the southeast corner. The flows from both pipelines are combined near the headworks.

The incoming wastewater flows over a set of three static sidehill screens, into vortex grit tanks. Screenings slide down off the screens into drop boxes. Underflow from the grit tanks flows by gravity into a settling basin. Overflow from the grit settling basin flows directly into the aeration basin. Grit is removed from the settling basin by hand.

Table 3-1. Design Data

Item	Value	Item	Value
Design flow		Secondary clarifiers	
Average dry weather,	0.75	Type	Circular
Peak wet weather flow,	1.5	Number	2
Design loading		Diameter, feet	1 @ 35, 1 @
BOD, average, ppd	1,000	Depth, feet	9 10
Suspended solids, ppd	1,100	Maximum RAS flow,	2.0
Pretreatment		Disinfection	
Screens		Chlorine contact basin	
Type	Sidehill	Number	1
Number	3	Volume, cubic feet	4,300
Width, inch	48	Chlorinator	
Opening width, inch	0.06	Number	1
Total capacity, mgd	2.6	Control	Manual
Grit removal		Capacity, ppd	150
Type	Vortex	Outfall	
Number	3	Diameter, inches	10
Total capacity, mgd	1.75	Sludge thickener	
Aeration basin		Type	Gravity belt
Volume, cubic feet	80,000	Number	1
Aeration		Belt width, meters	1
Type	Surface	Capacity, pounds/hour	800
Number	7	Thickened sludge pump	
Total horsepower	105	Type	Prog. cavity
Mixed liquor pumping		Capacity, gallons per	28
Pump type	Centrifugal	Anaerobic digester	
Number	3	Number	1
Capacity, each, mgd	1.0	Diameter, feet	30
		Side water depth, feet	14
		Volume, cubic feet	12,070
		Organic loading, lb	0.09

The degritted wastewater flows by gravity through a Parshall flume to the aeration basin. The basin is a single, shallow asphalt-lined pond with seven floating mechanical mixers. The basin has no baffles or dividing walls; it approximates a complete mix reactor. Raw wastewater is fed to the southwest corner of the basin; mixed liquor is withdrawn from the northeast corner. Return sludge flows by gravity to the center of the basin.

The mixed liquor is pumped from the aeration basin to the secondary clarifiers at a constant rate by three vacuum-primed centrifugal pumps. The pumps draw from a common suction line and discharge into a common header into an 8-inch pipeline. The mixed liquor distribution to the two clarifiers is controlled by gate valves.

The two secondary clarifiers have center feed and peripheral effluent launders. The sludge return rate is controlled by valves on the sludge piping. The secondary effluent flows by gravity through a 10-inch pipeline to the chlorine contact basin.

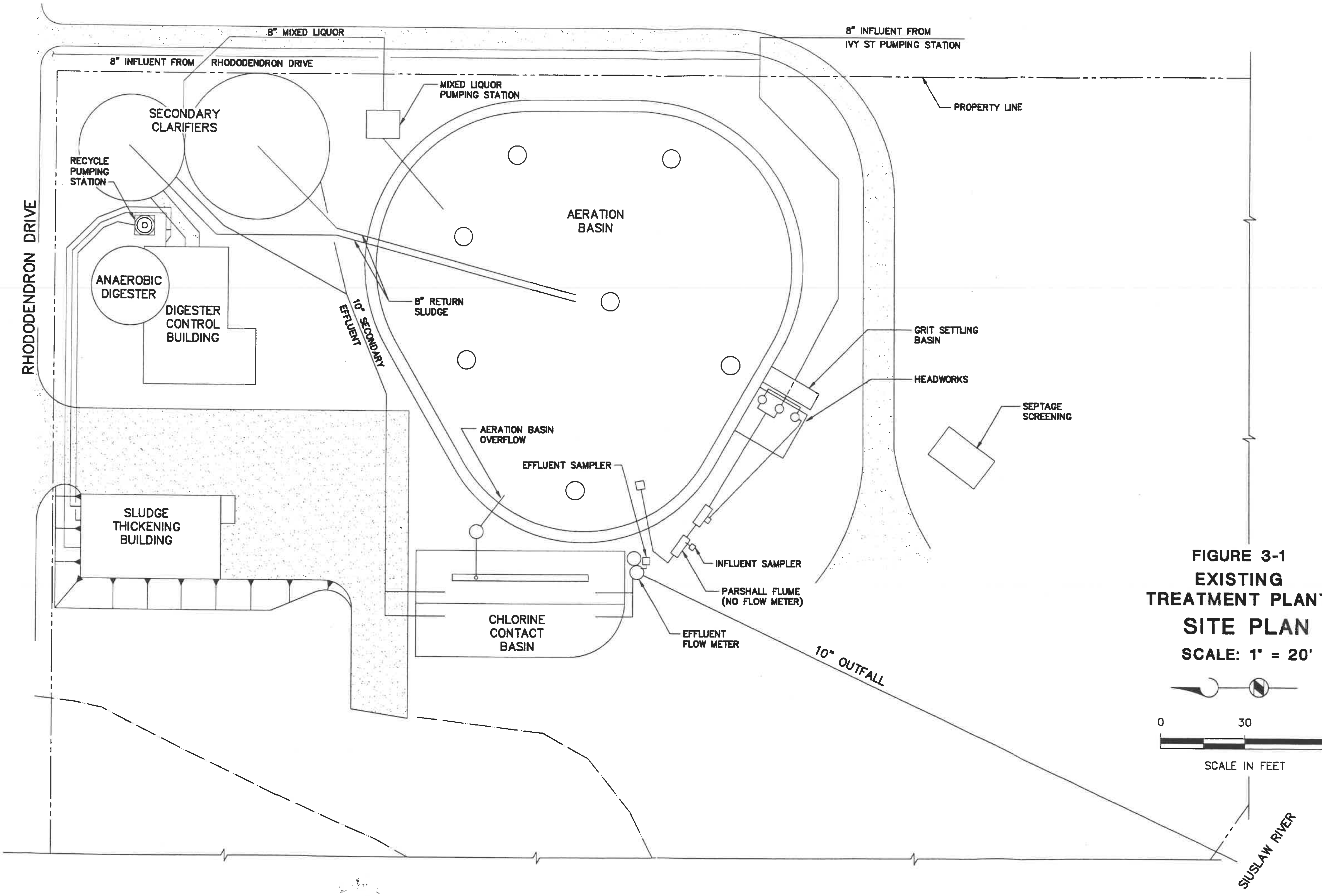

FIGURE 3-1
EXISTING TREATMENT PLANT
SITE PLAN
SCALE: 1" = 20'

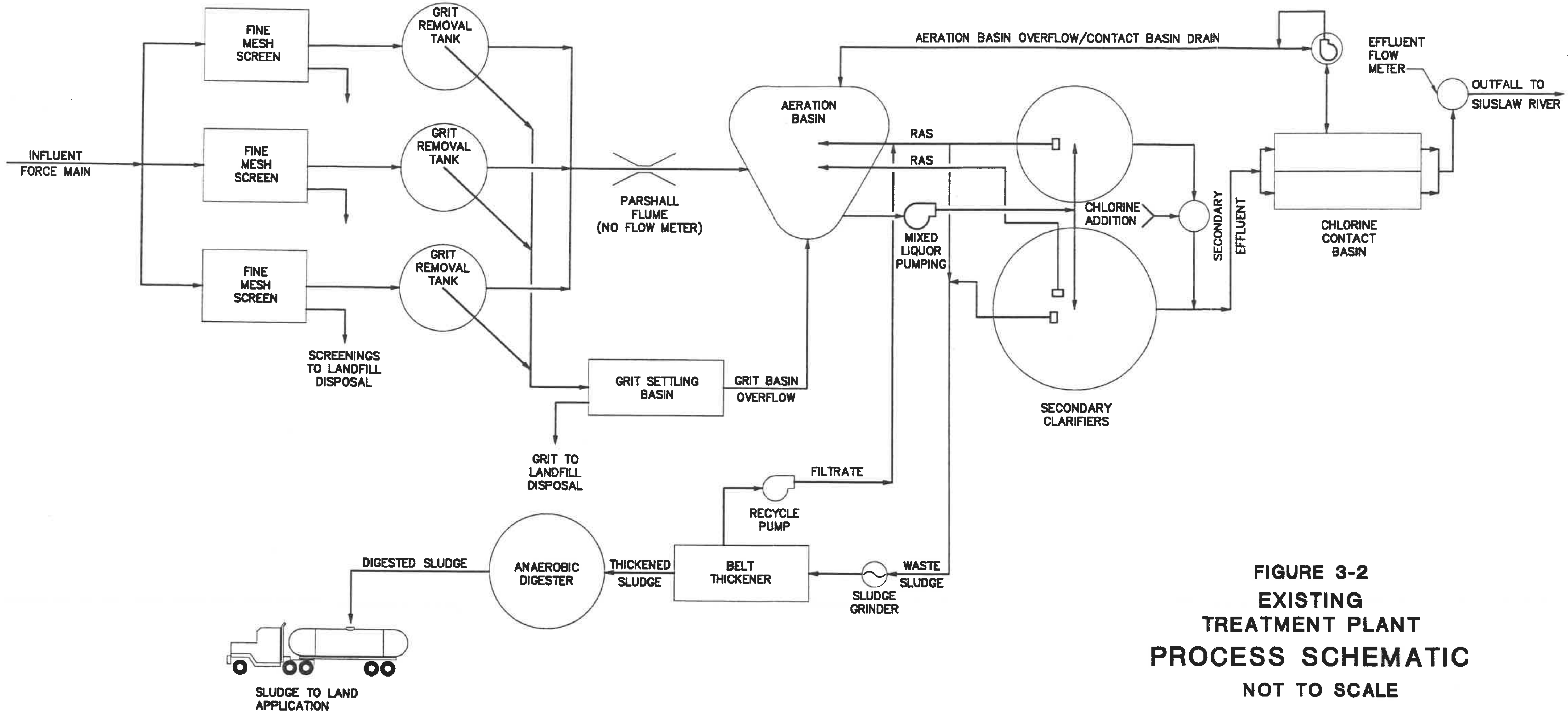

FIGURE 3-2

**EXISTING
TREATMENT PLANT**

PROCESS SCHEMATIC

NOT TO SCALE

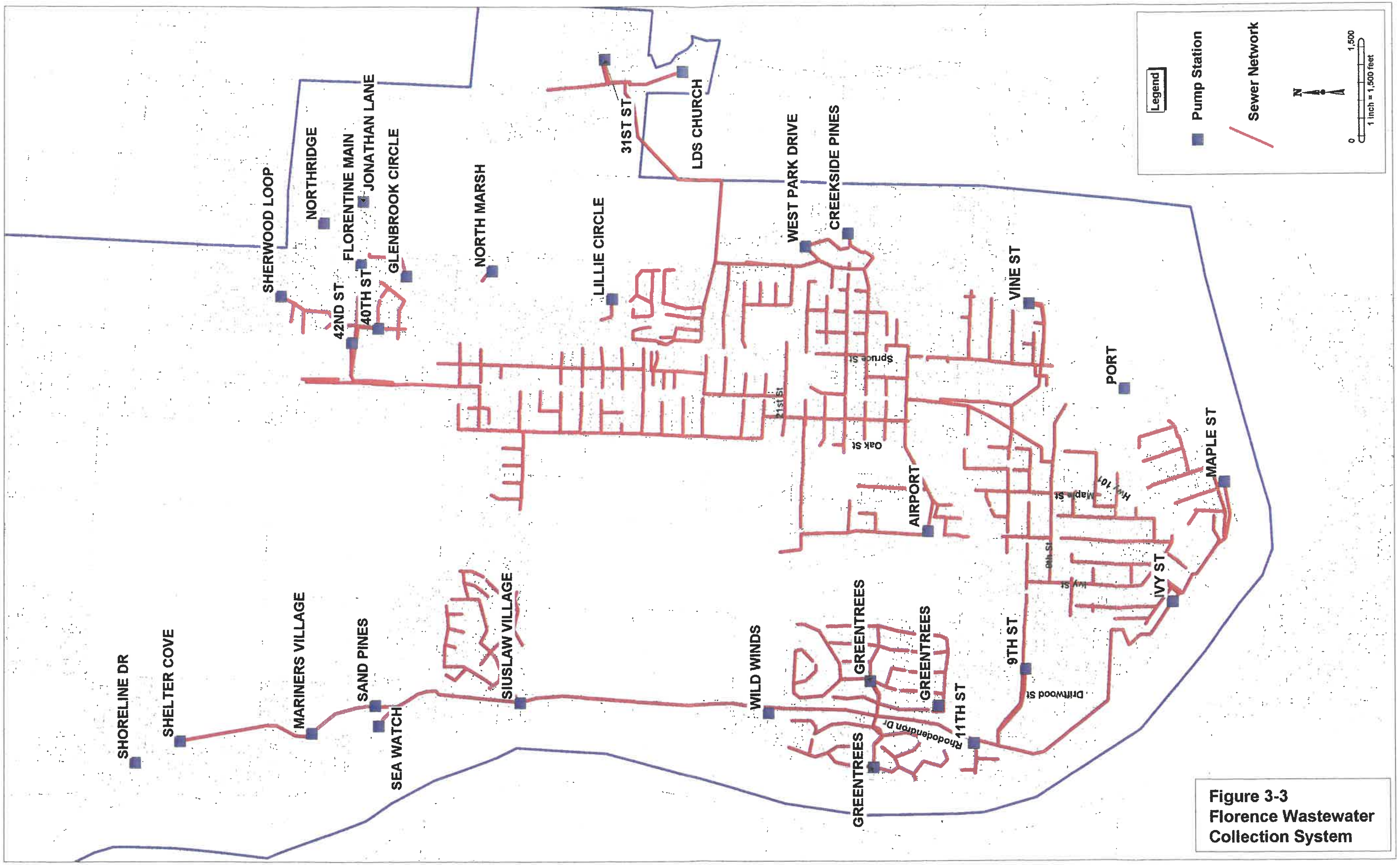

Figure 3-3
Florence Wastewater Collection System

Table 3-4. Collection System Pump Station Summary

Name	Installation date	Pump characteristics[b] Rated capacity/head, gpm, feet	Motor hp	Measured capacity/head, gpm, feet[c]	Wet well condition	Outlet structure condition	High level alarms	Bypasses
9th Street	1986	240/48	7.5	90/30	good	pressure main[d]	yes	no
11th Street	1979	100/18	2	105/9	good	pressure main[d]	yes	no
31st Street	1985	260/140	25	245/111	f	good	yes	yes
40th Street	1978	100/38	5	30/16	good	good	yes	no
42nd Street	1991	200/30	5	245/23	good	good	yes	no
Airport	1980	215/19	3	140/10	good	good	yes	no
Creekside Pines	1994	100/30	3	140/7	good	good	yes	yes
Florentine	1990	150/30	5	210/14	good	good	yes	yes
Glenbrook	1993	150/20	5	230/19	good	good	yes	no
Ivy	1961	a	15	1 pump: 540 2 pumps: 720 3 pumps: 990 4 pumps: a	good	e	yes	yes
Jonathan	1991	72/40	3	94/a	good	good	yes	no
LDS Church	1986	90/22	2	123/16	good	good	yes	no
Lillie Circle	1994	160/31	7.5	123/24	good	good	yes	no
Maple	1961	a	a	45,71g	good	good	yes	yes
Mariners Village	1991	130/25	3	70/16	good	pressure main[d]	yes	no
North Marsh	1995	40/25	3	73/a	good	good	yes	no
Northridge	1991	130/35	5	205/5	good	good	yes	no
Port	1992	95/40	5	141/22	good	good	yes	no
Sand Pines	1992	a	a	230/12	good	pressure main[d]	yes	no
Sea Watch	1990	100/22	3	55/12	good	pressure main[d]	yes	no
Shelter Cove	1995	130/25	3	45/11	good	pressure main[d]	yes	no
Sherwood Loop	1992	100/40	5	350/9	good	good	yes	no
Shoreline Dr	1995	150/21	3	175/7	good	good	yes	no
Siuslaw Village	1976	350/120	30	360/106	good	pressure main[d]	yes	no
Vine	1976	80/70	7.5	80/70	good	poor	yes	yes
West Park	1972	a	a	200/a	fair	good	yes	no
Wild Winds	1989	40/30	3	265/10	good	pressure main[d]	yes	no

Notes: [a] Information unavailable.
[b] Each pump station has two pumps, one serving as backup, except Ivy Street, which has 4 pumps that can operate simultaneously.
[c] Capacity measured by city staff by timing wet well pumpdown. Head measured with gauge in discharge piping, not adjusted for height above wet well water surface. Actual differential head across pump may be 10 to 15 feet greater.
[d] Pump station discharges into pressure main which cannot be inspected.
[e] Pump station discharges to treatment plant. No evidence of sulfide problems at headworks.
[f] Manhole inaccessible without ladder. Corrosion unlikely due to forced ventilation of wet well.
[g] Maple Street capacities are 45 and 71 gallons per minute (gpm) for pumps 1 and 2, respectively. Head information is unavailable.

The chlorine contact basin is an asphalt-lined basin divided into two parallel chambers. The flow is distributed to the two chambers. Final effluent exits the tank through submerged outlets and is conveyed to the Siuslaw River in a 10-inch outfall.

Waste sludge is drawn from the return sludge piping and pumped through a grinder to a gravity belt thickener. The filtrate drains to a sump and is pumped to the aeration basin. The thickened sludge is pumped to the digester.

The anaerobic digester is a concrete tank with a fixed, concrete cover. Mixing is provided by a propeller draft tube. Sludge is circulated through a spiral heat exchanger for heating. Heating is provided by an oil-fired boiler. Provisions exist for operating the boiler on digester gas, but the use of gas results in unreliable operation of the boiler. Sludge is removed from the digester by opening a valve in a line off the circulation piping.

OPERATIONS AND PERSONNEL FACILITIES

All operational facilities are housed in the sludge control building. The total area of the building is about 1,000 square feet. The sludge pumping room and chlorination equipment and storage rooms occupy about 600 square feet, leaving about 400 square feet for the office, laboratory, storage, restroom, and lockers. The facilities are not adequate for the current plant; some activities, including equipment maintenance, must be performed elsewhere.

There is no maintenance facility at the plant. Some small equipment maintenance can be performed in the tool and storage room, but most maintenance is performed at the city's public works maintenance facility. There is no maintenance or covered parking area for vehicles. The storage room occupies about 150 square feet. The room is currently used to capacity and would not be able to accommodate the additional storage requirements of an expanded treatment plant.

The laboratory occupies about 180 square feet. The shelf and counter space are inadequate and there is no fume hood. The room also serves as an office; however, there is little work or storage space.

There is no break room or meeting room; all meetings are conducted at the public works maintenance facility. A small restroom includes a lavatory, water closet, and shower. There are not separate men's and women's washrooms. Two lockers are provided in a hallway outside the restroom; there is no separate locker room.

The building is of reinforced masonry construction with a heavy timber roof system. The masonry appears sound, but leaks have developed in several places. After heavy rains puddles develop on the floor. The roof also has several leaks. A significant void has developed under the chlorination room as a result of a plumbing leak. However, because the building is pile-supported, the voids should not have a negative effect on the building.

UNIT PROCESS PERFORMANCE AND CONDITION

Loading and operating performance information was obtained from plant operating reports and discussions with plant personnel. Performance parameters for the major processes are summarized in Table 3-2.

Table 3-2. Treatment Process Loading and Performance

Unit Process	Value
Aeration basin	
Estimated average organic load, BOD, pounds per day per 1,000 cubic feet	21
Mean cell residence time, days	14
Mixed liquor suspended solids concentration, mg/L	4,300
Secondary clarification	
Hydraulic loading, gallons per day per square foot	
Average	250
Peak wet weather[a]	1,230
Disinfection	
Average chlorine usage, pounds per day	15
Average chlorine residual, mg/L	0.6
Contact time at peak wet weather flow, minutes[a]	15
Sludge thickening	
Solids loading, pounds per hour per meter	500
Thickened sludge concentration, percent	5
Anaerobic digestion	
Volatile solids loading, pounds per day per cubic foot[b]	0.09
Average detention time, days[c]	29.5

Notes [a] Based on current estimated peak wet weather flow of 3.6 mgd. However, limit to flow through the existing plant is about 1.5 mgd.

[b] Assumes solids are 80 percent volatile.

[c] Based on a feed sludge solids concentration of 5 percent.

Headworks

The headworks consists of three static wedgewire fine mesh screens mounted on top of vortex-type grit removal tanks. The screens, with 0.06-inch openings, remove large quantities of solids and organic material from the wastewater. They probably remove at least 10 percent of the biochemical oxygen demand (BOD) from the incoming wastewater. Although removal of the organic material reduces the load on downstream processes, it causes greater problems at the headworks. The screenings slide by gravity directly into drop boxes below. Eleven drop boxes are required to accommodate the screenings. The screens require steam cleaning each day. Handling the boxes and cleaning the screens require nearly an hour of labor each day. A large amount of organic material remains on the screens continuously during operation. Because the screens are exposed and highly visible, the screenings represent the most significant source of odor at the plant as well as being visually objectionable. The screened wastewater drops directly into the grit removal tanks below the screens.

The grit removal tanks are steel tanks ("Teacups") supported above grade on legs. Two of the three tanks show significant evidence of corrosion. The newest tank, of stainless steel, is in good condition.

The grit system operates on a vortex principle. The wastewater rotates at sufficient velocity within the tank to force the grit toward the center. The system incurs a head loss of more than a foot, requiring the screens to be elevated. Underflow from the bottom center of the tank carries the grit slurry to an adjacent settling basin for further separation. The degritted wastewater flows out the side of the tank into a header. The wastewater flows by gravity through a Parshall flume to the aeration basin. The flume currently serves no measurement or hydraulic control function.

The grit-laden underflow from the bottom of the tanks flows by gravity to a rectangular basin at grade. The grit settles and accumulates in the basin. It is removed by hand on an annual basis. The overflow from the basin flows directly into the aeration basin.

The grit tanks are very effective at removing grit down to 100-micron size, exceeding 95 percent capture. However, much of the finer grit is probably resuspended in the settling basin because it has a rather high overflow rate with no weir to eliminate short circuiting. The amount of grit removed is only about ten cubic yards per year. This is about half the typical amount for a plant this size. Much larger quantities would be expected, given the sandy nature of the soil in the Florence area.

Aeration

The aeration basin is a shallow asphalt-lined pond with sloped sides. An underdrain system enables operators to lower the surrounding water table before the basin is emptied to prevent rupturing the aeration basin. The water surface elevation in the basin varies, causing the depth to vary from about 5.0 to 6.25 feet. Because the basin is not divided into any separate cells or sections, it offers no flexibility to operate in special modes such as contact stabilization or step feed. Also, the basin cannot be removed from service unless the entire secondary process is shut down, resulting in bypassing screened raw wastewater to the river.

The aeration basin operates strictly as a completely mixed basin. Aeration and mixing are provided by floating mechanical aerators. Several areas within the basin receive little mixing energy. The poorly mixed areas have heavy scum accumulations. Grit has accumulated on the bottom in these areas as well. The grit is reportedly visible when the liquid level in the basin is low. The plant staff estimates the quantity of grit to be about 100 cubic yards. This represents a 3 percent decrease in aeration basin capacity.

The mixed liquor is pumped from the aeration basin to the secondary clarifiers at a constant rate. Because mixed liquor is pumped at a constant rate, the liquid level in the basin is controlled by varying the return sludge flow rate. The rate is manually adjusted on a daily basis to maintain the aeration basin level within the normal range. As the plant flow increases, the return sludge flow rate is *reduced*, providing a net increase in flow from the aeration basins. Conversely, as the plant flow rate decreases, the return sludge flow is *increased* to reduce the net flow from the basin. As long as plant flow remains low, the system works, resulting in a plant effluent with less than 10 milligrams per liter (mg/L) of BOD and suspended solids. The problem arises when plant flow reaches a point where the return sludge flow rate is too low to return the solids back to the aeration basin. At this point, the sludge blanket in the clarifier rises quickly and the solids are discharged in the effluent, resulting in a process upset. Alternatively, if the return sludge flow rate is maintained higher, the mixed liquor pumps cannot handle the entire mixed liquor flow (plant flow *plus* recycle flow). Consequently, the level in the aeration basin rises until it overflows to the outfall, resulting in a process upset.

The aeration basin also serves as an equalization basin. During short peaks in flow rate, the mixed liquor pumping rate remains constant. Hence, the peak is absorbed and the level in the basin rises. Flattening the flow peaks helps to counteract the problem caused by the limited sludge return rate described above. However, the usefulness of flow equalization is limited to flow peaks of short duration, typically a couple hours. The volume available for flow equalization is about 160,000 gallons.

The physical condition of the basin cannot be assessed because it cannot be drained without taking the entire process out of service. However, along the perimeter of the basin above the liquid surface, cracks in the asphalt liner are prevalent. Weeds are spreading in the cracks.

Secondary Sedimentation

There are two secondary clarifiers. Flow is distributed to them by throttling valves in the influent piping. Because the mixed liquor flow rate is usually constant, the valves can be set to optimize the flow distribution without requiring frequent adjustment.

The design loading rate for the clarifiers is 800 gallons per day per square foot. This is typical of the loading rate expected for shallow clarifiers of this type with outboard weirs without baffles. This loading rate would be exceeded during the current peak wet weather flow if flow equalization were not utilized. Hence, more clarifier capacity will be required as flows increase in the future. The sludge blanket often rises during high flows, but this is probably more attributable to the limited return sludge rate than to the surface overflow rate of the clarifier.

The 50-foot-diameter clarifier has separate sludge hoppers for waste sludge and return sludge. The 35-foot clarifier has a single hopper, with a waste sludge line branching off the return sludge piping.

Scum removal from the existing clarifiers is marginal. Excess scum accumulates on the water surface of the clarifier. The scum flows by gravity from the sump into the return sludge piping. However, large quantities of grease build up in the sumps.

Disinfection

The secondary effluent is disinfected by chlorination. Chlorine solution is added to the secondary effluent in a manhole near the secondary clarifiers. The current dosage of 15 pounds per day results in a concentration of about 2 mg/L, which is at the lower end of the typical range. The 150 pound per day capacity of the chlorinator is more than adequate to handle the maximum chlorine demand. There is no standby unit.

The contact basin is of similar construction to the aeration basin. It is an asphalt-lined pond with sloped sides. Underdrains provide a means for lowering the water table when the basin is drained.

The basin is adequately sized for the amount of wastewater that is currently able to flow through the plant (1.5 million gallons per day [mgd]), but not for the peak flow that could be delivered to the plant. A baffle divides the basin into two long, narrow (20 feet by 80 feet) parallel chambers. Although the shape is conducive to plug flow, the effluent is drawn directly into pipes about 15 feet upstream of the downstream end of the basin, potentially leading to some short circuiting. Collecting the effluent over a weir at the downstream end of the basin would reduce any short circuiting.

Performance of the disinfection system is generally good as long as the effluent quality is good. However, during high flows, the effluent quality degrades severely or some wastewater bypasses the plant entirely. During these incidents, the coliform count exceeds acceptable levels.

Sludge Thickening

Waste activated sludge is thickened to about 5 percent solids on a one-meter gravity belt thickener. The waste sludge passes through a grinder upstream of the thickener. The thickened sludge is pumped to the anaerobic digester by a progressing cavity pump. The capacity of the thickener is more than adequate for the existing plant. It is usually operated 2.5 to 3 hours per day.

The thickener feed system has a minor deficiency. The feed rate varies as the concentration of the sludge changes while pumping. The change in feed rate affects the balance of the polymer concentration adversely. This could be remedied by adding a feedback loop from the thickener feed flow meter back to the pump variable frequency drive. The system could then be programmed to vary the speed of the pump as necessary to maintain a constant flow rate.

The ventilation system in the building is generally good. However, there is no hood over the thickener. Consequently, when digested sludge is thickened, odors are present in the building.

Anaerobic Digestion

The anaerobic digester was constructed as part of the original treatment plant in 1961. It was designed as an unmixed digester with supernatant drawoff. It has since been upgraded to include sludge heating and mixing. The supernatant drawoff is no longer used.

The capacity of the digester is more than adequate now that the waste activated sludge is thickened. The organic loading rate and the hydraulic retention time indicate that the digester is lightly loaded. The digester normally achieves 55 percent reduction of volatile solids.

The digester is normally operated at 100 degrees F, although occasional variations occur. Other operational parameters including pH, alkalinity, volatile acids, and gas production are not regularly measured. There is no longer a waste gas burner at the plant. Consequently, gas is simply vented at the cover.

The digester appears to be in good structural condition. Walls have some minor hairline cracking, but no leakage or past evidence of leakage was observed. The digester was drained and cleaned around 1990. The plant staff inspected interior and judged it to be in good condition with no evidence of concrete deterioration.

Overall Performance

The overall performance of the plant is measured in terms of effluent BOD and total suspended solids (TSS) concentrations. The monthly average effluent quality for the period January 1993 through June 1996 is summarized in Table 3-3. The average concentrations for BOD and TSS are about 10 mg/L, which is considered a good quality effluent. However, the winter average is much higher, with TSS averaging 18 mg/L. The high average in the winter is caused by process upsets and raw sewage bypasses on individual high flow days. Such events were most notable in December 1995 and March 1996, resulting in the maximum reported monthly effluent concentrations. The process upsets and bypasses on high flow days are a result of limited mixed liquor pumping capacity and return sludge capacity, as discussed above in the section on aeration.

Overall, the secondary process requires major upgrades to allow the plant to meet permit requirements during high flow periods. As discussed above, the aeration basin offers no process flexibility or backup provisions. Multiple basins or cells will be required to meet the Department of Environmental Quality reliability criteria. Mixed liquor pumping capacity must be increased or the aeration basins raised to allow gravity flow to the clarifiers. Clarifier upgrades will also be required to handle the increase in peak flows resulting from elimination of bypasses and growth of the service area.

Table 3-3. Plant Effluent Monthly Averages

Month	Effluent			
	BOD, mg/L	BOD, ppd	TSS, mg/L	TSS, ppd
Jan-93	12	64	5	27
Feb-93	9	45	4	19
Mar-93	6	38	4	26
Apr-93	6	38	5	29
May-93	7	41	4	26
Jun-93	6	39	4	25
Jul-93	8	45	5	28
Aug-93	6	36	3	14
Sep-93	7	36	3	15
Oct-93	7	34	4	22
Nov-93	8	38	5	24
Dec-93	9	44	5	24
Jan-94	5	21	4	18
Feb-94	6	32	4	22
Mar-94	8	45	5	28
Apr-94	8	42	5	28
May-94	6	33	4	19
Jun-94	6	31	3	16
Jul-94	7	34	3	16
Aug-94	9	50	5	27
Sep-94	8	47	5	27
Oct-94	8	36	4	21
Nov-94	7	33	5	23
Dec-94	6	36	4	22
Jan-95	6	44	4	26
Feb-95	6	44	4	31
Mar-95	9	79	7	64
Apr-95	5	34	3	20
May-95	6	39	3	21
Jun-95	4	26	3	17
Jul-95	6	34	3	18
Aug-95	11	61	9	52
Sep-95	6	31	4	22
Oct-95	6	27	4	19
Nov-95	7	37	8	42
Dec-95	60	498	105	916
Jan-96	8	65	12	94
Feb-96	21	246	62	716
Mar-96	57	527	93	871
Apr-96	27	242	40	366
May-96	8	54	5	34
Jun-96	8	53	12	78
Max	60	527	105	916
Min	4	21	3	14
Avg	10	73	11	94
Winter avg	13	104	18	156
Winter max	60	527	105	916
Summer avg	7	39	5	26
Summer max	11	61	12	78

SOLIDS HANDLING

The city currently applies liquid digested sludge on land for beneficial use. Refer to Appendix D for the city's Sludge Management Plan and sludge analysis data. The sludge management plan provides useful information about the application sites and sludge loading rates; however, it appears to underestimate the quantity of sludge produced by the treatment plant.

Sludge Quantity

The city currently hauls about 3,000 gallons per day of digested sludge to application sites. Using a 3,000-gallon tank truck, this averages to about one trip per day. Plant staff report that the truck is in a rather worn condition and is becoming less reliable. Based on the quantity of solids wasted from the secondary process per day and assuming a volatile solids destruction of 55 percent in the digester, the estimated quantity of solids removed from the digester is 800 pounds per day.

Sludge Quality

The sludge meets the Environmental Protection Agency (EPA) requirements for Class B biosolids. Refer to Chapter 5 for a description of EPA categories and requirements for biosolids. The metals concentrations are well below EPA limits for a clean sludge. Pathogen reduction and vector attraction reduction to meet Class B standards are provided by the anaerobic digestion process.

Application Sites

The city currently applies sludge on six sites, totaling about 150 acres. The city is negotiating a contract for sludge application on a seventh site of about 40 acres. Except for the airport site, all the sites are on privately owned land. Sludge is applied in accordance with contracts between the city and the land owners. The land area currently available for sludge application is barely adequate for the quantity of sludge generated. The sites reach their limit each year for agronomic loading rates for nitrogen. The new 40 acre site will improve the situation, but it is about 60 miles from Florence. Hauling sludge this distance is time consuming and costly.

COLLECTION SYSTEM

In this section, the collection system is described and observed problems identified. Infiltration and inflow are evaluated. The results of a flow modeling study are then presented.

DESCRIPTION

The collection system was originally constructed in 1961 and has been expanded periodically as required. Most of the system is in good condition except for some of the older sewers in the downtown area and other isolated areas. Because the topography includes high dune areas and low interdune areas, the system consists of many small basins, each with a pump station and pressure main to convey sewage from the basin to the plant. All wastewater enters the treatment plant via pressure mains. A map of the collection system showing the major sewers and pump stations is shown on Figure 3-3.

Gravity Sewers

The gravity system includes pipes ranging from 6 to 14 inches in diameter. Only a few isolated reaches at the upper end of some basins are 6 inch diameter. There is one 14-inch interceptor that conveys wastewater to the Ivy Street pump station. Most of the sewers are 8 inch diameter. Slopes generally range from 0.2 to 1.5 percent, although isolated reaches have steeper slopes. Because the high water table makes deeper excavation difficult, most of the sewers are shallow, as little as 4 feet below grade. Pipe materials are mainly asbestos concrete and PVC, although a substantial number of sewers are concrete. One reach in the oldest part of the system is clay pipe, but that section is scheduled to be replaced in the near future. All of the more recently constructed sewers are PVC pipe.

Pump Stations and Pressure Mains

The collection system includes 27 pump stations and associated pressure mains, ranging from 4 to 8 inches in diameter. The characteristics of the pump stations are summarized in Table 3-4. Most of the pump stations (except the Ivy Street pump station) are packaged duplex units with self-priming pumps. A few of the stations have submersible pumps. The Ivy Street pump station was custom designed and constructed. It has a separate wet well and dry well.

The pump station wet wells and discharge manholes were inspected for evidence of hydrogen sulfide corrosion. The inspections consisted of probing and scraping the concrete surfaces with a screwdriver to ascertain the condition of the concrete. Several of the pump stations discharge directly into a manifold pressure main that discharges at the treatment plant. Although the discharge from these pump stations could not be evaluated, there is no evidence of hydrogen sulfide corrosion at the treatment plant headworks. Of the pump stations that could be inspected, only the Vine Street discharge was in poor condition.

Problem Areas

City staff report that there is one location in the collection system where bypasses have occurred: immediately upstream of the Ivy Street pump station. Occasionally during high winter flows, the capacity of the pump station is exceeded and raw sewage is bypassed through a short ditch directly to the Siuslaw River. Although several other pump stations have bypass facilities, no other incidents of bypassing or overflowing manholes have been reported. Observations made during high storm flow periods indicate that even under high flow conditions, no surcharging of sewers takes place.

Several reaches of sewers in the older parts of the system are structurally defective. Many of the suspect sewers were inspected in the fall of 1996 using remote controlled closed circuit TV cameras. About 20 sections were inspected, totaling about 4,000 lineal feet. One section constructed of clay pipe has several large holes and breaks, allowing significant amounts of infiltration and inflow (I/I) and grit to enter the system. Several other reaches could not be inspected because the camera could not pass by breaks in the pipe. All these sections are scheduled for replacement in the near future.

Several other problem areas in the gravity system have been reported and corrected by the city. Smoke testing completed by the city several years ago indicated the presence of connections from storm drains and roof drains. These inflow sources have been eliminated. Another source of inflow consisted of manholes in low-lying areas subject to ponding. During winter, significant inflow entered the system through submerged manhole covers. This problem has been eliminated by raising these manholes above the water level.

With the exception of the Ivy Street and Maple Street pump stations, few problems are reported with the pump stations. The pump stations have ample capacity for the current flows and perform reliably. As discussed above, examination of pump station and pressure main discharges has revealed little evidence of hydrogen sulfide problems, except at the Vine Street pump station.

The Ivy Street pump station is in good condition. It has been refurbished recently with new controls and impellers. However, as discussed above, its capacity is occasionally exceeded resulting in raw sewage bypasses. When a new interceptor is constructed (refer to Collection System Improvements below), the flow to the pump station will be reduced nearly in half, alleviating the capacity problem.

The pumps in the Maple Street pump station need refurbishing. The impellers are worn resulting in greatly reduced pumping efficiencies. An overhaul of this pump station is planned in the near future.

INFILTRATION AND INFLOW ANALYSIS

The goal of the I/I evaluation is to determine the extent of the problem and to determine the cost-effective limits of an I/I reduction program. This analysis is based on flow and run-time records of the pump stations throughout the collection system and flow at the treatment plant.

Infiltration and Inflow Guidelines

I/I can be handled by either conveyance and treatment or removal. An I/I analysis aids in determining the more economic option. The first step in evaluating I/I is to determine whether it is excessive. If it is non-excessive, it is assumed that removal is not cost-effective and no further analysis is required. The EPA has established guidelines for the preliminary determination of non-excessive infiltration and inflow.

Infiltration. The guideline for infiltration is based on a dry weather flow defined as the highest 7-day average flow recorded over a 7- to 14-day period during seasonally high groundwater. This condition would occur in the winter when no precipitation falls during a 7- to 14-day period. If the flow during such a period exceeds 120 gallons per capita per day (gcd), the infiltration is considered excessive. For a service population of 6,200, this results in a total system flow of 0.75 mgd. During the winter of 1995-1996, there were five 7-day periods with little or no rain. The flows during these periods are summarized in Table 3-5.

Table 3-5. High Groundwater Dry Weather Flows

Period	Seven-day average flow, mgd	Seven-day average flow, gcd	Total precipitation, inches
12/20/95 through 12/26/95	0.79	127	0.00
2/9/96 through 2/15/96	1.33	214	0.01
3/13/96 through 3/19/96	0.97	156	0.07
4/2/96 through 4/8/96	0.81	131	0.02
5/4/96 through 5/10/96	0.81	131	0.00
Average	0.94	152	0.02

As the table shows, the system flow exceeded the guideline amount of 0.75 mgd during each of the periods analyzed. Therefore, infiltration may be excessive and a more detailed infiltration analysis is required. This analysis is presented in subsequent sections.

Inflow. The EPA guideline for inflow is 275 gcd based on wet weather flow, defined as the highest daily flow recorded during a storm event. For a service population of 6,200, this results in a total system flow of 1.72 mgd. The highest reported daily flow was 1.58 mgd in February 1996. Although some bypassing occurred on that date, it is estimated to be less than 0.15 mgd. This results in a total flow of less than 1.73 mgd, essentially within the EPA guideline. Examining plant flow records for the years 1993 through 1994 provides additional evidence that inflow is not excessive. As discussed in Chapter 4, peak flows remained rather low even during winter storms. A relatively dry period occurred during these winters, resulting in lower groundwater levels throughout the winter. The fact that peak flows coinciding with storms during this period were less than 1 mgd indicates that inflow is minor.

It is not surprising that the inflow is minor because the city has eliminated the visible sources indicated by smoke testing. Manholes in low areas subject to ponding were raised. In other cases, special manhole covers were provided to eliminate inflow.

Infiltration Rates

As discussed above, infiltration rates in the Florence wastewater collection system may be excessive. To determine the amount of infiltration from individual basins, wet weather and dry weather flows from each basin are compared by examining the flows from the pump stations. The average daily pump station and plant flows from dry and high groundwater periods are presented in Table 3-6. The first line of the table shows that the pump stations account for about one third of the plant flow during dry conditions. The remaining two-thirds of the flow originates in the basin served by the Ivy Street pump station. The Ivy Street pump station receives much of its flow from other pump stations, making it impossible to determine how much of the Ivy Street flow originates in the local basin. The bottom two lines of the table show the flow increase during a high groundwater period. This increase is attributable to infiltration.

Table 3-6. Summary of Infiltration From Pump Station Records[a]

Condition	Pump station flow, gpm	Plant flow, gpm
Dry (Oct 1-7, 1995)	127	380
High groundwater (Feb 9-15, 1996)	146	920
Increase during high groundwater	19	540
Percent increase	14	140

Notes: [a]Ivy Street pump station not included.

From Table 3-6, the amount of infiltration contributed by the monitored pump stations is only 19 gpm, almost negligible. Low infiltration is expected because much of the piping in these areas was installed recently, constructed with PVC pipe with gasketed joints. Because so little infiltration originates in the monitored basins, most of the infiltration must come from the Ivy Street basin, as expected. This area encompasses the oldest part of town which has many old, failing sewers. City staff report that sewers in this area have structural failures and leaky joints. Recent TV inspection of some of these reaches verified the poor condition of these sewers. Several reaches had joints offset too far to allow the camera through. Many lateral connections were damaged as well. At some lateral connections, significant quantities of gravel were deposited in the sewer. One reach of clay pipe had sections of crushed pipe with significant openings. This reach alone may contribute as much as 20 gpm, or 4 percent, of the infiltration indicated in Table 3-6. This represents about 15 percent of the infiltration removal required to lower the average plant flow during the high groundwater season to 0.75 mgd, the level considered non-excessive. Refer to Table 3-5.

Infiltration Removal Program

The above discussion shows that most of the basins in the collection system contribute very little infiltration. Sewer rehabilitation should be concentrated in the Maple Street and Ivy Street basins. The city is aware that the sewers in these areas are problematic and is currently in a program to replace the failing sewers. Approximately 2,500 lineal feet of sewer will be replaced this fall. The sections to be replaced are those for which recent TV inspection showed significant structural problems. These reaches are shown on Figure 3-4. Because these sections are structurally deficient, they should be replaced on the basis of structural inadequacy, regardless of infiltration analysis results. The city plans to continue the program of inspecting sewers in the older parts of the system and fixing

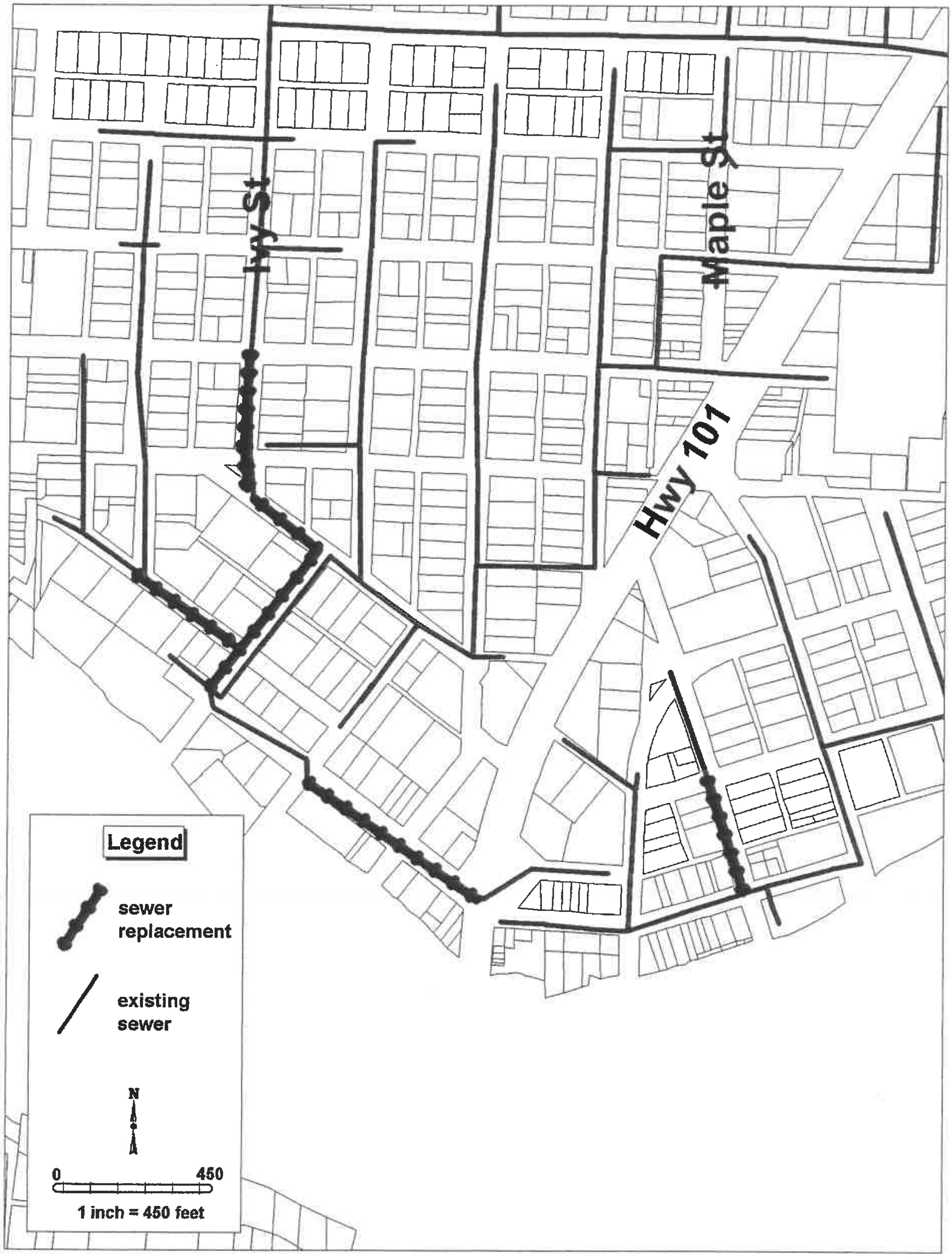

Figure 3-4. Sewer Rehabilitation Sites

structural deficiencies. Based on the low levels of infiltration in other basins, it is expected that once the structural deficiencies are corrected, infiltration will be non-excessive.

If infiltration reduction is less than expected and it continues to be excessive, a rigorous cost-effectiveness analysis should be performed for the remaining sewers in the Maple Street and Ivy Street basins. Based on the preliminary cost estimates for the recommended plan for upgrading the treatment plant, the cost to provide additional hydraulic capacity is about $1,500 per gpm treated. Therefore, it would be cost-effective to perform rehabilitation work that results in a flow reduction of greater than 1 gpm per $1,500 spent. Assuming a cost of about $100 per foot for sewer replacement under city streets and 70 percent removal of infiltration from rehabilitated lines, rehabilitation could be cost-effective for those sewers that contribute more than 0.1 gpm per foot of pipe.

FLOW MODELING OF EXISTING SYSTEM

A computer model of the Florence wastewater collection system was developed to pinpoint possible capacity problems in the system under existing conditions and to predict future problem areas. Improvements to the collection system are recommended based on the model results.

Flow Model Description

The software selected was XP-SWMM, a graphical version of the widely-used EPA Storm Water Management Model. The SWMM model consists of several computational blocks, including RUNOFF, TRANSPORT, and EXTRAN. The RUNOFF block simulates both the quantity and quality runoff phenomena of a drainage basin as well as routing flows and pollutants into sewer lines using a nonlinear reservoir methodology. The TRANSPORT block routes flows using a kinematic wave approach, usually for larger pipes than the RUNOFF block. Both RUNOFF and TRANSPORT work poorly for handling surcharging pipes and neither can handle backwater effects. EXTRAN is used for these more complicated hydraulic situations, since it provides for solution of the complete Street Venant (gradually varied flow) equations. The Florence system was modeled by generating hydrographs and solving using the EXTRAN block.

In addition, land use, population, pipe lengths, elevations, and other data were incorporated using the geographic information system (GIS), ARC-VIEW. ARC-VIEW is compatible with the GIS, ARC-INFO used by Lane Council of Governments (LCOG), allowing the use of LCOG data in the model. The model includes only the pipe network; the pump stations were analyzed separately, as discussed previously in this chapter.

Sanitary flows were generated in the model by assuming the residential and employee densities provided by city planning staff. A sanitary flow of 107 gcd for residential lots and 27 gcd for commercial and industrial employees was assumed. The employees were assumed uniformly distributed throughout the commercial areas.

Model Results

The existing wastewater collection system was modeled under three scenarios: existing conditions, projected population and employment for 2020, and projected population and employment for buildout. A detailed map showing the modeled system components is shown in Appendix A. Summaries of the results are discussed below.

Existing System. The collection system piping appears adequately sized for existing conditions. Figures in Appendix A show the capacity and flow in the system at each node in the model. As the figures show, capacity exceeds flow at each node. The existence of excess capacity is confirmed by city staff who report no evidence of surcharging in the system. However, as discussed earlier, the Ivy Street pump station is overloaded, resulting in occasional bypasses.

Year 2020 Model. As population and commercial land use increase, the collection system will become overloaded. Capacity problems will develop in the main interceptor along Highway 101 between 10th and 8th Streets (model nodes 1365-1350), along 8th Street from Laurel to Ivy, and on Ivy from 8th to 4th Street. Refer again to the figures in Appendix A for these flows and capacities.

Buildout Conditions. By the time the entire study area is completely built out, the collection system will experience additional capacity problems. Isolated sewers on the east side will be overloaded as a result of flows from new developments in that area. The sewer along Oak Street between 28th and 22nd will also be overloaded. Surcharging will increase beyond that experienced in the year 2020 in the sewers along the sections of Highway 101, 8th Street, and Ivy Street. Refer again to the figures in Appendix A for these flows and capacities.

CHAPTER 4

WASTEWATER CHARACTERISTICS

In this chapter the current wastewater flows and loads are presented. These are then used together with population projections developed in Chapter 2 to develop the design flows and loads expected in the future.

CURRENT FLOWS AND LOADS

Developing accurate estimates of current plant flows and loads is a critical step in the facilities planning process. The current flows and loads serve as the basis for estimating future flows and loads; these flow and load projections are in turn used in the sizing of new wastewater treatment and conveyance facilities. For this evaluation, wastewater treatment plant (WWTP) and wastewater pump station operating records were analyzed for January 1993 through June 1996.

WASTEWATER FLOWS

Several different average and peak flow rates are necessary for different aspects of facility design. These flows are defined below, and then developed, based on plant flow data and rainfall records.

Definitions of Flow Terms

Flow rates which are important in the design and operation of treatment plants include:

- The *average dry weather flow* (ADWF) is the average flow at the plant during the dry weather season, usually defined as May through October. The ADWF is used by the Oregon Department of Environmental Quality (DEQ) for calculating mass discharge limits for biochemical oxygen demand (BOD) and total suspended solids (TSS) for the dry weather season.
- The *average wet weather flow* (AWWF) is the average flow at the plant during the wet weather season typically November through April. The AWWF is used for calculating mass discharge limits for BOD and TSS for the wet weather season.
- The *maximum month dry weather flow* (MMDWF) is defined by DEQ as the flow experienced at the WWTP when rainfall quantities are at the 1-in-10 year probability level for the month of May. MMDWF is important in the design of effluent irrigation and storage systems.
- The *maximum month wet weather flow* (MMWWF) is defined by DEQ as the flow at the WWTP when rainfall quantities are at the 1-in-5 year probability level for the month of January. MMWWF is used in the design of a plant's secondary process.
- The *peak day flow* is the flow rate at the plant that corresponds to a 1-in-5 year, 24-hour storm event that occurs during a period of high groundwater and saturated soils.

- The *peak wet weather flow* (PWWF) is expected to occur during the peak day flow. The PWWF is the highest flow at the plant sustained for one hour. The PWWF dictates the hydraulic capacity of the WWTP. This flow is also known as the peak instantaneous flow.

Flow Records

Historical daily plant flows are depicted in Figure 4-1. It should be noted that flows increased significantly and became far more variable starting in January 1995. Prior to this time, western Oregon was experiencing an extended period of drought. City staff report that groundwater levels were below the elevation of most of the sewers before the start of 1995. However, since that time, increased rainfall has raised the groundwater table and infiltration into the sewers has increased significantly.

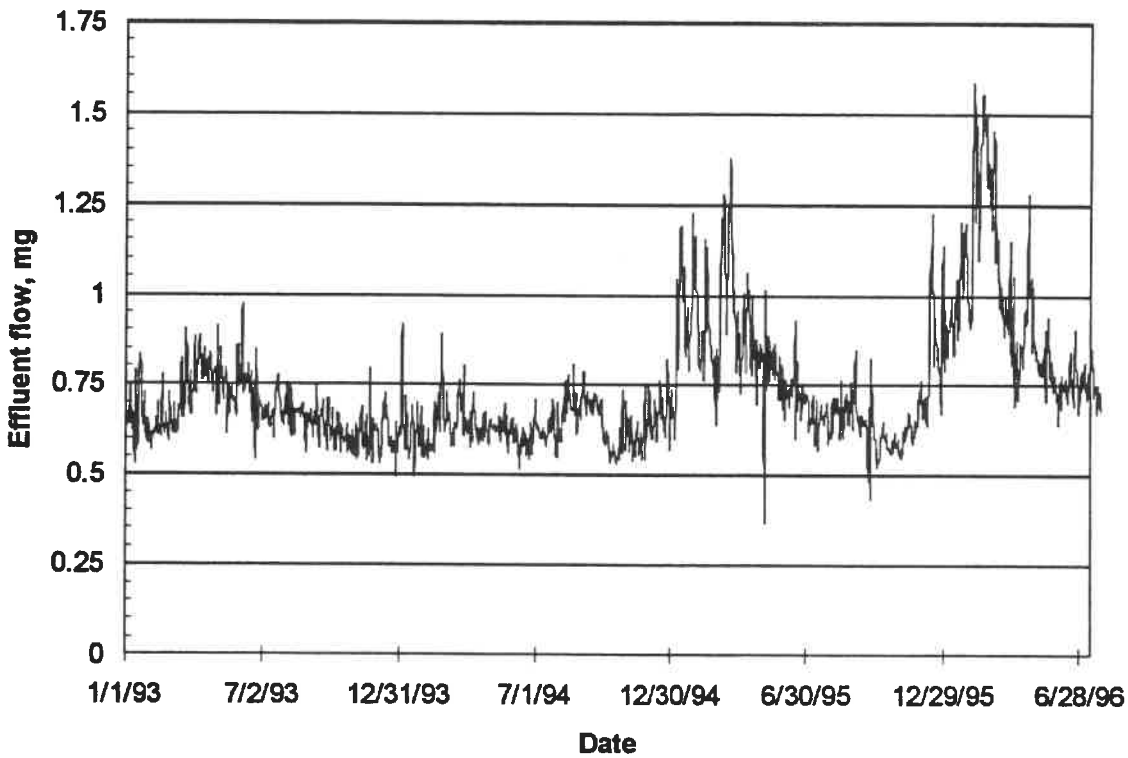

Figure 4-1. Daily Plant Flows

In evaluating wastewater flows records, it is necessary to first identify any limitations in flow measurement or pumping capacity. In addition, any unique or unusual conditions which could affect historical flow records should be ascertained. At the Florence plant, four physical limitations affect the accuracy of historical flow records:

- Effluent flow meter. The existing flow element is a V-notch weir with a level detector; it measures the flow rate of plant effluent. The capacity of the V-notch is limited to about 1.5 million gallons per day (mgd). At flow rates above 1.5 mgd, the meter reading is skewed lower than the true flow rate because effluent flows over the entire width of the weir. Also, measured effluent flow does not reflect influent flow peaks because the aeration basin acts as a surge basin.
- Ivy Street pump station. The Ivy Street pump station reportedly conveys approximately 75 percent of Florence's wastewater to the WWTP. The remaining flow comes from multiple pump stations feeding a separate force main. During extreme rainfall events, the Ivy Street pump station is unable to keep pace with the incoming wastewater flow, resulting in sewage bypasses. Wastewater that overflows the Ivy Street pump station is not measured by the plant flow meter. Overflows at Ivy Street are noted on the plant operating records; however, there is no way to accurately measure the quantity of wastewater that is bypassed.
- Plant bypasses. As discussed previously, the plant has an interstage pump station which conveys mixed liquor from the aeration basin to the secondary clarifiers. When the capacity of this pump station is exceeded for extended periods, mixed liquor overflows from the aeration basin to the chlorine contact basin. The plant bypasses are noted on operating records; however, accurate estimates of bypass volume are often not possible because they usually occur when the capacity of the effluent flow meter is exceeded.
- Aeration basin. The water depth in the aeration basin can fluctuate by as much as 15 inches. As influent flows increase, the water level in the aeration basin increases. Therefore, the aeration basin acts as a surge basin, dampening the effects of peak flows. A 15-inch change in aeration basin depth represents a volume of about 160,000 gallons.

As a result of the above factors, the peak flow records for the Florence WWTP are not highly accurate. Therefore, the analysis of peak flows considers only selected data which best represent actual peak flows. Furthermore, assumptions must be made in some cases to fill in gaps in data.

Rainfall Records

Peak wastewater flows are heavily influenced by rainfall. Therefore, the techniques suggested by DEQ for calculating plant flows require consideration of statistical recurrences of rainfall quantities. Statistical rainfall analyses for Florence are unavailable; however, there are statistical rainfall summaries for Reedsport, approximately 20 miles south of Florence. Table 4-1 compares monthly average rainfall values for Honeyman State Park (3 miles south of the treatment plant) and Reedsport. The average rainfall quantities for Honeyman State Park and Reedsport are similar; therefore, the statistical analysis of Reedsport rainfall records will be used to approximate Florence rainfall.

Table 4-1. Rainfall Comparison

Month	Honeyman State Park Average rainfall, inches	Reedsport Average rainfall, inches
January	9.97	13.28
February	9.66	9.62
March	9.32	9.61
April	4.92	5.51
May	3.76	3.33
June	2.43	1.74
July	0.94	0.49
August	1.31	1.21
September	2.32	2.38
October	5.27	5.54
November	10.90	10.48
December	11.75	13.20
Total	72.09	76.39

Monthly Flows

Monthly average flows for January 1993 through June 1996 are presented in Table 4-2. The annual average flow for this period was 0.733 mgd.

The ADWF is essentially unaffected by rainfall. Therefore, the ADWF is taken as the average May through October flow for the latest full dry weather season for which records are available (1995), or 0.68 mgd.

The AWWF is influenced by rainfall. Figure 4-2 plots average flow and total rainfall for November through April for the past 3 years. Because the flows in 1993-4 and 1994-5 were unusually low due to low groundwater, a line drawn through the ADWF and 1995-6 points is more representative of wet weather flows. The long-term average November through April rainfall of 56.52 inches corresponds to a November through April flow of 0.85 mgd. However, because the effluent flow meter is inaccurate at flows above 1.5 mgd, the wet weather flow data are skewed lower than actual flows, as indicated previously. Although the maximum monthly average flows have not exceeded 1.5 mgd, the averages are still affected by the flow meter error because peak flows during the month are skewed lower, thereby lowering the reported monthly average. Plant staff estimate that the true wintertime monthly flows may be about 10 percent higher. This estimate is based on measurements taken manually from an influent Parshall flume. Measurements have been taken at about 2-hour intervals on a daily basis since the fall of 1995. Increasing the calculated wet weather flow by 10 percent yields an AWWF of 0.94 mgd. Inaccuracy in the determination of AWWF does not affect treatment plant design; the flows used in design (maximum month, peak day, and PWWF) are developed independently.

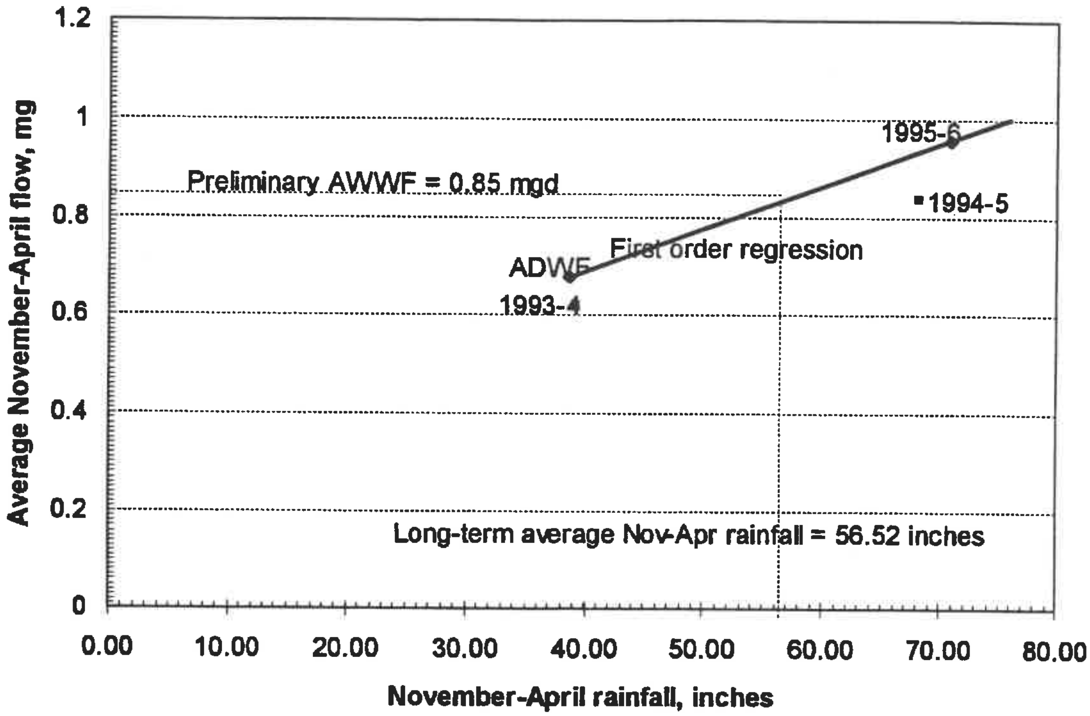

Figure 4-2. Average Wet Weather Flow

Table 4-2. Monthly Flows

Month	Flow, mgd
Jan-93	0.666
Feb-93	0.630
Mar-93	0.697
Apr-93	0.797
May-93	0.741
Jun-93	0.740
Jul-93	0.679
Aug-93	0.670
Sep-93	0.646
Oct-93	0.606
Nov-93	0.601
Dec-93	0.607
Jan-94	0.624
Feb-94	0.631
Mar-94	0.653
Apr-94	0.627
May-94	0.630
Jun-94	0.589
Jul-94	0.620
Aug-94	0.675

Month	Flow, mgd
Sep-94	0.707
Oct-94	0.579
Nov-94	0.606
Dec-94	0.669
Jan-95	0.911
Feb-95	0.911
Mar-95	1.007
Apr-95	0.878
May-95	0.787
Jun-95	0.725
Jul-95	0.656
Aug-95	0.676
Sep-95	0.651
Oct-95	0.581
Nov-95	0.636
Dec-95	0.855
Jan-96	0.985
Feb-96	1.314
Mar-96	1.059
Apr-96	0.900
May-96	0.800
Jun-96	0.749
Max	1.314
Min	0.579
Avg	0.733
Winter avg	0.785
Winter max	1.314
Winter min	0.601
Summer avg	0.675
Summer max	0.800
Summer min	0.579

To calculate maximum month flows, DEQ recommends plotting monthly plant flows and associated rainfall values for January through May of the most recent year (Figure 4-3). The MMWWF is estimated as the flow at the plant corresponding to the 1-in-5 year January rainfall. For the nearby weather station at Reedsport, the 1-in-5 year January rainfall is 18.56 inches. Therefore, from Figure 4-3, the MMWWF is assumed as 1.6 mgd. This value compares well to the maximum month flow reported since January 1993: 1.35 mgd in January 1997.

In similar fashion, the MMDWF is approximated as the flow associated with a 1-in-10 year May rainfall (5.93 inches). From Figure 4-3, the MMDWF is 1.0 mgd. The highest dry weather flow reported since January 1993 was 0.80 mgd in May 1996 (Table 4-2).

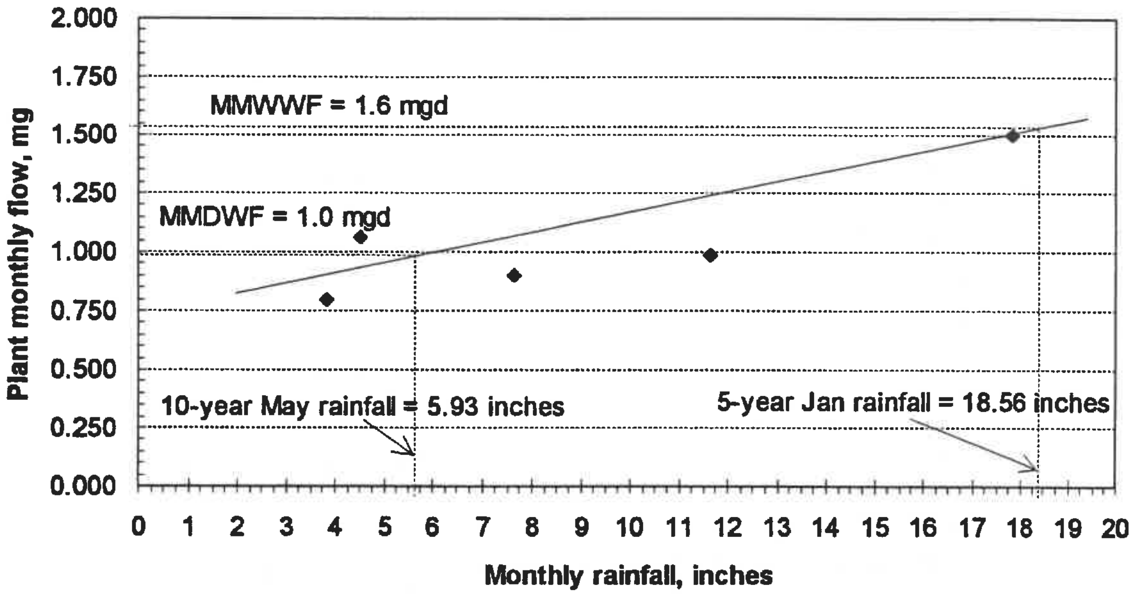

Figure 4-3. Maximum Month Flows

Peak Flows

The peak flows of interest are the peak day flow, the peak week flow, and the PWWF. The peak day flow is estimated as the flow associated with the 1-in-5 year, 24-hour storm event. For Florence, this storm event is 4.5 inches of rainfall. However, it is also important to ensure that the antecedent conditions contribute to maximum infiltration and inflow to the collection system. That is, the groundwater level should be high and there should be several days of significant rainfall prior to the 1-in-5 year, 24-hour storm event to ensure soil saturation. However, because some peak flows are bypassed and are not measured, only past storm events with no plant or pump station bypasses, probable high groundwater conditions, and several days of rainfall preceding the storm were considered. Unfortunately, this limits the evaluation to moderate storm events; peak storms must be omitted due to bypasses. Table 4-3 lists the storm events considered in estimating peak day flow. Note that the storm on February 8, 1996, resulted in a reported flow of 1.6 mgd as measured by the effluent meter. As discussed previously, the meter is inaccurate at flows above 1.5 mgd. Readings taken at 2-hour intervals throughout the day from the influent Parshall flume indicate an average daily flow of 1.8 mgd for that date. This data point is a conservative estimate because the readings were taken during the day, when flows tend to be higher than during the night. A first-order regression line through these points indicates a peak day flow of about 2.5 mgd (Figure 4-4). The slope of this line is driven largely by the data point representing the February 8 flow, because that event occurred during substantially higher rainfall than the other data points represent. The conservative estimate for the February 8 flow results in a conservative estimate for the peak day flow.

Table 4-3. Storm Events

Date	Rain, inches	Plant flow, mgd[a]
1/9/95	1.77	1.043
1/11/95	1.98	0.89
1/12/95	1.72	1.08
1/13/95	2.12	1.18
1/29/95	1.32	0.982
3/9/95	1.91	1.2
4/12/95	1.6	1
2/8/96	2.45	1.8[b]
2/17/96	1.65	1.402
2/18/96	1.73	1.422

Notes

[a]Evaluation limited to storm events where no bypasses were reported, high groundwater table was anticipated, and rainfall occurred for several days before storm.

[b]Reported value was 1.6 mgd. However, effluent flow meter provides an inaccurate low reading for flows above 1.5 mgd. Influent flume measurements taken frequently throughout the day indicated an average flow of about 1.8 mgd.

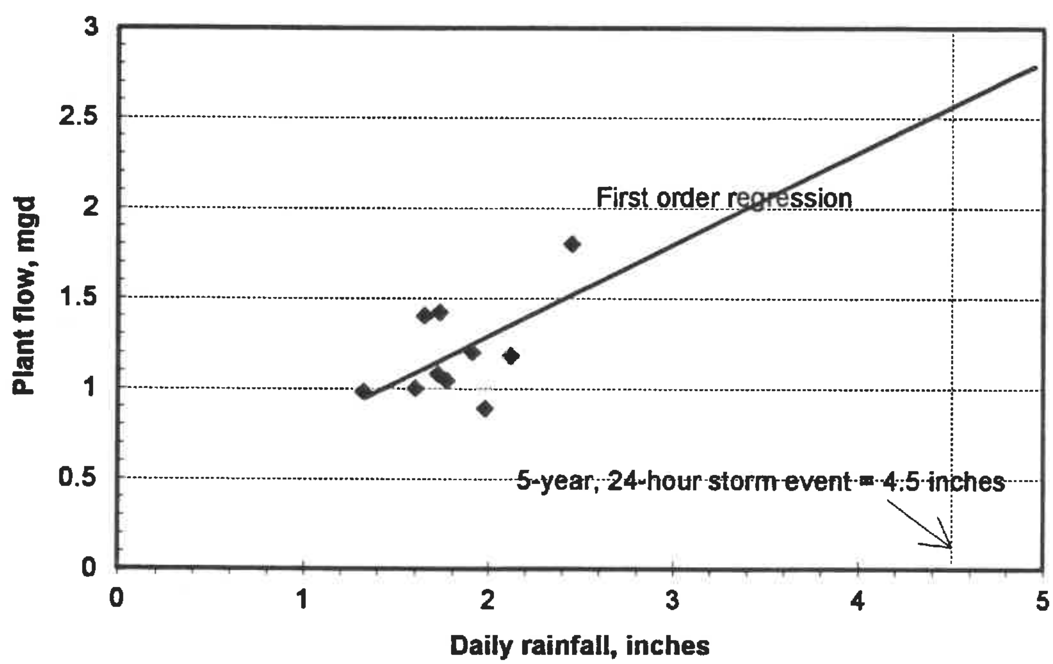

Figure 4-4. Peak Day Flow

The DEQ suggests using probability methods to estimate other peak flows. From the above analysis of rainfall and historical flow data, three flow rates and their associated recurrence probability are known: annual average flow, MMWWF, and peak day flow. The annual average flow has a recurrence probability of 50 percent. Assuming that the wet weather flows of interest all occur during a year with 1-in-5 year recurrence probability rainfall, the MMWWF has a recurrence probability of 1 month in 12 months, or 8.33 percent. Similarly, the peak day flow has a recurrence probability of 1 day in 365 days, or 0.27 percent. As predicted in the DEQ flow calculation guidelines, plotting these three points on log-probability scales approximates a straight line (Figure 4-5).

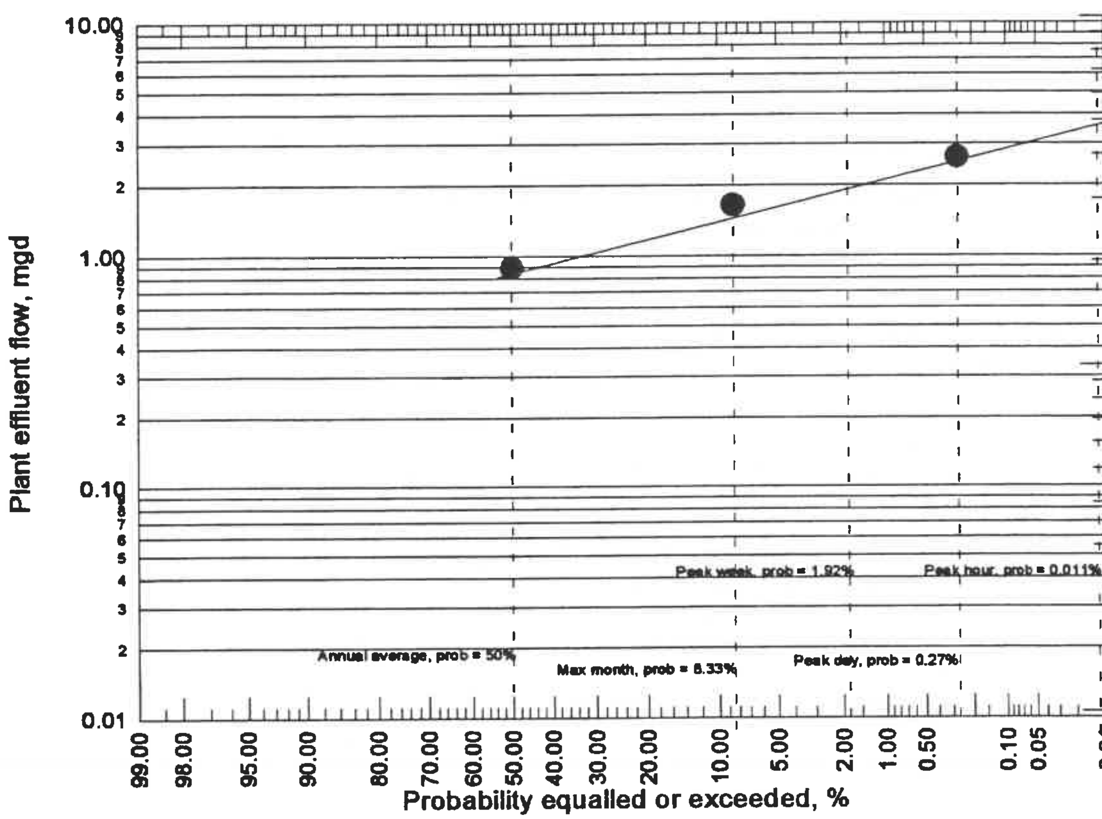

Figure 4-5. Flow Probability Analysis

Figure 4-5 can now be used to estimate PWWF and peak week flow. PWWF is defined as the peak flow sustained for 1 hour. PWWF has a recurrence probability of 1 hour in 8,760 hours (1 year), or 0.011 percent. From Figure 4-5, the current estimated PWWF is about 3.6 mgd. For a rough check on this estimate of PWWF, peak instantaneous flows as measured at the influent flume were analyzed. The peak flows observed during the past two winters and the associated precipitation are presented in Table 4-4. As the table shows, a maximum flow of 2.2 mgd has been observed once; a flow of 2.0 mgd has been observed on several occasions during high flow days. The observed flow readings are expected to be substantially lower than the estimated PWWF because they probably do not represent the true PWWF condition (peak hour flow with a

5-year recurrence), and limitations at Ivy Street pump station would reduce the peak flow observed at the plant. Therefore, the observed flow of 2.2 mgd compares well with the estimated PWWF of 3.6 mgd.

Table 4-4. Peak Instantaneous Flows Observed at Treatment Plant

Date	Observed peak flow, mgd	Rainfall, inches	Previous day's rainfall, inches
2/7/96[a]	2.0	1.3	4.2
2/11/96	2.0	0	0
2/19/96	2.0	0.7	1.7
1/2/97	2.2	0.5	0.5
1/20/97	2.0	1.07	0.6
1/22/97	2.0	0	0.7
1/31/97[a]	2.0	0.56	3.6

Notes: [a]Bypass occurred at Ivy Street pump station on this date. Bypass estimated by plant staff at less than 0.1 mgd.

Peak week flow is estimated in the same manner as PWWF. The peak week flow has a recurrence probability of 1 week in 52 weeks (1.92 percent); this corresponds to a flow of about 2.0 mgd.

Current flows for the Florence WWTP are summarized in Table 4-5.

Table 4-5. Current Wastewater Flows

Description	Flow, mgd
ADWF	0.68
Average annual flow	0.77
AWWF	0.94
MMDWF	0.93
MMWWF	1.6
Peak week flow	2.0
Peak day flow	2.5
PWWF	3.6

BOD AND TSS LOADS

The BOD and TSS loads at a treatment plant affect the following factors:

- Secondary process sizing. The design of a secondary process is based on the BOD load.

- Aeration system design. The capacity of the aeration system is determined by the peak BOD load.
- Sludge production. BOD and TSS removed by the plant are converted into sludge that must be stabilized and disposed of.
- Solids treatment and handling system design. Solids handling facilities, such as digesters and thickeners, must be sized to accommodate expected sludge quantities.

Current plant BOD and TSS loading is evaluated below.

BOD and TSS Records

As with plant flows, it is important to identify any limitations or irregularities in the historical data. For the Florence BOD and TSS records, it is significant that the influent sampler is located downstream of fine mesh screens and grit removal tanks. Because the screens have a very narrow spacing between the bars (0.06 inches), they remove a significant portion of the raw sewage BOD and TSS. The BOD and TSS removal probably increases still further as the screens become clogged with solids and the effective bar width spacing is reduced.

There are no data available with which to estimate the BOD and TSS removal efficiency of the screens and grit tanks. The removal efficiency probably varies with the hydraulic load on the screens. For this analysis, it is assumed that the screens remove 20 percent of the TSS and 10 percent of the BOD in the raw sewage. These values can be verified by sampling upstream and downstream of the screens and grit tanks.

Monthly Plant Loading

Table 4-6 summarizes plant BOD and TSS concentrations and loads for January 1993 through June 1996. As discussed above, the influent values were calculated from the reported values assuming 20 percent TSS removal and 10 percent BOD removal rates through the screens and grit system.

Examining the monthly loading can reveal whether seasonal variations in load occur. For example, one might expect an increase in load during the summer tourist season. As shown in Table 4-6, the average dry weather and wet weather loads to the plant are essentially identical; there appears to be no seasonal variation in plant loading. For the current sewer service area population of about 6,000, the average BOD load of 1,883 pounds per day (ppd) corresponds to a unit load of 0.31 pounds per capita per day (pcd). This is significantly higher than the textbook value of 0.2 pcd. Possible explanations for the high BOD values include the prevalence of recreational vehicle dump sites and contributions from the marina. Additional sampling in various parts of the collection system should provide a clearer indication of the source of the high BOD loads.

Dividing the average TSS load of 1,347 ppd by the current population results in a unit load of 0.22 pcd close to the textbook value of 0.2 pcd. This information also suggests that some unidentified, high strength soluble load is entering the wastewater collection system.

Table 4-6. Monthly Plant Loading, BOD and TSS

Month	Flow, mgd	Screened[1]				Raw sewage[2]			
		BOD, mg/L	BOD, ppd	TSS, mg/L	TSS, ppd	BOD, mg/L	BOD, ppd	TSS, mg/L	TSS, ppd
Jan-93	0.666	233	1,279	185	1,050	259	1,422	231	1,313
Feb-93	0.630	324	1,712	188	990	360	1,902	235	1,238
Mar-93	0.697	344	2,009	193	1,116	382	2,233	241	1,396
Apr-93	0.797	288	1,870	165	1,078	319	2,078	207	1,347
May-93	0.741	321	1,935	164	998	356	2,150	205	1,247
Jun-93	0.740	323	2,030	173	1,088	358	2,255	217	1,359
Jul-93	0.679	351	1,978	196	1,105	390	2,197	245	1,382
Aug-93	0.670	350	1,962	207	1,159	389	2,180	259	1,449
Sep-93	0.646	332	1,708	191	988	369	1,898	239	1,235
Oct-93	0.606	363	1,785	187	926	403	1,984	234	1,158
Nov-93	0.601	339	1,669	180	891	377	1,855	225	1,114
Dec-93	0.607	301	1,563	175	903	334	1,737	219	1,128
Jan-94	0.624	304	1,437	183	865	338	1,597	229	1,082
Feb-94	0.631	262	1,397	185	966	291	1,553	231	1,208
Mar-94	0.653	248	1,321	206	1,104	276	1,468	258	1,381
Apr-94	0.627	372	1,921	253	1,294	414	2,135	316	1,617
May-94	0.630	315	1,637	178	927	350	1,819	223	1,159
Jun-94	0.589	283	1,417	179	893	315	1,574	223	1,116
Jul-94	0.620	311	1,583	197	1,003	345	1,758	246	1,254
Aug-94	0.675	357	2,009	218	1,225	397	2,233	273	1,531
Sep-94	0.707	337	1,969	195	1,140	374	2,188	244	1,425
Oct-94	0.579	339	1,613	199	944	377	1,792	248	1,180
Nov-94	0.606	304	1,521	178	888	337	1,690	223	1,110
Dec-94	0.669	296	1,696	189	1,086	329	1,885	236	1,357
Jan-95	0.911	266	1,851	167	1,156	296	2,057	209	1,445
Feb-95	0.911	202	1,514	137	1,053	224	1,682	171	1,316
Mar-95	1.007	260	2,188	157	1,319	289	2,431	196	1,649
Apr-95	0.878	256	1,820	159	1,151	285	2,022	199	1,439
May-95	0.787	274	1,749	162	1,015	305	1,944	203	1,268
Jun-95	0.725	212	1,281	188	1,121	236	1,423	236	1,402
Jul-95	0.656	310	1,731	198	1,104	344	1,923	247	1,380
Aug-95	0.676	307	1,697	211	1,167	341	1,886	263	1,459
Sep-95	0.651	319	1,734	215	1,189	354	1,927	269	1,486
Oct-95	0.581	269	1,318	197	966	299	1,464	246	1,208
Nov-95	0.636	244	1,294	189	1,007	271	1,438	236	1,258
Dec-95	0.855	213	1,557	148	1,119	237	1,730	185	1,399
Jan-96	0.985	254	2,040	144	1,176	283	2,266	180	1,470
Feb-96	1.314	172	1,854	99	1,091	191	2,060	124	1,364
Mar-96	1.059	237	2,162	148	1,356	263	2,402	185	1,695
Apr-96	0.900	194	1,475	166	1,229	215	1,639	207	1,537
May-96	0.800	213	1,415	180	1,191	237	1,572	225	1,488
Jun-96	0.749	234	1,484	192	1,232	260	1,649	240	1,540
Max	1.314	372	2,188	253	1,356	414	2,431	316	1,695
Min	0.579	172	1,279	99	865	191	1,422	124	1,082
Avg	0.733	286	1,695	181	1,078	318	1,883	227	1,347
Winter avg	0.785	269	1,689	172	1,086	299	1,876	215	1,357
Winter max	1.314	372	2,188	253	1,356	414	2,431	316	1,695
Winter min	0.601	172	1,279	99	865	191	1,422	124	1,082
Summer avg	0.675	306	1,702	191	1,069	340	1,891	239	1,336
Summer max	0.800	363	2,030	218	1,232	403	2,255	273	1,540
Summer min	0.579	212	1,281	162	893	236	1,423	203	1,116

Notes:

1. "Screened" refers to actual reported values. Sampling occurs downstream of screens and grit tanks.
2. Raw sewage values were estimated using 20 percent TSS removal and 10 percent BOD removal across screens and grit tanks.

The highest monthly BOD load of 2,431 ppd occurred in March 1995. The maximum month BOD load will be assumed as 2,500 ppd. The highest monthly TSS load was 1,695 ppd; 1,700 ppd will be assumed as the maximum month TSS load.

Peak Plant Loading

Weekly BOD and TSS loads are shown in Figures 4-6 and 4-7, respectively. The highest weekly BOD load of about 3,600 ppd appears to be an outlying point. The peak week BOD load will be assumed as 3,000 ppd. The highest weekly TSS load reported since January 1993 (about 2,700 ppd) also appears to be an anomaly. The peak week TSS load will be assumed as 2,000 ppd.

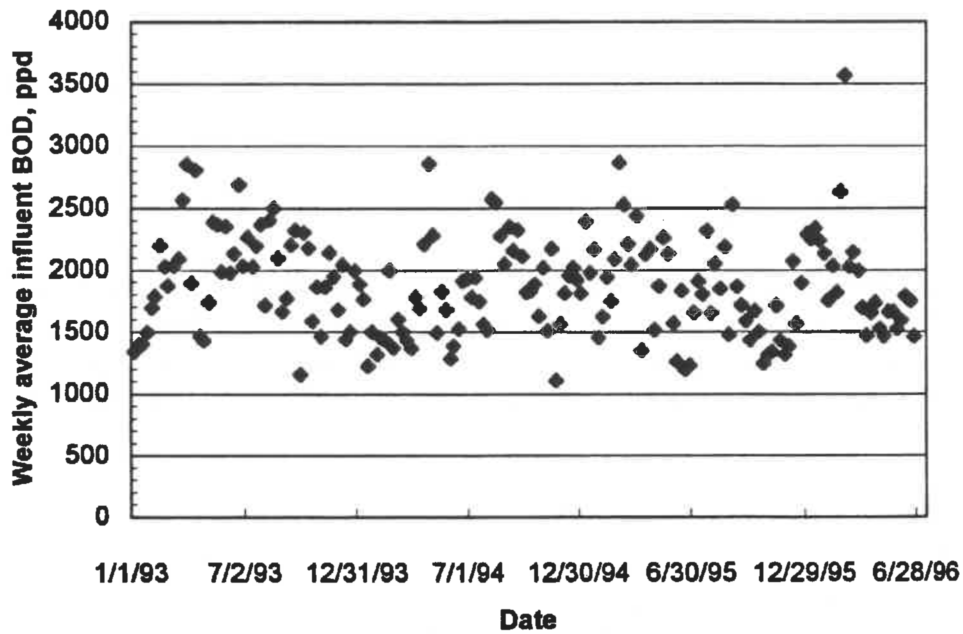

Figure 4-6. Weekly Average Influent BOD

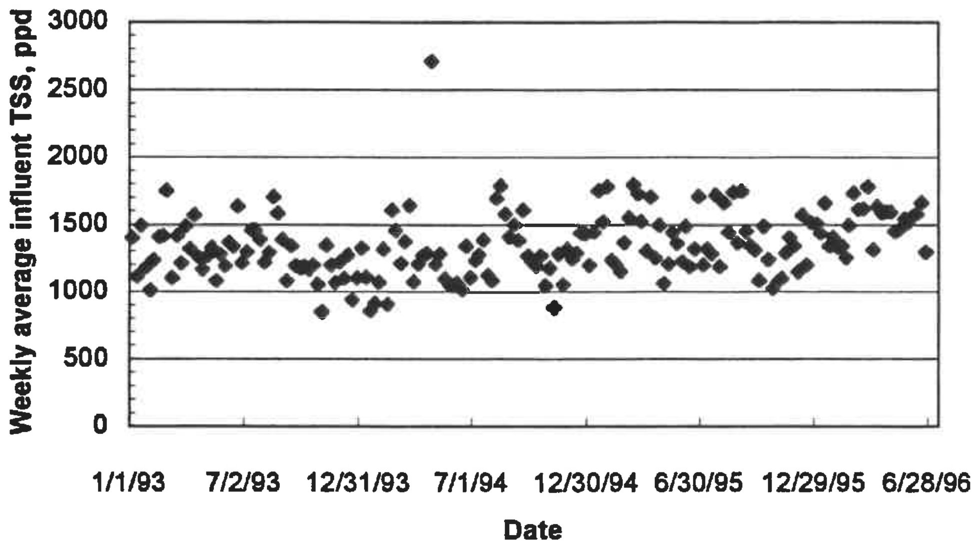

Figure 4-7. Weekly Average Influent TSS

The daily BOD loads are presented in Figure 4-8. There were several days during which the BOD load exceeded 3,500 ppd and one where 4,000 ppd was exceeded. The peak day BOD load is estimated as 4,000 ppd. The highest daily TSS load was about 4,400 ppd (Figure 4-9); however, this is far higher than any other value and probably not representative. The peak day TSS load is assumed as 2,500 ppd.

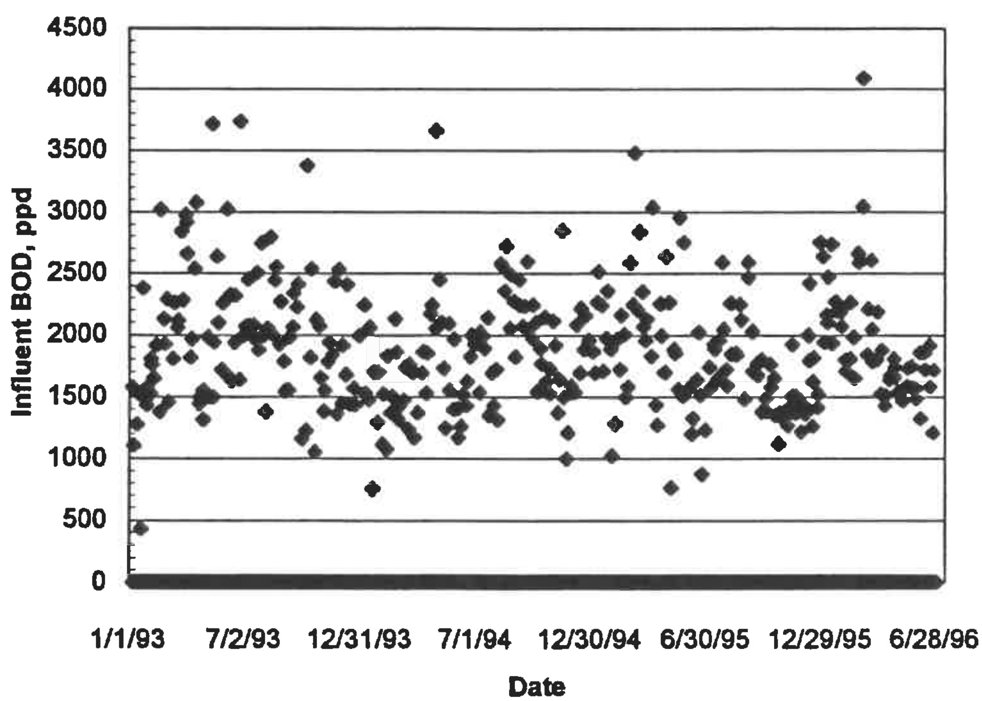

Figure 4-8. Daily Influent BOD

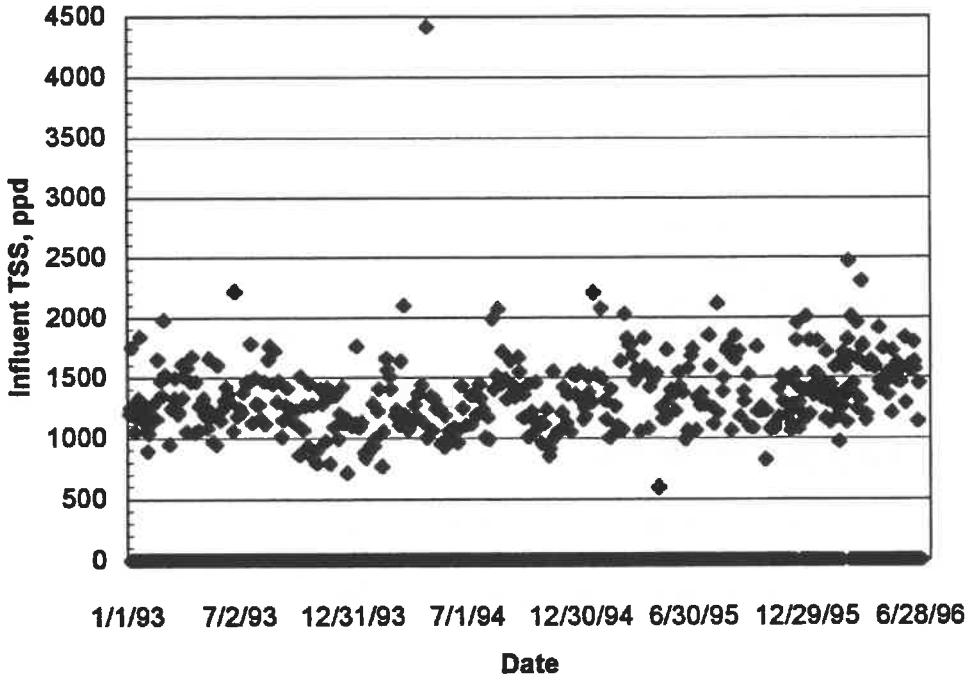

Figure 4-9. Daily Influent TSS

Estimated current plant flows and loads are summarized in Table 4-7.

Table 4-7. Current Flows and Loads

Item	Value
Flows	
ADWF, mgd	0.68
Average annual flow, mgd	0.73
AWWF, mgd	0.78
MMDWF, mgd	0.93
MMWWF, mgd	1.3
Peak week flow, mgd	1.6
Peak day flow, mgd	2
PWWF, mgd	3

Item	Value
Loads	
Average BOD load, ppd	1,900
Maximum month BOD load, ppd	2,500
Peak week BOD load, ppd	3,000
Peak day BOD load, ppd	4,000
Average TSS load, ppd	1,350
Maximum month TSS load, ppd	1,700
Peak week TSS load, ppd	2,000
Peak day TSS load, ppd	2,500

OTHER WASTEWATER CONSTITUENTS

Other components of concern in wastewater include ammonia, grease, and grit. Although the quantities of these components are not measured on a regular basis, an approximation can be made, based on typical textbook values and the occasional data collected at the plant.

Ammonia

Ammonia concentration in average domestic wastewater is typically about 25 milligrams per liter (mg/L).[1] Ammonia concentrations generally don't exceed 50 mg/L. Data collected at the plant indicate an average of about 30 mg/L, within the expected range.

Grease

Florence has a relatively large number of restaurants per capita as a result of the tourism in the area. A city ordinance requires restaurants to have grease traps; however, some grease continues to enter the wastewater system. When the sewers are cleaned, accumulations of grease are removed from the pipelines in some areas. Although notable quantities of grease enter the treatment plant, the amount is not excessive to the point of requiring special treatment.

Grit

Grit quantities in domestic wastewater normally range from 0.5 to 25 cubic feet per million gallons of flow.[1] A typical value is about 2 cubic feet per million gallons. The amount of grit removed from the Florence plant is about 1 cubic foot per million gallons. However, as discussed in Chapter 3, much of the settled grit probably gets washed back into the aeration basin. Because the city is located in a coastal area with sandy beaches and dunes, grit quantities in the upper portion of the normal range would be expected. New grit removal facilities should be designed accordingly.

FLOW AND LOAD PROJECTIONS

To develop flow and load projections, unit design values (the projected capacity) are established based on current flows and loads and current population. These values are then used in conjunction with the future, "design" population to develop flow and load projections. To develop projections for peak flows, infiltration and inflow (I/I) must also be considered.

UNIT DESIGN VALUES

The unit design values for the flows and loads are based on the current flows and loads as determined previously in this chapter and the estimated 1996 service area population of 6,401. The wet weather flows developed in Tech Memo 2.1 have since been revised based on more detailed flow information from the plant staff. The revised derivation of the current flow rates will be included in the final facilities plan. The unit design values are presented in Table 4-8.

The unit value of 106 gallons per capita per day (gcd) for ADWF is at the upper end of the typical range expected for wastewater flow rates. As discussed earlier in this chapter, the BOD loading is substantially higher than typical; whereas the suspended solid loading is within the typical range.

Table 4-8. Unit Design Values

Item	Value
Wastewater flow	
ADWF, gcd	106
Average annual flow, gcd	127
AWWF, gcd	147
Wastewater composition	
BOD	
Average, pcd	0.30
Peak month, pcd	0.39
Peak week, pcd	0.47
Peak day, pcd	0.63
Suspended solids	
Average, pcd	0.21
Peak month, pcd	0.27
Peak week, pcd	0.31
Peak day, pcd	0.39

PROJECTED WASTEWATER FLOW

As expected from the breakdown of land use types presented in Chapter 2, wastewater comes primarily from residential sources, commercial sources, and schools. It is expected that the commercial sources and schools will grow at approximately the same rate as the overall population. Therefore, the projections for the three sources can be combined into one projection based on population growth. Dry weather I/I is typically a small fraction of the ADWF. Night time observations of portions of the collection system indicate nearly zero flow, confirming that dry weather I/I in Florence is small. Consequently, dry weather I/I is not separated from the sanitary flow in developing flow projections. The unit design value is simply applied to the

ADWF. Applying the unit design value of 106 gcd to the design population of 17,937 yields a projected ADWF of 1.9 mgd. The projected average annual and average wet weather flows are determined in a similar manner. These flows are summarized in Table 4-9.

Table 4-9. Flow Projections

Item	Current value[a]	Design value
ADWF, mgd	0.7	1.9
Average annual flow, mgd	0.8	2.2
AWWF, mgd	0.9	2.6
MMDWF, mgd	1.0	2.5[b]
MMWWF, mgd	1.6	3.6[c]
Peak week flow, mgd	2.0	4.3[c]
Peak day flow, mgd	2.5	5.2
Peak wet weather flow, mgd	3.6	6.9[c]

[a] Current values are revised from flows developed in TM 2.1.
[b] Ratio of increase assumed as average of ratios for ADWF and MMWWF increases.
[c] From Figure 4-6.

Because peak flows contain a significant I/I component, an estimate of future I/I is necessary to determine future peak flows. It is generally recognized that newly constructed sewers contribute less I/I than older sewers. Part of this difference can be attributed to improved construction techniques and materials used for new sewers, and part can be attributed to deterioration of sewers, service connections, and manholes. To ascertain the difference between the I/I contributions of new and old sewers, the flows from relatively new basins were compared to the flows from basins about 20 years old. The flow comparisons were achieved by evaluating pump station run times for the basins to be evaluated.

The pump station run times for four pump stations are summarized in Table 4-10. The Siuslaw Village and 40th Street pump stations are older, representing older sewers. The Sea Watch and 42nd Street pump stations are newer, representing new sewers. As expected, I/I in the new basins was lower.. The maximum peaking factor observed in the new basins was 1.3 (peak day to ADWF). This is assumed as the peaking factor for new sewers. For the older basins, a maximum peaking factor of 3.0 was observed. This is used as the peaking factor for older sewers.

Table 4-10. Comparison of Pump Station Run Times in Old and New Basins

Item	Pump station			
	Siuslaw Village	40th Street	Sea Watch	42nd Street
Age, years	21	19	7	6
Run time for date shown, hours				
August, avg daily (ADWF)	1.51	3.11	1.97	3.81
2/6/96	2.12	4.2	2.0	4.9
2/7/96	2.13	4.2	1.4	4.8
2/8/96	4.57	4.7	1.9	5.0
12/25/96	1.45	4.4	1.5	4.2
12/30/96	1.85	4.0	1.2	4.2
12/31/96	1.88	4.3	1.5	4.7
Peak day/ADWF[a]	3.03	1.5	1.0	1.3

Note: [a]Calculated from the maximum observed daily run time (shaded box) and the August average.

In the design year 2020, the additions to the collection system will be a combination of sewers constructed now through the design year. Assuming a linear growth rate of the collection system and a linear increase in I/I with increase in sewer age, the overall peaking factor for sewers added during the next 20 years will be the average of the peaking factors for new basins and old basins. This results in an overall peaking factor of 2.2 (peak day to ADWF) for sewers constructed during the design period. To determine the increase in the peak day flow between the present and the design year, the peaking factor is multiplied by the increase in ADWF over the design period. From Table 4-9, the increase in ADWF is 1.22 mgd. Therefore, the increase in peak day flow is 2.7 mgd. This increase is added to the current value of 2.5 mgd, resulting in a design peak day flow of 5.2 mgd.

To determine the other flows, a probability of exceedance curve is developed from the average and peak day flows, as shown in Figure 4-10. This technique is similar to that used in previously in this chapter in developing the current peak flows. Peak month, week, and hour flows are determined based on the fraction of the year that these periods represent. Generally, these points fall in a straight line assuming no limitations to the collection system. Assuming a straight line relationship in this case results in the points indicated on Figure 4-10. These flows are summarized in Table 4-9.

WASTEWATER LOADS

Wastewater load projections are developed by applying the unit design values to the design population. Unlike peak flows, all loads are assumed to increase in proportion to population. The design loads are presented in Table 4-11.

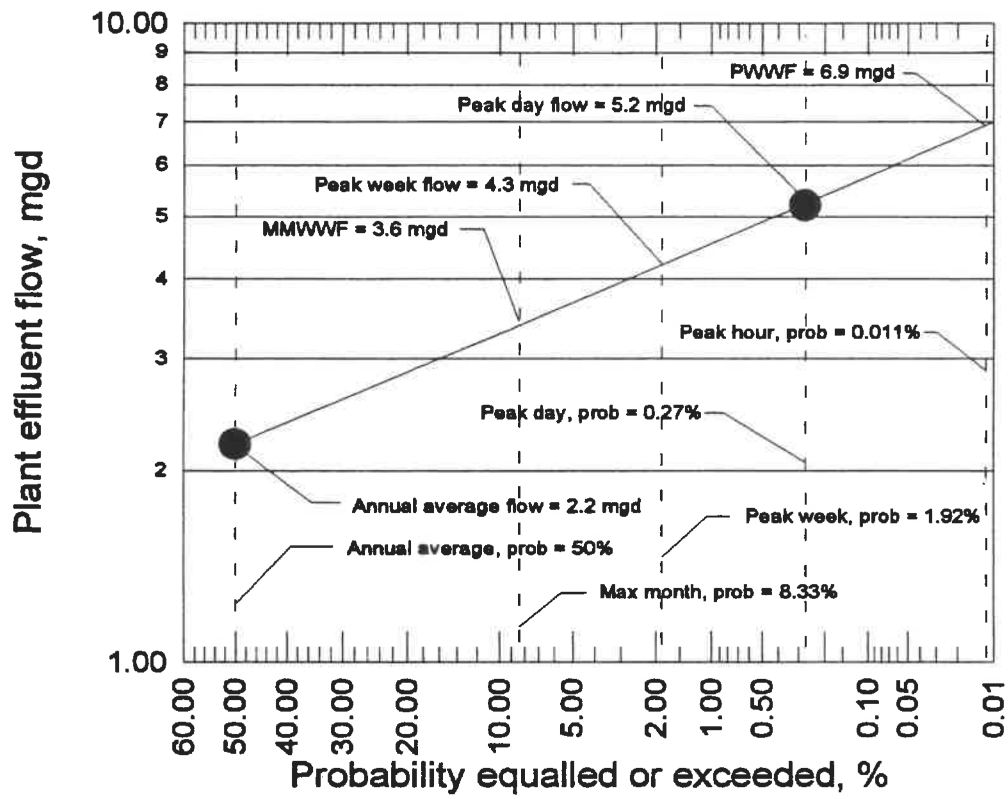

Figure 4-10. Peak Flow Projections

Table 4-11. Load Projections

Item	Current value	Design value
BOD		
Average, ppd	1,900	5,300
Maximum month, ppd	2,500	7,000
Peak week, ppd	3,000	8,400
Peak day, ppd	4,000	11,200
Suspended solids		
Average, ppd	1,350	3,800
Maximum month, ppd	1,700	4,800
Peak week, ppd	2,000	5,600
Peak day, ppd	2,500	7,000

[1] Metcalf & Eddy, Inc. **Wastewater Engineering: Treatment, Disposal, Reuse, Second Edition.** 1979.

CHAPTER 5

REGULATORY REQUIREMENTS

The City of Florence recognizes that the Siuslaw River represents a vital asset for the community. Protecting this resource has always been one of the city's priorities. The river's fisheries, wildlife habitat, and scenic and recreational opportunities are closely tied to water quality. This chapter presents the standards for protecting water quality and how they affect wastewater treatment requirements.

REGULATORY AUTHORITY

Standards for protection of water quality are set forth by the Environmental Protection Agency (EPA) and administrated by the Oregon Department of Environmental Quality (DEQ) through Chapter 340 of the Oregon Administrative Rules (OAR). The general policy followed in these rules is one of antidegradation of surface waters. Discharges from wastewater treatment plants are regulated through the National Pollutant Discharge Elimination System (NPDES). The criteria in the NPDES permit are based on existing water quality in the receiving stream, beneficial uses, size of discharge, and other factors.

DISCHARGE CRITERIA

Numerous factors must be considered in developing treatment limits for a wastewater treatment plant (WWTP). Prior discharge permits serve as a starting point in determining future requirements. Water quality regulations must be observed. The water quality of the receiving stream must be considered to ensure that water quality standards are not violated and beneficial uses are not impaired. This section examines the regulatory issues related to discharge of Florence WWTP effluent to the Siuslaw River Estuary.

CURRENT DISCHARGE REQUIREMENTS

Current treatment requirements are described in the NPDES permit and the recently signed Mutual Agreement and Order (MAO). This section examines these two documents.

Existing Discharge Permit

The Florence WWTP's NPDES permit limits are summarized in Table 5-1. Refer to Appendix B for a copy of the entire permit. The permit limits are consistent with those for a mechanical secondary treatment plant. Mass discharge limits for biochemical oxygen demand (BOD) and total suspended solids (TSS) are based on a design average dry weather flow (ADWF) of 0.75 million gallons per day (mgd). The permit expired in July 1997.

Table 5-1. Existing Permit Limits

Parameter	Average effluent concentrations, mg/L		Mass discharges, pounds per day		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
May 1—Oct 31					
BOD	20	30	125	188	250
TSS	20	30	125	188	250
fecal coliform per 100 mL	200	400			
Nov 1—Apr 30					
BOD	30	45	188	281	376
TSS	30	45	188	281	376
fecal coliform per 100 mL	200	400			

Other parameters (year-round)	Limitations
pH	6.0—9.0
BOD and TSS removal efficiency	85 percent, monthly average

Mutual Agreement and Order

An MAO is a legal agreement entered into by the city and DEQ. Refer to Appendix B for a copy of the entire MAO. The purpose of an MAO is threefold:

- Ensure environmental protection.
- Require the city to operate the existing WWTP to the best of their ability.
- Resolve the noncompliant status by setting achievable milestones. Penalties for noncompliance with the basic permit may be waived during this period.

The City of Florence signed their MAO in April 1996. In doing so, the city has agreed to adhere to the following schedule:

- Within 30 days after signing the MAO, a sign must be posted at the Ivy Street pump station stating that raw sewage bypasses occasionally occur.
- A draft notification and response plan must be submitted to DEQ 90 days after signing the MAO.
- A consultant must be retained within 3 months after signing the MAO.
- A draft wastewater facilities plan must be submitted to DEQ 9 months after retaining a consultant.

- A final facilities plan must be submitted to DEQ 3 months after receiving comments on the draft facilities plan.
- A preliminary design report must be submitted to DEQ 6 months after receiving approval of the final facilities plan.
- Draft plans and specifications for WWTP and collection system improvements must be submitted to DEQ 6 months after receiving approval of the preliminary design report.
- Final plans and specifications must be submitted 3 months after receiving comments on the draft plans and specifications.
- A construction contract must be awarded 6 months after receiving approval of the final plans and specifications.
- The WWTP and collection system improvements must be completed 16 months after awarding the construction contract.
- The upgraded plant must be in compliance with the discharge permit 3 months after construction is completed.

The DEQ helped protect the city from fines by assigning temporary treatment limits that the WWTP can comply with. The city has agreed to pay stipulated penalties in the event of noncompliance with the terms of the MAO. The discharge limits in the MAO are similar to those in the existing NPDES permit with the following exceptions:

- The daily maximum mass discharge limits for BOD and TSS are suspended when plant flow exceeds 0.75 mgd.
- The BOD and TSS concentrations measured on days when flows exceed 0.75 mgd are not used in calculating the monthly and weekly concentrations.
- The BOD and TSS concentrations measured on days when flows exceed 0.75 mgd are not used in calculating the monthly percent removal efficiency.
- The mass discharges on days when flows exceed 0.75 mgd are not used in calculating the monthly and weekly mass discharges.
- The fecal coliform counts measured on days when the flow exceeds 0.75 mgd are not used on calculating the monthly or weekly geometric mean values.

FUTURE DISCHARGE REQUIREMENTS

Future discharge permits for the Florence WWTP will conform to the requirements of OAR Division 340-41. Specifically, the Florence WWTP must comply with the water quality standards and treatment requirements for discharge to estuarine waters. In addition, special limitations may be applied to the WWTP if the Siuslaw River is found to be water quality limited for certain parameters.

Siuslaw River Water Quality Limitations

As required by Section 303 (d) of the Clean Water Act, the DEQ recently published a list of all streams that do not comply with applicable water quality standards. These waterways are referred to as water quality limited. The Siuslaw River is listed as water quality limited for temperature during the summer months. Discussions with DEQ indicate that this temperature listing will not place limits on future discharges from the WWTP which are more restrictive than those listed in the OARs. However, Florence will be required to monitor the temperature of the effluent to help determine the necessity of the city's participation in the development of a temperature management plan for the Siuslaw River basin.

Discussions with DEQ indicate that the Siuslaw River could be listed as water quality limited for other parameters in the future. The water quality parameters of concern include:

- Dissolved oxygen during the summer. Some past excursions of water quality standards have been noted.
- Habitat modification. More data is needed to determine if stream channelization or alterations to riparian areas is a problem.
- Nutrients. More data is needed to evaluate nutrients.
- Sediment. More data is needed to evaluate sediment.

It is unclear at this time if the Siuslaw River violates the water quality standards for the above parameters.

Oregon Administrative Rules

Division 340-41 of the OARs contains the state's water quality standards. For the Florence WWTP, the OARs of most interest are:

- OAR 340-41-026. This section describes policies and guidelines applicable to all basins.
- OAR 340-41-120. This section addresses implementation issues applicable to all basins.
- OAR 340-41-245. Water quality standards specific to the Mid Coast Basin are listed in this section.

These sections were last updated in January 1996.

Water Quality Parameters

This section discusses the standards for the water quality parameters critical to wastewater facilities planning for Florence.

- Temperature. As mentioned previously, the Siuslaw River is listed as water quality limited for temperature during the summer. The temperature standards for estuaries are somewhat less restrictive than for most fresh water bodies. For marine and estuarine waters, no significant increase in temperature above natural background levels is allowed (OAR 340-41-245 2. b. D.). Current DEQ policy considers no measurable increase to be a maximum increase of 0.25 degrees F at the edge of the mixing zone.

- Mass discharge limits. The general policy of the EQC is to maintain mass discharge limits for BOD and TSS at current levels; higher wastewater flows and loads associated with growth are to be accommodated with increased treatment efficiency (OAR 340-41-026 2). However, there are exceptions to this policy. In order to qualify for increased mass discharge limits, the city must demonstrate that the higher BOD and TSS loads would not cause water quality standards to be violated and that none of the river's beneficial uses would be impaired (OAR 340-41-026 3.). An analysis of the effect of WWTP discharges on Siuslaw River water quality is needed and will be provided after the Siuslaw River water quality analysis is completed.
- Dissolved oxygen. As with temperature, the dissolved oxygen (DO) requirement for estuarine waters is relaxed compared to most fresh waters in the state. The DO concentration in estuaries must be maintained above 6.5 mg/L (OAR 340-41-245 2. a. G.). The DO standards are summarized in Table 5-2. If the Siuslaw River is listed as a water quality limited stream in the future, new, more restrictive DO limits will probably be set.
- Intergravel dissolved oxygen. In an effort to improve salmonid spawning habitat, DEQ recently developed standards for intergravel dissolved oxygen (IGDO). Because the Florence WWTP discharges into a segment of the Siuslaw River where salmonid spawning does not occur, the IGDO limits should not apply to Florence wastewater facilities.
- pH. The pH for all fresh and estuarine waters must remain between 6.5 and 8.5 (OAR 340-41-245 2. d.).
- Bacteria. Because the WWTP discharges into shellfish-growing estuarine waters, the bacteria standards are relatively stringent. The median fecal coliform concentration cannot exceed 14 organisms per 100 milliliters (mL). In addition, no more than 10 percent of the samples can have more than 43 organisms per 100 mL (OAR 340-41-245 2. e.).
- Toxic substances. DEQ's limits on discharge of toxic substances (OAR 340-41-245 2. p.) are based on the EPA document *Quality Criteria for Water* (1986). For a WWTP that does not treat significant amounts of industrial waste, the toxic substances of greatest concern are typically chlorine and ammonia. Chlorine toxicity is a problem for plants that use chlorine as a disinfectant. In most cases, dechlorination or an alternative form of disinfection, such as ultraviolet light, is needed to comply with the saltwater limits of 0.0075 milligrams per liter (mg/L) for chronic toxicity and 0.013 mg/L for acute toxicity. Determining ammonia toxicity is more complex as it is dependent on water temperature, pH, and salinity. Ammonia toxicity can be addressed by converting the ammonia to nitrate in the secondary process through nitrification, by providing adequate mixing of plant effluent and the receiving water, or through a combination of both.

Table 5-2. Dissolved Oxygen and Intergravel Dissolved Oxygen Criteria

Class	Concentration and Period[a] (All Units Are mg/L)				Use/Level of Protection
	30-D	7-D	7-Mi	Min	
Salmonid Spawning		11.0[b,e]		9.0[c] 8.0[d] 6.0[e]	Principal use of salmonid spawning and incubation of embryos until emergence from the gravels. Low risk of impairment to cold-water aquatic life, other native fish and invertebrates. The IGDO criteria represents an acute threshold for survival based on field studies.
Cold Water	8.0[f]		6.5	6.0	Principally cold-water aquatic life. Salmon, trout, cold-water invertebrates, and other native cold-water species exist throughout all or most of the year. Juvenile anadromous salmonids may rear throughout the year. No measurable risk level for these communities.
Cool Water	6.5		5.0	4.0	Mixed native cool-water aquatic life, such as sculpins, smelt, and lampreys. Waterbodies includes estuaries. Salmonids and other cold-water biota may be present during part or all of the year but do not form a dominant component of the community structure. No measurable risk to cool-water species, slight risk to cold-water species present.
Warm Water	5.5			4.0	Waterbodies whose aquatic life beneficial uses are characterized by introduced, or native, warm-water species.
No Risk	No Change from Background				The only DO criterion that provides no additional risk is "no change from background." Waterbodies accorded this level of protection include marine waters and waters in wilderness areas.

[a]*30-D = 30-day mean minimum as defined in OAR 340-41-006.*
7-D = 7-day mean minimum as defined in OAR 340-41-006.
7-Mi = 7-day minimum mean as defined in OAR 340-41-006.
Min = Absolute minimum for surface samples when applying the averaging period, spatial median of IGDO.

[b]*When Intergravel DO levels are 8.0 mg/L or greater, DO levels may be as low as 9.0 mg/L, without triggering a violation.*

[c]*If conditions of barometric pressure, altitude, and temperature preclude achievement of the footnoted criteria, then 95 percent saturation applies.*

[d]*Intergravel DO action level, spatial median minimum.*

[e]*Intergravel DO criterion, spatial median minimum.*

[f]*If conditions of barometric pressure, altitude and temperature preclude achievement of 8.0 mg/L, then 90 percent saturation applies.*

Note:

***Shaded** values present the absolute minimum criteria, unless the Department believes adequate data exists to apply the multiple criteria and associated periods.*

- Mixing zone. The standard for effluent mixing generally limits the size of the mixing zone to half the width of the receiving stream. In sizing the mixing zone, factors such as effluent toxicity and available dilution are considered. The area within the limits of the assigned mixing zone shall be free of materials in concentrations that cause acute toxicity to aquatic life. The area outside the mixing zone cannot have substances in concentrations that cause chronic toxicity. The DEQ can also establish a small area within the mixing zone where acute toxicity is allowed. This area is known as the zone of immediate dilution (ZID). Without a ZID, the plant effluent must comply with acute toxicity standards at the end of the outfall pipe.
- Nutrients. The presence of chlorophyll-a is used as an indicator of excessive nutrient concentrations. For estuaries, the chlorophyll-a limit is 0.015 mg/L (OAR 340-41-150).
- Other parameters. OAR 340-41-245 also contains standards restricting liberation of dissolved gases, growth of fungi, creation of deleterious tastes and odors, formation of bottom deposits, formation of scum and oily sleeks, creation of offensive aesthetic conditions, discharge of radioisotopes, discharge of effluent with excessive dissolved gas concentrations, and creation of excessive total dissolved solids concentrations. None of these standards should significantly affect the design or operation of wastewater treatment facilities in Florence.

Mass Discharge Limits

Future mass discharge limits for BOD and TSS will be dictated by Siuslaw River water quality and the requirements of the OARs. Increases in mass discharge limits could be granted by DEQ if studies show that the river can assimilate higher waste loads without impairing water quality or beneficial uses. In the unlikely event that the service population exceeds 10,000 when the plant is started up, the plant would be considered a major facility, and the Environmental Quality Commission (EQC) would have the authority to evaluate the discharge limits. The effluent flow from the WWTP is a small fraction of the total Siuslaw River flow; therefore, it is unlikely that the plant has a significant effect on water quality. Discussions with DEQ indicate that a mass discharge limit increase for the Florence WWTP is possible.

Historically, mass discharge limits have been calculated based on a WWTP's design flow and the assigned concentration limits (OAR 340-41-120 9). For the dry weather season (May through October), the design ADWF is usually used to calculate monthly mass discharge limits. Weekly and daily limits are calculated as 1.5 and 2 times the monthly limit, respectively. Wet weather mass discharge limits are calculated in a similar manner, except that the design average wet weather flow (AWWF) is used. If the water quality analysis shows that higher loads to the river do not affect water quality, future mass discharge requirements would probably be calculated in this way. Table 5-3 presents potential future permit limits assuming that the increased loads are shown to have no effect on Siuslaw River water quality.

Table 5-3. Potential Future Permit Limits Based on Future Design Flows

Parameter	Average effluent concentrations, mg/L		Mass discharges, pounds per day		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
May 1—Oct 31					
BOD	20	30	334	500	667
TSS	20	30	334	500	667
Fecal coliform per 100 mL	14	--			
Nov 1—Apr 30					
BOD	30	45	575	863	1,150
TSS	30	45	575	863	1,150
Fecal coliform per 100 mL	14	--			

Other parameters (year-round)	Limitations
pH	6.0—9.0
BOD and TSS removal efficiency	85 percent, monthly average

May 1 - October 31 mass limits calculated using a design ADWF of 2 mgd

November 1 - April 30 mass limits calculated using a design AWWF of 2.3 mgd

If the water quality analysis demonstrates that the river's assimilative capacity has been reached, the current mass discharge limits would be retained. It is unlikely that DEQ would reduce the mass discharge limits below the levels in the existing permit unless the Siuslaw River is found to be water quality limited for DO and total maximum daily loads are assigned. Table 5-4 shows required effluent BOD and TSS concentrations at the estimated design flows if the current mass discharge limits are retained.

It is likely that Tables 5-3 and 5-4 define the limits of potential future mass discharge requirements. The results of the water quality analysis may show that future mass discharge limits should be set at some level between those shown in the two tables. In addition, DEQ may reduce the BOD and TSS concentration standard. Many plants are required to comply with a 10 mg/L concentration limit during the summer months. However, in most cases, these plants discharge to rivers with low summer flows, not to estuaries.

Table 5-4. Potential Future Permit Limits - Current Mass Discharge Limits Retained

Parameter	Average effluent concentrations, mg/L		Mass discharges, pounds per day		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
May 1—Oct 31					
BOD	7.5	11	125	188	250
TSS	7.5	11	125	188	250
Fecal coliform per 100 Ml	14	—			
Nov 1—Apr 30					
BOD	10	15	188	281	376
TSS	10	15	188	281	376
Fecal coliform per 100 mL	14	—			

Other parameters (year-round)	Limitations
pH	6.0—9.0
BOD and TSS removal efficiency	85 percent, monthly average
Mass discharge limits retained from existing permit.	

May 1 – October 31 required average concentration calculated based on an ADWF of 2 mgd.

November 1 – April 30 required average concentration calculated based on an AWWF of 2.3 mgd.

IMPACT OF PLANT DISCHARGE ON WATER QUALITY

As a result of population growth, the flows and loads to the plant are expected to increase by a factor of about 2.8 by the year 2020. Refer to Chapter 4 for the derivation and summarizing of these values. As a result of the increased flows and loads to the plant, the mass load in the effluent will increase. The effect of the increased mass discharge on the water quality is evaluated below.

LEVEL OF TREATMENT

The magnitude of the mass load increase is dependent on the pollutant concentration in the effluent, which is a function of the degree of treatment provided. In determining allowable mass discharges, the DEQ looks at expected plant performance during the maximum month, week, and day flows, on a two-year frequency of return basis. For the water quality evaluation, we have assumed that normal secondary treatment will be provided without effluent filtration.

The secondary treatment process can reasonably be expected to produce an effluent with BOD and suspended solids (SS) concentrations of 15 mg/L during the maximum month (2-year return) in the summertime. The expected effluent concentrations during the maximum weekly and daily flows are typically assumed to be 1.5 times and 2.0 times the monthly concentration, respectively. This results in a maximum weekly concentration of 23 mg/L and a maximum daily concentration of 30 mg/L.

In the wintertime, the maximum monthly effluent concentrations of BOD and SS are typically assumed to be 1.5 times those in the summer. The expected concentration during the maximum flow wintertime month is 23 mg/L. As with summertime flows, the weekly and daily maximum concentrations are expected to be 1.5 times and 2.0 times the monthly concentration, or 35 mg/L and 45 mg/L, respectively.

The expected effluent concentrations in the design year are summarized in the first column of values in Table 5-5 below.

Table 5-5. Expected Effluent Parameters From Upgraded Plant

Period	Concentration, mg/L	Flow, mgd	Mass load, ppd
Summer			
Monthly	15	2.3	288
Weekly	23	2.5	480
Daily	30	2.8	700
Winter			
Monthly	23	3.0	575
Weekly	35	3.4	990
Daily	45	4.0	1,500

EXPECTED MASS LOADING

The mass loading to the river is a function of the effluent pollutant concentrations and the concurrent flow rate. As discussed above, the allowable mass load is normally calculated based on maximum monthly, weekly, and daily flow rates on a 2-year return interval. These flow rates and the resulting mass discharges are summarized in Table 5-5.

EFFLUENT DILUTION

The amount of dilution affects the quantity of pollutants that can be assimilated by the river without degrading the water quality. A mixing zone study was recently completed to determine the level of dilution that could be expected for an assumed outfall configuration. The results of this study are summarized in more detail in Chapter 6. For a mixing zone encompassing a distance up to 210 feet from the outfall diffuser, the chronic toxicity dilution factor was determined to be 120:1. For a 21-foot ZID, the acute toxicity dilution factor was determined to be 30:1. The overall dilution in the estuary over a tidal cycle was estimated at a minimum of 930:1 during low streamflow periods.

EFFECT ON WATER QUALITY PARAMETERS

Oregon Administrative Rule 340-41-026 requires that before an increase in the allowable mass discharge can be granted, it must be shown that the increased discharge does not degrade the waters of the receiving stream. Specifically, the increased load may not cause water quality standards to be violated and must not impair any recognized beneficial uses. Because the Siuslaw is water quality limited with respect to temperature, the pollutant parameters associated with the increase must be unrelated to temperature. Each of these issues is addressed below.

Water Quality Standards

The basin standards for water quality parameters are summarized earlier in this chapter. An increased discharge must not cause any of these parameters to exceed the standards. The effect of the expected future discharge on each of these parameters is summarized below.

Dissolved Oxygen. The standard for DO in an estuary is 6.5 mg/L. Although DO data are lacking for the estuary, data upstream indicate that the ambient DO concentration is normally above 7.0 mg/L. Even if the plant effluent DO concentration were zero and BOD concentration were 20 mg/L, the DO depression at the edge of the mixing zone would be less than 0.1 mg/L.

Temperature. Temperature is the only parameter for which the river is water quality limited. Therefore, no measurable temperature increase in the river is acceptable. An increase of up to 0.25 degrees F is considered no measurable increase. For the temperature evaluation, an effluent temperature of 75 degrees F was assumed in the absence of temperature data at the Florence plant. This assumption is conservative. As an example, the maximum effluent temperature measured in the summer at the Bandon wastewater treatment plant is typically 68 degrees F. The ambient river temperature was assumed at 54 degrees F. This is the lowest mean observed at several sampling sites. Under these conditions, the temperature increase at the edge of the mixing zone would be 0.17 degrees F.

Turbidity. The maximum allowable increase in turbidity is 10 percent. Turbidity in the effluent is not a problem; it will probably be lower than the ambient turbidity in the river.

pH. The standard for pH 6.5 to 8.5. The plant effluent is expected to fall within this range. However, the current permit allows a range of 6 to 9. If the effluent pH were 9.0 and the ambient pH were at the 90th percentile level of 8.3, the resulting pH at the edge of the mixing zone would be 8.303, representing an insignificant increase.

Bacteria. For shellfish waters, the bacteria standard specifies a fecal coliform median of 14 cells per 100 mL, with no more than 10 percent of the samples at 43 cells per 100 mL. Assuming an effluent concentration of 200 cells per 100 mL, the increase at the edge of the mixing zone would be 1.7 cells per 100 mL. This increase is not significant.

Toxic Substances. The major toxic substances of concern are chlorine and ammonia. Chlorine will not be present in the effluent, as disinfection will be accomplished with ultraviolet light. Ammonia will be present, assuming that it is not nitrified in the aeration process. Determining ammonia toxicity is complex because it is dependent on pH, temperature, and salinity. However, reasonable ammonia permit limits can be set using the US EPA document, *Technical Support Document for Water Quality-based Toxics Control* (TSD, 1985). Using the dilution factors established above and conservative estimates for salinity, temperature, and pH, limits were calculated using a spreadsheet based on the TSD. These limits are 55 mg/L (as NH_3-N) for a daily maximum and 21 mg/L (as NH_3-N) for a monthly average. The effluent will be well under these limits. As discussed in Chapter 4, even the plant *influent* ammonia concentration should be below these levels.

Other toxic substances could also be present in wastewater effluent, depending on the characteristics of the incoming wastewater. A bioassay is generally recommended by DEQ to ascertain the toxicity of the effluent in helping to determine dilution requirements. A bioassay recently performed on treatment plant effluent indicated no acute or chronic toxicity. A summary of this study is included as Appendix E.

Beneficial Uses

The OAR states that the DEQ may rely on the presumption that if the water quality standards are met, then beneficial uses are protected. The beneficial uses in the Siuslaw include boating, fishing, and harvesting of clams and crabs. Maintaining the water quality standards will protect the beneficial uses.

DISCHARGE RECOMMENDATIONS

A mixing zone study has indicated that sufficient dilution can be achieved in the vicinity of the outfall to allow increased mass discharges without adversely affecting the water quality parameters. It is recommended that the mass limits summarized in Table 5-5 be incorporated into the revised discharge permit.

TREATMENT PLANT DESIGN CRITERIA

The design criteria that apply to sizing and selection of treatment units and equipment are presented below.

EQUIPMENT AND UNIT PROCESS RELIABILITY

The Florence wastewater treatment plant will fall into Reliability Class I or II as defined by the EPA, depending on the beneficial uses of the river in the area affected by the plant effluent. It is reported that some crab and fish are harvested in close proximity to the outfall, and that some clam beds are located within a few hundred feet. The most significant clam beds are reported to be more than a half a mile upstream. Because shellfish are harvested near the outfall, the plant may fall into Class I, depending on how the effluent plume is distributed in the river. Outfall modeling studies will be performed to determine distribution and dilution of the effluent plume. If the plant is not considered Class I, Class II requirements would apply. The requirements for the two reliability classes are summarized in Table 5-6.

Table 5-6. Treatment Plant Reliability Requirements

Component	Class I Requirements	Class II Requirements
Pumps	One backup pump. With one pump out of service, the remaining pumps can handle the peak flow.	Same as Class I.
Mechanically cleaned bar screen	One backup manually cleaned screen.	Same as Class I.
Primary sedimentation	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.[a]	Same as Class I.
Secondary clarifiers	With the largest unit out of service, the capacity of the remaining units to be at least 75 percent of the total design flow to the process.[a]	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.
Trickling filters	With the largest unit out of service, the capacity of the remaining units to be at least 75 percent of the total design flow to the process.[a]	With the largest unit out of service, the capacity of the remaining units to be at least 50 percent of the total design flow to the process.
Aeration basins	At least two equal-volume basins shall be provided.	Same as Class I.
Aeration equipment	With the largest unit out of service, the design oxygen transfer to be maintained.	Same as Class I.
Disinfection	With the largest basin out of service, the capacity of the remaining units to be at least 50 percent of the design flow to the process.[a]	Same as Class I.
Anaerobic digesters	At least two tanks to be provided.	Same as Class I.
Electric power[b]	A backup source required. Sufficient to operate all vital components during peak flow conditions, together with critical lighting and ventilation.	Similar to Class I, except secondary process components such as aeration need not be supported.

Notes: [a] For sedimentation, clarification, and disinfection, design flow is assumed to be peak wet weather flow (PWWF). For trickling filters, design flow is assumed to be peak month flow.

[b] Two separate, independent sources of power shall be provided whether Class I or II. Because only one substation feeds the vicinity of the treatment plant, an engine-generator set will be required.

As the table indicates, the requirements for the two classes differ only for trickling filters, secondary clarifiers, and backup power. Secondary clarifiers for a Class I plant would have a diameter about 20 percent greater than for a Class II plant.

PLANT FLEXIBILITY

Flexibility implies that portions of unit processes or entire unit processes can be bypassed with little effect on effluent quality. Operators should be able to take units out of service for maintenance without overloading the units remaining in service. Components should have provisions for isolating them from the flow stream for maintenance. The design should also account for the future addition of treatment units with minimum interference in plant operations.

SEISMIC CONDITIONS

Florence is located in Seismic Zone 3 as defined by the Uniform Building Code. All structures, piping, and equipment anchorage shall be designed to withstand the seismic forces as required by the code for Zone 3.

TSUNAMI PROTECTION

The Uniform Building Code specifies requirements for tsunami inundation zones. Restrictions are placed on critical facilities such as hospitals and emergency response facilities. The code specifies that "tanks and similar structures" may be constructed in a tsunami inundation zone. No other requirements are specified for such structures.

The treatment plant lies within the inundation zone as defined by the city planning department. The maximum wave, based on a 500-year recurrence interval, is estimated to rise to an elevation of about 16 feet above sea level in the vicinity of the plant. Grade at the existing plant ranges from about 10 to 12 feet in elevation. Design considerations include locating electrical equipment and the tops of tank walls several feet above the estimated maximum wave height.

SLUDGE MANAGEMENT

In order to apply sludge on land many requirements must be met to protect the health and safety of the public and to ensure that water bodies remain free of contamination. In general, higher quality biosolids (lower level of metals, pathogens, and vector attraction) have fewer restrictions in land application. These requirements are spelled out in the EPA Part 503 rule and in OAR Chapter 340, Division 50. The requirements and their impacts are summarized below.

PARAMETERS FOR CLASSIFYING SLUDGE

Classification of sludge is based on three independent parameters: metals concentration, presence of pathogens, and vector attraction level. The levels for the parameters are described below.

Metals Concentration

The *Ceiling Concentration Limits* are the maximum contaminant levels for all sludge applied to land. If these limits are exceeded for any one of ten metals, the sludge cannot be applied to land until tests show that the limits are no longer exceeded. High levels of metals indicate that industrial pretreatment may be necessary.

The *Pollutant Concentration Limits* are somewhat lower limits for metals concentration. Sludge with metals concentration within these limits can be land-applied without restrictions on metal accumulations in the soil. Tracking of metals is not required. Most domestic sewage sludges fall within these limits. For the metals that have been tested for Florence, the levels are within these limits.

Presence of Pathogens

The two categories for pathogen reduction are *Class A* and *Class B*. To meet Class A requirements, fecal coliform or *Salmonella* bacteria levels must meet specific density requirements at the time of use or disposal. Class A sludge contains minimal numbers of pathogens and is considered safe for public use. In addition to the bacteria requirement, the sludge must be treated by one of several alternatives. The alternatives include high temperature treatment, high pH treatment, or use of a Process to Further Reduce Pathogens (PFRP). PFRPs include composting, heat drying, irradiation, and pasteurization, among others.

To meet Class B requirements, the sludge must meet the requirements in one of two alternatives: testing for fecal coliform density at the time of use, and use of a Process to Significantly Reduce Pathogens (PSRP). Anaerobic digestion with a mean cell residence time of 15 days at 35 degrees C is considered a PSRP. The Florence digestion process currently meets this requirement. Class B sludge may contain significant numbers of pathogens, including coliform bacteria, salmonella, tapeworms, nematodes, cholera, amoebas, and virus. Consequently, restrictions apply to the use of Class B biosolids to prevent the transmission of disease.

Vector Attraction Levels

There are ten options to achieve vector attraction reduction. Anaerobic digestion, achieving a volatile solids reduction of 38 percent is an acceptable option. The Florence digestion process currently meets this requirement.

CATEGORIES OF SLUDGE

Sludge is classified into four categories, depending on the levels of metals and pathogens. The categories are described below. The terms used here for each category are not explicitly defined in the Part 503 rule, but are used in the EPA Guide to Part 503 Rule. The criteria for the categories and application restrictions are summarized in Table 5-7.

Table 5-7. Summary of Sludge Category Descriptions

Category	Meet Pollutant Concentration Limits	Pathogen Class	Site Restrictions[b]	Management Practices[c]	Track Metals
EQ	Yes	A	No	No	No
PC	Yes	B[a]	Yes	Yes	No
CPLR	No	B[a]	Yes	Yes	Yes
APLR	No	A	No	Label bags only	Yes

Notes: [a] Subcategories of PC and CPLR sludge exist for Class A sludge. Refer to Part 503 Rule for details.

[b] Site restrictions typically restrict harvesting for specified period after application, grazing after application, and public contact.

[c] Management practices address application on frozen, snow-covered, or flooded land, application near water bodies, and effect on threatened or endangered species.

Exceptional Quality (EQ)

Sludge in this category meets all the most stringent requirements: the Pollution Concentration Limit for metals, the Class A requirement for pathogens, and one of the vector attraction reduction requirements. This sludge can be applied to any land with very little restriction, similar to normal fertilizer.

Pollutant Concentration (PC)

This sludge meets the more stringent metals concentration limit (*Pollutant Concentration Limit*), but is not considered in Class A with respect to pathogens. Because PC sludge is categorized as Class B, land application is subject to site restrictions and management practices. The management practices take into account the life expectancy of the pathogens, which could be from 1 to 3 years, depending on the biosolids application procedure. Examples of the waiting periods required between application and contact or food harvesting are presented in Table 5-8. Because pathogens are present, this sludge cannot be bagged and given to the public. Digested sludge currently produced at the Florence plant would most likely be in this category.

Table 5-8. Examples of Management Practices for PC Biosolids

Description	Required waiting period
Food crops whose harvested parts touch soil	14 months
Food with harvested parts below ground where biosolids remain on surface at least 4 months	20 months
Food with harvested parts below ground with biosolids incorporated in less than 4 months	38 months
Animal grazing	30 days
Public access with high potential for exposure (for example, ballpark)	1 year

Cumulative Pollutant Loading Rate (CPLR)

This is the lowest quality sludge that can be applied to land. It does not meet the more stringent metals concentration limits but must meet the metals *Ceiling Concentration Limits*. It may meet either the Class A or B pathogen requirements. It must meet the vector attraction reduction requirements. Because of the higher metals concentrations, metals must be tracked during application, and the quantity of sludge applied to the land is limited by the accumulation of metals on the land. It is unlikely that digested sludge from Florence would fall in this category.

Annual Pollutant Loading Rate (APLR)

This sludge falls into Class A with respect to pathogens, but does not meet the more stringent metals concentration limits. Hence, the metals must be tracked and annual application is limited by the accumulation of metals on the land. It can be provided to the public in bags which are labeled to provide the application restriction information to the user.

IMPACTS OF SLUDGE APPLICATION RULES

Sludge from Florence will probably be classified as either EQ or PC because it will probably meet the more stringent metals concentration limits. The significant difference between these two categories is the pathogen requirement. To produce EQ sludge, an additional process, such as composting, would be required to meet the Class A pathogen requirement. The advantage of producing EQ sludge is that there are no site restrictions on its application. Because the city is currently limited in available site options, producing EQ sludge could be advantageous by providing more site options and disposal flexibility. If the city is unable to acquire additional sites within a reasonable distance of the plant, producing EQ sludge may prove cost effective.

OTHER REQUIREMENTS FOR SLUDGE MANAGEMENT

The OARs contain other requirements pertaining to land application of sludge. For most physical requirements of the sludge, the OARs reference the Part 503 Rule. However, the OARs impose certain regulatory, monitoring, and reporting requirements in addition to the requirements of Part 503.

A permit or license is required for any land application of sludge or preparation of sludge-derived products. To renew the permit, a sludge management plan must be submitted to the DEQ. The management plan must be kept current. Modifications require approval of the DEQ. The management plan must include a description of the treatment process, the quantities of sludge produced, a description of the sludge sampling and monitoring program and sludge analysis, and a description of application sites. The management plan also includes letters showing DEQ approval of all application sites.

EFFLUENT REUSE

Treatment requirements for reuse of WWTP effluent are detailed in OAR 340-55-015 and summarized in Table 5-9. Treating to Level II standards would be adequate for irrigating city-owned pasture land on which access is controlled. For essentially unrestricted irrigation of golf courses and city parks, treatment to Level IV standards is required. To comply with Level IV reclaimed water standards, the wastewater must receive biological treatment, coagulation, and filtration, and meet stringent turbidity and disinfection requirements. In contrast, Level II reclaimed water need only receive biological treatment and disinfection. Accordingly, the cost associated with producing Level IV reclaimed water are substantially higher than that for Level II treatment.

Table 5-9. Treatment and Monitoring Requirements for Use of Reclaimed Water

Category	Level I	Level II	Level III	Level IV
Biological treatment	X	X	X	X
Disinfection	N/R	X	X	X
Clarification	N/R	N/R	N/R	X
Coagulation	N/R	N/R	N/R	X
Filtration	N/R	N/R	N/R	X
Total coliform (organisms/100 mL):				
Two consecutive samples	N/L	240	N/L	N/L
7-Day median	N/L	23	2.2	2.2
Maximum	N/L	N/L	23	23
Sampling frequency	N/R	1 per week	3 per week	1 per day
Turbidity (NTU)				
24-Hour mean	N/L	N/L	N/L	2
5% of time during a 24-hour period	N/L	N/L	N/L	5
Sampling frequency	N/R	N/R	N/R	Hourly
Public access	Prevented (fences, gates, locks)	Controlled (signs, rural or nonpublic lands)	Controlled (signs, rural or nonpublic lands)	No direct public contact during irrigation cycle
Buffers for irrigation:	Surface: 10 ft. Spray: site specific	Surface: 10 ft. Spray: 70 ft.	10 ft.	None required

Note: N/L - no limit
N/R - not required

CHAPTER 6

DEVELOPMENT OF LIQUID STREAM TREATMENT ALTERNATIVES

In this chapter, liquid stream treatment alternatives for handling the future wastewater flows and loads are developed. The alternatives are developed from a wide range of processes considered initially, but screened out based on impracticality, inflexibility, or other flaws. Much of the screening is based on discussions with city staff and the Department of Environmental Quality (DEQ) during a brainstorming session held on March 12, 1997. The alternatives developed in this chapter are evaluated in Chapter 8, resulting in selection of preferred unit processes for the recommended plan. Alternatives for solids handling are developed in Chapter 7.

GENERAL ALTERNATIVES

Before any evaluation of system upgrade alternatives can be performed, it must be determined whether the existing plant should be upgraded or whether it is more beneficial to build a new plant elsewhere. The three general alternatives considered in this study are: No Action, New Plant at Alternate Location, and Expand Existing Plant. As discussed below, the recommendation of this study is to expand the existing plant.

NO ACTION

The city has entered into a Mutual Agreement and Order (MAO) with DEQ requiring that the city make improvements to the treatment plant. The MAO was established because the existing plant is overloaded to the point where frequent violations to the discharge permit occur. The city is subject to fines of $250 per day for each day of violation of the compliance schedule stipulated by the MAO. The MAO is described in more detail in Chapter 5. Choosing the No Action alternative is considered unacceptable because it represents an unacceptable level of risk to public health. This risk is the primary reason that DEQ is requiring the city to correct the problems through the MAO process.

NEW PLANT AT ALTERNATE LOCATION

Construction of a new plant at an alternate location has been discussed with city staff. Because most of the growth in the urban area is projected to take place northward, a more northerly location was considered. However, this alternative was screened out for several reasons. The current southern location is well suited to the topography in Florence. The land generally slopes gradually from north to south. A more northerly plant location would require pumping most of the city's wastewater from the south end of town back to the north. A new location would also require a new ocean outfall, construction of which would have significant environmental consequences. Furthermore, northward expansion of the city is limited by the presence of National Forest land to the north end of the study area.

EXPAND EXISTING PLANT

The existing site is large enough to easily accommodate the unit processes required to handle the flows and loads projected through the study period. There is adequate space for future expansions that may be required at least 20 years beyond the study period. Expansion of the existing plant can be accomplished without interrupting the treatment process. Furthermore, savings can be realized by continuing to use some of the solids handling unit processes at the existing plant. Therefore, future flows and loads will be accommodated by expanding the existing plant.

UNIT PROCESS ALTERNATIVES

In this section alternatives are presented and evaluated for each unit process of the liquid stream. In this preliminary evaluation, the less desirable alternatives are eliminated while one or two alternatives for each unit process are retained for further evaluation.

INFLUENT PUMPING

Currently all wastewater is pumped to the plant from other points in the collection system. However, upgrades to the collection system will include a new interceptor that will carry some of the system flow by gravity to the treatment plant. Refer to Chapter 9 for details on upgrades to the collection system. An influent pump station will be required to lift this flow to the elevation of the headworks.

The influent pump station would utilize self-priming pumps similar to those used in the majority of the pump stations in the collection system. Three pumps would be provided, one of which would be a standby. The pumps would be operated by variable speed drives. The wet well would be self-cleaning by virtue of its narrow width and sloped bottom. The wastewater velocity in the narrow wet well exceeds scouring velocity each day, flushing out any solids that may have settled. The small size also prevents scum accumulation by minimizing the wastewater surface area. Minimizing the accumulation of scum and solids reduces odor potential and maintenance requirements.

HEADWORKS

The headworks would include screening and compaction equipment for removal of large solids. A grit removal system would be provided directly downstream of the screens. The headworks would also include flumes for controlling water surface elevation through the screens and measuring plant flow. The design data for the proposed headworks are included in the design data tables for the overall treatment alternatives. Refer to Tables 6-1 and 6-2 presented later in this chapter.

Screening and Compacting

Screening equipment generally falls into two categories: fine-mesh screens and bar screens. Fine mesh screens typically have a mesh of less than one-quarter inch, while bar screens normally have openings of at least one-half inch.

Fine-mesh screens. These remove relatively large amounts of material and allow very few discernible solids to pass through. Fine screening provides excellent protection of downstream process by removing essentially all rags, plastics, and other solids that could cause plugging problems downstream. The resulting sludge is nearly free of recognizable debris, making it more acceptable to end users. The removal of most solids at the headworks is of greater advantage in a plant without primary sedimentation because the lack of sedimentation allows these solids to enter downstream processes where they can cause more problems.

The primary disadvantage of these screens is that they remove large amounts of putrescible material. This can cause odor problems at the headworks and can be cause for rejection by landfill operators. These problems can be overcome by providing a screen that includes screenings washing equipment that removes most of the fecal matter. Another potential disadvantage is the tendency for fine screens to blind and for fibers and hairs to wrap around the individual wires in the screening element. This problem can be overcome by providing a screen that is cleaned with a mechanical rake or brush rather than only spray water. Another alternative is the use of perforated plate rather than parallel wires for a screening element.

Bar Screens. Conventional bar screens remove only larger debris and rags; small items and hairs pass through. The advantage is that the screenings are relatively free of fecal matter and are less voluminous. Less hauling is required and unwashed screenings can be more acceptable to landfills. However, without primary treatment, the debris that passes through the screen can cause problems downstream. Equipment subject to plugging includes pumps, grit separators, and ultraviolet disinfection units. The bar screen mechanism has a high profile, normally extending more than 15 feet above the top of the influent channel. This can be of concern where aesthetics are important. It also adds to the cost of containment and odor control.

Recommended Screening Installation. An in-channel fine-mesh screen with some form of positive mechanical cleaning is recommended for the headworks. The major consideration in this decision is the fact that primary treatment will not be provided. Fine screens also provide the flexibility to incorporate the trickling filter process in this phase or in the future without the addition of primary sedimentation.

To prevent the screen from blinding, a positive mechanical cleaning device will be included. This may consist of a rake device or brushes integral with the screenings auger, depending on which manufacturer is selected. The screening equipment will be pivot-mounted. This allows the unit to be lifted out of the channel easily for maintenance. Screenings washing equipment should also be included. The washing equipment would reduce the amount of organics in the screenings and return them to the process stream for appropriate treatment. Washed screenings would be more acceptable to landfill operators. Washing would also reduce the quantity of screenings, reducing hauling costs, and reducing the load on decreasing landfill space. Disposal of screenings will be increasingly difficult in the future. By washing and compacting to minimize the volume of screenings, the city will have taken all steps possible to minimize the screenings disposal problem.

For current cost comparison purposes, it is assumed that two screens will be installed in parallel channels with a combined total capacity equal to the peak wet weather flows (PWWF). A third channel will serve as an emergency overflow. The overflow channel will have a hand-cleaned bar rack. Other configurations, such as a single unit with PWWF capacity, will be evaluated during predesign.

A screenings compactor will also be included. The compactor dewaters screenings up to 40 percent solids. The compacted screenings are more readily accepted by landfills and cost less to haul. A bagging device should be considered. Bagged compacted screenings are easy to handle and generate less odor.

Odor control will be provided for the screenings area. The in-channel screen with an integral compactor is self contained, thereby reducing the amount of odor released. Total enclosure of the screening area is not required; only the screenings storage area need be covered.

Grit Removal

Two types of grit removal systems were considered for the Florence plant: a true vortex system similar to the existing (Eutek Teacup) and an induced vortex tank, which relies on gravitational force to settle grit. Because two of the existing four Teacup units perform well and could be reused, utilizing this system was considered. However, because the Teacups require several feet of pressure head to operate, the screening channel would need to be mounted several feet above the Teacups, or at least 10 feet above grade. This would require an imposing headworks structure which would be costly and aesthetically displeasing. Alternatively, if the screens were situated at grade, raw sewage pumping would be required between the screens and grit system.

An induced vortex grit removal tank requires no driving head and can be situated at the elevation required for the plant hydraulics. Because the induced vortex system has a lower grit removal efficiency at higher flows, the unit should be oversized to ensure efficient grit removal at peak flows. Alternatively, two tanks could be provided with the second unit in service only during peak flow periods. The grit tank would require a gallery below grade to house the grit pump and other ancillary equipment.

The grit slurry pumped from the bottom of the grit tank is further concentrated in a cyclone separator. The underflow from the separator is dewatered in a screw classifier. The dewatered grit falls into a collection box with the screenings. Odor control will be provided for the classifier.

Collection System Cleanings

The headworks could also include a station for receiving material removed from the collection system during cleaning operations. This material consists primarily of grit and grease, with occasional large objects such as rocks and other debris. A bar rack would remove rocks and debris. The material would then flow into the headworks for fine screening and grit removal. Grease would pass through the treatment process to the secondary clarifiers where it would be removed with the scum and pumped to the digester. The station would include a high pressure hose station and a hot water spray to aid in cleaning.

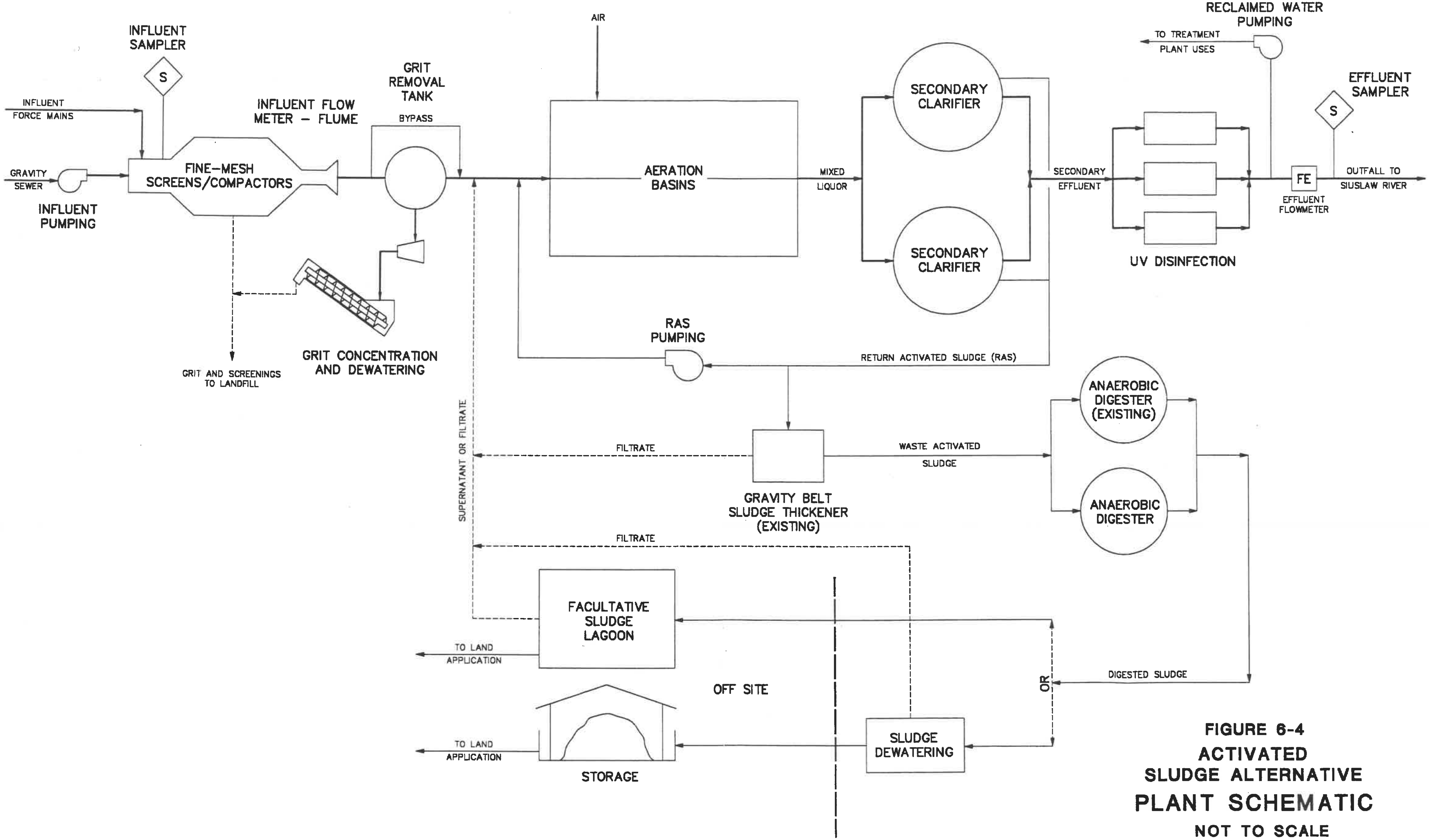

FIGURE 6-4

ACTIVATED SLUDGE ALTERNATIVE

PLANT SCHEMATIC

NOT TO SCALE

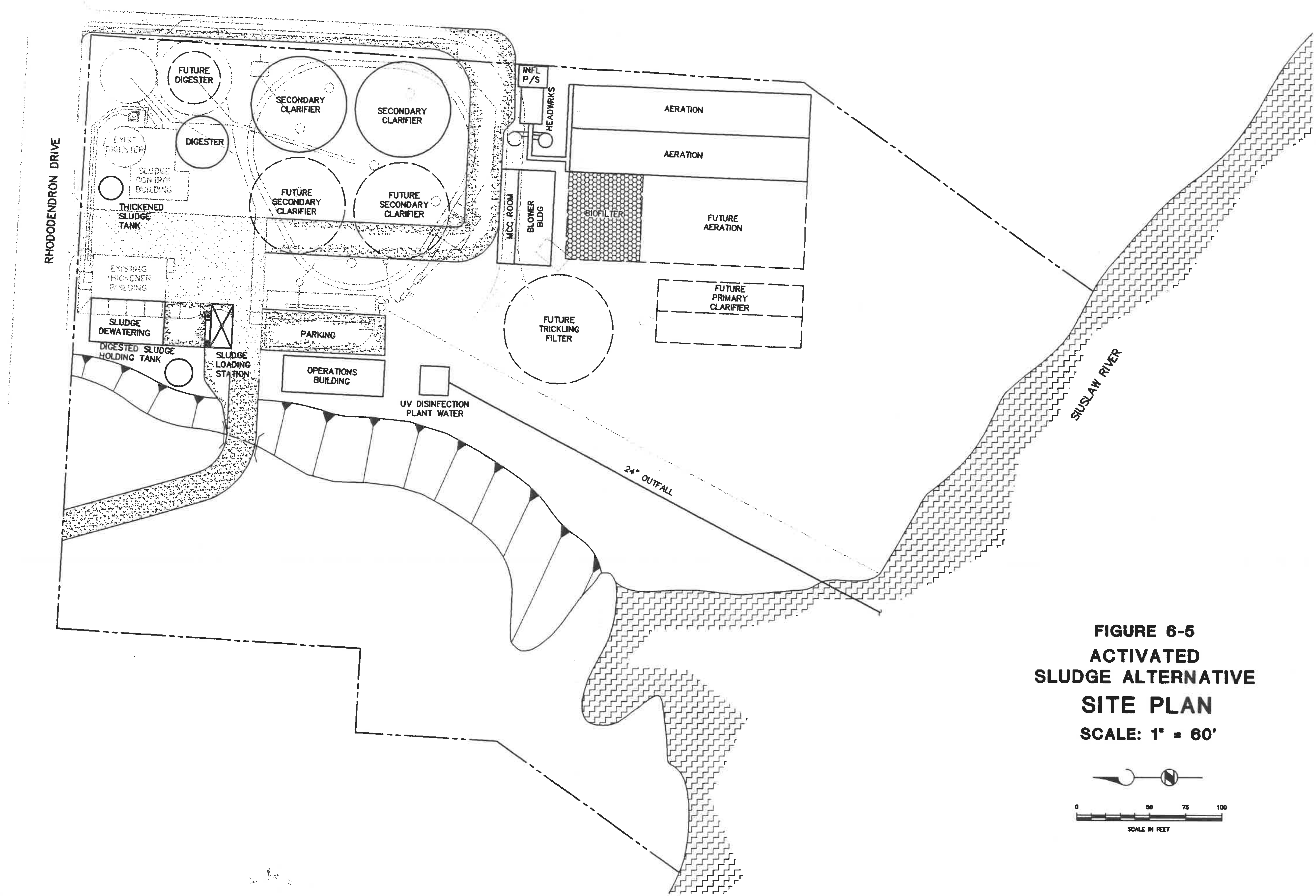

FIGURE 6-5
ACTIVATED SLUDGE ALTERNATIVE
SITE PLAN
SCALE: 1" = 60'

PRIMARY TREATMENT

Primary sedimentation basins generally remove about 70 percent of the incoming suspended solids and about 30 percent of the biochemical oxygen demand (BOD). The BOD removal reduces the load to the secondary process, resulting in smaller basins and a lower air requirement. The lower air requirement translates to less power consumption and annual cost savings. However, primary sedimentation represents an additional process at the plant with associated maintenance effort. The plant staff have noticed odor problems in the past when primary treatment was in use at the plant. Consequently, primary treatment will not be provided in this phase. Fine-mesh screens in the headworks provide a good removal of small solids that could cause problems in the absence of primary treatment. In developing site plan alternatives, space will be provided for the addition of primary sedimentation in a future expansion. Adding primary clarification in the future would allow the city to delay additional expansions to the secondary process.

SECONDARY TREATMENT

Secondary treatment is the heart of the wastewater treatment process. The design of the overall treatment plant is driven by the selection of the secondary treatment process. The biological treatment alternatives and secondary clarification are discussed below.

Biological Treatment

Three alternatives for biological treatment are discussed below: activated sludge, trickling filter/solids contact (TF/SC), and sequencing batch reactor (SBR). Detailed design data and preliminary layouts for the alternatives are presented in the summary section at the end of this chapter.

Activated Sludge. The conventional activated sludge process is the most common process used for secondary treatment of domestic wastewater. The existing system in Florence uses this process. Advantages include stability, a proven track record, simplicity of operation, flexibility, and operator familiarity with the process. The flexibility in the process applies to modes of operation and to alternatives for future expansion. Disadvantages include high energy consumption and operator attention required to change operating parameters or modes under changing flow and load conditions.

In this alternative, two new aeration basins would be provided with fine bubble diffuser aeration. The existing aeration basin would be eliminated, as it is too shallow for fine bubble aeration and could not be adapted to provide the desired modes of operation.

Modes of operation to be provided in this alternative include plug flow, step feed, contact stabilization, and anaerobic selector. Plug flow is the normal mode of operation in which raw sewage and return activated sludge (RAS) enter the head end of the basin and exit the downstream end. When flows are very high, the contact stabilization mode is used to prevent washout of solids. In this mode the first cell contains only RAS; raw sewage is fed to the basin further downstream. The high concentration of solids in the first cell enables the system to retain more solids despite high plant flows. Step feed is a combination of plug flow and contact stabilization. It is used during moderately high flows or as a transition between plug flow and contact stabilization. The anaerobic selector is an unaerated, mixed cell at the head end of the basin. The anaerobic selector is used during low flows in the summer to improve settling characteristics of the sludge.

As shown on Figure 6-5 in a later section of this chapter, the activated sludge process is flexible with respect to future expansion. When the system reaches capacity, it can be expanded by adding more basins incrementally. Alternatively, a trickling filter could be added, in which case, the aeration basins would serve as the solids contact portion of a TF/SC plant. This would provide substantial energy savings in the future. Another alternative would consist of adding primary sedimentation in the future, thereby increasing the effective secondary treatment capacity by about 30 percent. This alternative would provide capital and annual cost savings.

Trickling Filter/Solids Contact. In the TF/SC process, raw wastewater is pumped over the top of a trickling filter containing plastic media. The trickling filter effluent is collected at the bottom and flows into a small aeration basin. The aeration basin provides additional BOD removal and improves the settling characteristics of the sludge. In this alternative, a single trickling filter is provided with two parallel solids contact basins. The basins serve as backup aeration basins when the trickling filter is out of service.

The major advantages of the TF/SC process are its stability, ease of operation, and low energy consumption. Very little operator attention is required, regardless of changes in flow and load conditions. The trickling filter handles shock loads well, and washout of the process solids is difficult because much of the inventory is fixed to the media. The aeration requirements are about one-fourth that of the activated sludge process, resulting in energy savings. Another advantage is that the process produces sludge with better settling characteristics. Consequently, the secondary clarifiers can be smaller.

Disadvantages of the process include odor potential, lack of flexibility in unit process sizing, and growth of snails in the filter. Because raw wastewater is sprayed on the top of the filter odor can be released. The high and exposed location of the spray could result in the odor migrating off site. Because of the importance of odor control at this plant, a trickling filter should be covered and provided with odor control. At many trickling filter installations, large quantities of snails grow in the filter and eventually slough off, entering downstream processes. However, this problem can be addressed by including a snail removal section at the upstream end of the solids contact tank. This section acts as an aerated grit tank, settling the snails into a hopper. The snails are then pumped to the grit removal system.

Because the trickling filter must be sized for the maximum load in the design year, it would be oversized for most loads experienced during the early part of the design period. This could result in less effective treatment at times. One solution to this problem is to provide only part of the media initially, adding the rest in the future as the load dictates.

Sequencing Batch Reactor. The SBR is a modified activated sludge process that treats the wastewater in batches. It has a long sludge age similar to extended aeration. Advantages include the lack of secondary clarifiers and a lower operations staff requirement because of automation. SBRs are well suited to nutrient removal because they include adjustable aerobic and anoxic cycles.

Disadvantages include the lack of flexibility in future expansion, reliance on the automatic control system, difficulties with scum removal, difficulties in handling flows with high peaking factors, and poorer effluent quality characteristic of extended aeration.

Secondary Clarification

Two new secondary clarifiers would be provided in either the activated sludge or the TF/SC alternative. No secondary clarifiers are necessary in the SBR alternative. The clarifiers in the TF/SC alternative are slightly smaller than for activated sludge because they can handle a higher overflow rate due to the superior settling characteristics of TF/SC sludge. The clarifiers are sized to handle 75 percent of the PWWF with one unit out of service in accordance with Class I reliability requirements. The clarifiers would have flocculating centerwells to improve the settling characteristics of the sludge. The effluent launders would be mounted peripherally. Stamford baffles will be provided under the trough to deflect upward sidewall currents that would otherwise tend to carry solids over the weir. The advantage to peripheral launders is the ease of access for cleaning.

For the activated sludge alternative, return sludge pumping would be accomplished by two pumps dedicated to each clarifier. The pumps would draw from a sump attached to the side of each clarifier. Submersible pumps or self-priming pumps have typically been used for return sludge pumping. The final selection of pump type will be made during predesign. The pumps would be provided with variable speed drives. The operator would have the option of pacing the pumping rate to plant flow rate or selecting a pumping rate directly. The capacity with both pumps operating simultaneously would be about 150 percent of the average dry weather flow (ADWF) to the clarifier.

Sludge would be wasted through a branch pipeline of the return sludge piping. The existing thickener feed pump may continue to be used for this purpose pending an evaluation in the predesign phase.

DISINFECTION

The two means of disinfection considered were ultraviolet (UV) light and chlorination. Chlorination has been eliminated because of safety and effluent toxicity concerns. Chlorine storage requires containment and scrubbing in addition to other safety requirements. The safety concerns are greater because of the close proximity of several homes. Furthermore, the water quality and mixing zone evaluations have determined that the presence of chlorine in the effluent would cause toxicity. Consequently, dechlorination would also be required, increasing the cost and operator attention required.

Ultraviolet disinfection systems are available with two types of bulbs: low pressure and medium pressure. The medium pressure systems require about one-tenth as many bulbs as the low pressure systems, resulting in a much lower maintenance and space requirement. Both closed-vessel and open-channel systems are available with medium pressure bulbs. Recent evaluations and discussions with manufacturers indicate that for a plant this size, the closed-vessel system is more cost-effective. A closed-vessel medium pressure system is proposed for Florence, although the specific selection will be further evaluated during predesign. The system includes three parallel units with a combined capacity equal to the PWWF.

In sizing a UV system, it is necessary to know the transmittance of the effluent. A pilot test conducted by the city indicated a transmittance of nearly 70 percent. This is higher than is typically assumed for design and indicates that UV disinfection should be very effective for the Florence plant. An additional test should be conducted during the design process.

OUTFALL AND MIXING ZONE EVALUATION

A mixing zone and outfall evaluation was recently performed to determine the effect of plant effluent on the Siuslaw River water quality and to provide a basis for preliminary design of a new outfall. The results of the mixing zone evaluation provide the target treatment requirements that are used in developing the alternatives for liquid stream treatment. The detailed evaluation report is included as Appendix C.

Background

A more detailed evaluation of the water quality of the Siuslaw River is presented in Chapter 2. From that evaluation, it was determined that temperature is the only parameter for which the river is water quality limited. The dissolved oxygen (DO) concentration is also of concern because a few excursions of the water quality standards have been noted. Other parameters which are pertinent to the wastewater treatment plant evaluation include pH, bacteria, and toxic substances. The toxic substances of concern are ammonia and chlorine. A summary of these parameters is presented in Table 6-1.

Data for the estuarine portion of the river are scarce. Although tide information is available, the resulting velocity of the current in the river is mostly unknown. The best estimate of velocity is from local boaters. Local fishermen report that the maximum velocity of the current is about 4 knots.

Salinity data are also generally unavailable. However, the city recently gathered limited salinity and temperature data near the existing outfall location. These results are compiled in the evaluation in Appendix C.

Hydraulic Analysis

In order to perform the mixing zone analysis, an outfall location and diffuser configuration must be assumed. A configuration was chosen that could allow the plant to discharge the PWWF at high tide without effluent pumping while providing good dilution characteristics. The hydraulic analysis indicated a total head loss of 5.3 feet in the entire outfall and diffuser at PWWF.

Table 6-1. Summary of Pertinent Water Quality Parameters

Water quality parameter	Comments
Temperature	The Siuslaw River is listed as water quality limited for temperature during the summer. For marine and estuarine waters, no significant increase in temperature above natural background levels is allowed above 0.25 degrees F at the edge of the mixing zone.
Dissolved oxygen	DO concentration in estuaries must be maintained above 6.5 milligrams per liter (mg/L). DEQ may set more restrictive DO limits in the future if the Siuslaw River is listed as a water quality limited stream.
pH	pH for all fresh and estuarine waters must remain between 6.5 and 8.5.
Bacteria	Bacteria standards are relatively stringent because the wastewater treatment plant discharges into an estuary containing shellfish-growing areas. The median fecal coliform concentration cannot exceed 14 organisms per 100 mL. In addition, no more than 10 percent of the samples can have more than 43 organisms per 100 mL.
Toxic substances	Toxicity limits for chlorine in marine water are 0.0075 mg/L for chronic toxicity and 0.013 mg/L for acute toxicity. Ammonia toxicity is dependent on water temperature, pH, and salinity.

There were two goals in performing the dilution analysis. The first goal was to roughly characterize the amount of overall dilution of plant effluent within the estuary over a tidal cycle. The second goal was to determine the dilution occurring near the outfall on a shorter time scale. The shorter term localized dilution results are used to establish the mixing zone.

Overall Tidal Dilution. An estimation of the volume of water entering the estuary from stream flows and tidal ocean exchange can be used to determine if effluent accumulation in the estuary could be a problem. The amount of ocean water entering the estuary can be estimated from the tidal fluctuations and the extent to which saltwater reaches upstream. Recent salinity measurements taken by the city indicate that the saltwater reaches about 13,000 feet upstream of the treatment plant, or about 6.5 miles upstream of the mouth of the river. This is a conservative estimate of saltwater intrusion because the measurements were taken in wintertime during fairly high river flows. During the summer, saltwater would extend further upstream.

A search through the NOAA tide predictions for Florence yields a minimum tidal elevation change of 0.9 feet. Applying this elevation change over the lower 6.5 miles of the river, which has an average channel width of about 1,100 feet, results in a minimum tidal prism of about 38 million cubic feet. This prism represents an average flow rate of 1,700 cubic feet per second (cfs) occurring over a 6-hour tidal cycle. Jet-like discharge conditions through the jetty and consistent littoral currents disperse the river plume so it is not reintroduced with the next tidal cycle. Combining this tidal flow rate with the summertime river flow of 75 cfs provides dilution of the plant's ADWF to a ratio of about 930:1. With a dilution ratio this high, effluent accumulation in the estuary should not be a significant environmental problem.

Mixing Zone Dilution. An analysis of dilution in the vicinity of the outfall diffuser is used to develop the size of the mixing zone and of the zone of initial dilution (ZID). Chronic toxicity can be exceeded only within the mixing zone and acute toxicity can be exceeded only within the ZID. Computer simulations were used to estimate the dilution provided by an assumed outfall configuration. Acute dilution factors were estimated using the computerized model, PLUMES, while chronic dilution factors were estimated using the program, CORMIX2. PLUMES is more stable in the highly turbulent area near the diffuser whereas CORMIX2 is more suited to the chronic condition because it takes into account boundary effects such as stream bank reflections.

The dilution results predicted by the CORMIX2 simulation are plotted in Figure 6-1. For the evaluation of chronic toxicity, an ambient current velocity of 1 meter per second was assumed. This velocity represents the average of the two extremes of zero and 2 meters per second observed in the estuary. From Figure 6-1, a dilution factor of 120:1 corresponds to this velocity.

The dilution results predicted by the PLUMES simulation are plotted in Figure 6-2. For the evaluation of acute toxicity, an ambient current velocity of 0.1 meter per second was assumed. This velocity represents the 10^{th} percentile current velocity in the estuary. From Figure 6-2, a dilution factor of 30:1 corresponds to this velocity.

Applying the above levels of dilution to the expected amounts of pollutants in the effluent provides an estimate of the impact of the effluent on the water quality. The standards for water quality parameters and the effect of the effluent on these parameters are discussed in detail in Chapter 5. To summarize, the expected mass discharge in the design year 2020 will not cause any violations of the water quality standards based on the dilution described above.

Toxicity must be evaluated on both acute and chronic levels. The two parameters normally of concern are ammonia and chlorine. Chlorine will not be present in the effluent because disinfection will be accomplished using ultraviolet light. Ammonia toxicity is a complex calculation dependent on several parameters. Using conservative estimates of ambient salinity, temperature, pH, and background ammonia concentration, the potential permit limits for ammonia were calculated using a statistical approach documented in the Environmental Protection Agency *Technical Support Document for Water Quality-based Toxics Control*.

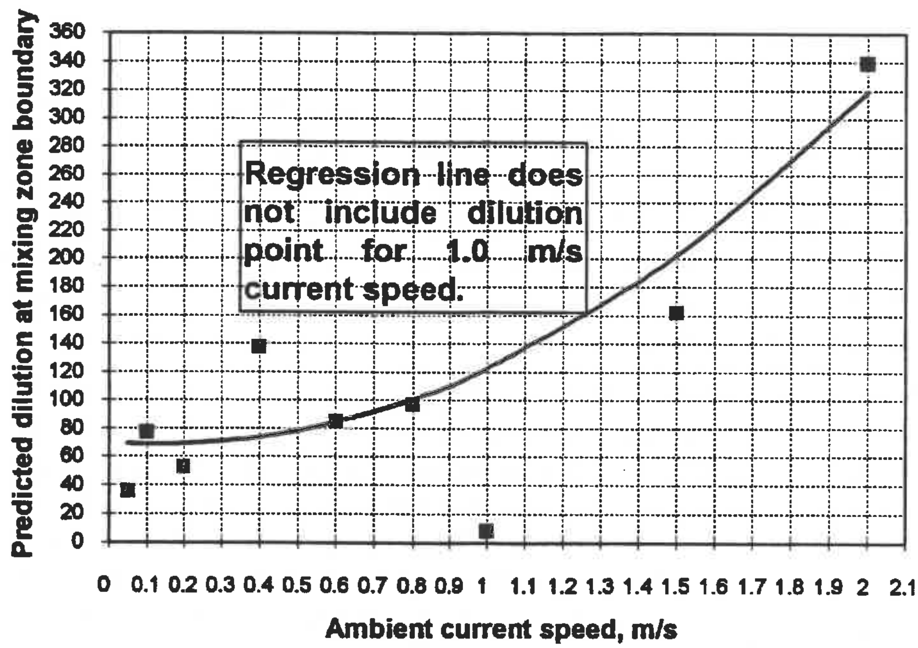

Figure 6-1. Chronic Dilution Predicted by CORMIX2 Model

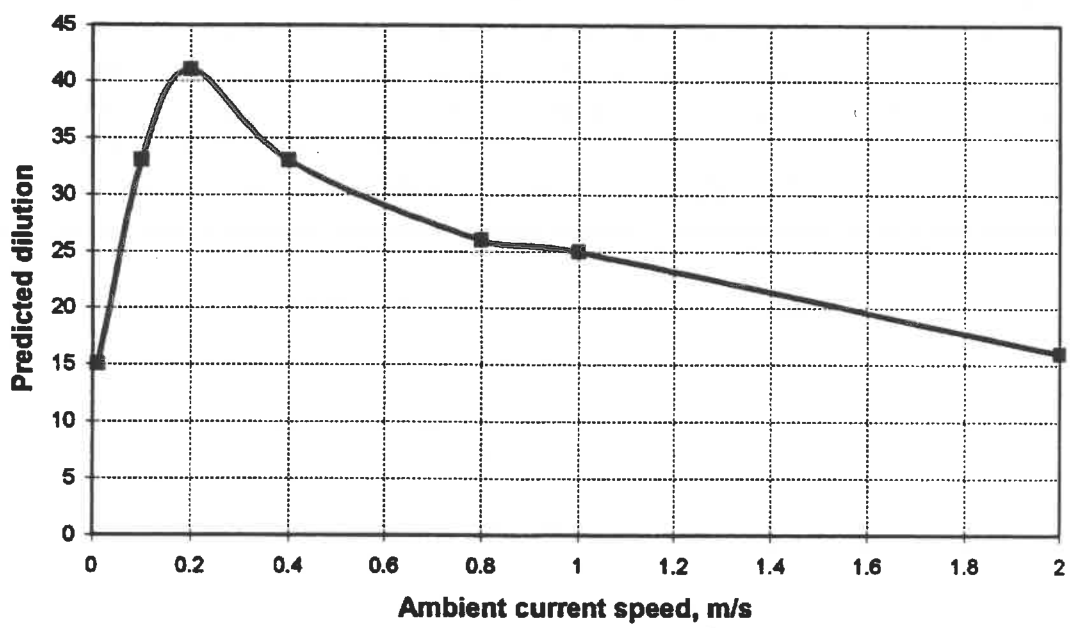

Figure 6-2. Acute Dilution Predicted by PLUMES Model

Based on this approach, the expected permit limits for ammonia would be 55 mg/L (as NH_3-N) for a daily maximum limit and about 21 mg/L for a monthly average limit. Refer to Attachment C in Appendix C for the detailed calculation.

Outfall Recommendations

Based on the mixing zone evaluation, DEQ should define the mixing zone to extend at least 210 feet both directions from the diffuser. The ZID should extend at least 21 feet upstream and downstream. More field data should be collected during the environmentally critical months of late summer. These data should include current velocity, salinity, temperature, pH, and background concentrations of DO and ammonia. The mixing zone study was based on conservative assumptions. Field data may show that less dilution is required, allowing construction of a smaller diffuser.

A new outfall would be required to carry the projected flows and to provide sufficient dilution to protect the water quality of the river. A profile of the proposed outfall and the river cross section is shown in Figure 6-3. Due to the inaccessibility of an outfall once constructed, the pipeline should be sized conservatively. The preliminary configuration consists of a 24-inch pipeline with a 200-foot diffuser extending nearly 700 feet from shore. The head loss through the 24-inch pipeline would be about 1.4 feet at the PWWF. The 24-inch pipeline would be adequate far beyond the design period. Even with a doubling of the design PWWF to 14 mgd, the head loss through the pipeline would be less than 6 feet. At the assumed population growth rate, flows this high would not be expected for more than 50 years.

The diffuser would be as close as possible to the dredged channel in the center of the river to utilize deeper water for better dilution. Submergence would be more than sufficient to prevent any interference with boating or other river activities. Precise location would be coordinated with the US Army Corps of Engineers to ensure that the diffuser would not interfere with future dredging operations. The diffuser would have 50 ports of 2-inch diameter. The head loss through the diffuser would be about 4 feet at the PWWF. The small ports would provide adequate velocity for good dilution at the design flows; however, the diffuser would require modification to handle flows much greater than the design PWWF.

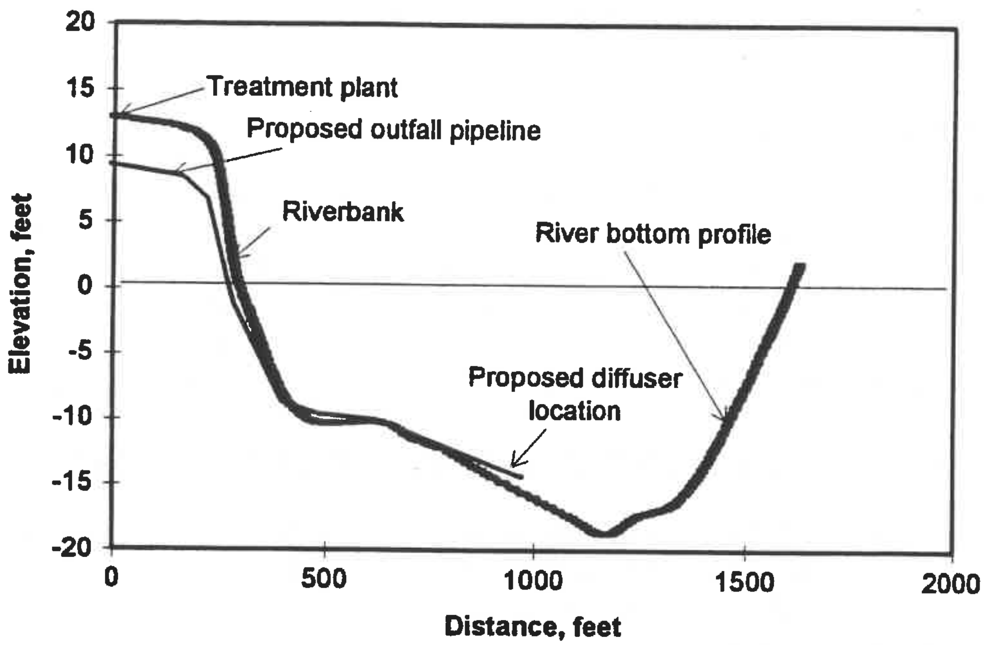

Figure 6-3. River Bottom and Proposed Outfall Profile

EFFLUENT REUSE

Effluent reuse is an option for treatment plants which are not allowed to discharge during the summer or whose mass discharge cannot be increased because of limitations in the receiving stream. The Siuslaw River water quality evaluation (refer to Chapter 5) has shown that the river is capable of assimilating an increased load. Because effluent reuse would require a much higher level of treatment, several miles of pipeline with effluent pumping facilities, and effluent storage, it is not a practical alternative to river discharge.

SEPTAGE RECEIVING

The existing plant does not receive or treat septage. Most of the septage haulers in the Florence area must either provide their own treatment or haul the septage at least 60 miles to Willamette Valley facilities. The treatment units described in this document have been sized without considering any septage load. The city has no obligation to accept septage; it is the city's choice whether to accept it, based on economic and political considerations.

If the city decides to accept septage, a septage receiving station would be included in the facilities. The receiving station would include coarse screening and grinding. The screened septage would probably be conveyed to the digester for treatment. Alternatively, it could be conveyed to the headworks. The digester would need to be larger to accommodate the additional load.

The septage receiving station and larger digester would add to the capital cost of the treatment plant improvements. Annual cost increases would be incurred in maintenance of the receiving station and in the additional sludge hauling and processing. However, septage treatment could possibly represent a source of revenue through fees for septage acceptance.

OPERATIONS BUILDING

As discussed in Chapter 3, the existing building is inadequate for storage, maintenance space, and laboratory space. A new building will be constructed housing the main control room, a laboratory, storage space, an office for the wastewater division supervisor, a meeting room, and locker and washroom facilities. The storage area will include a separate small room for flammable materials. The layout and relative area dedicated to each function will be developed during predesign. The area required is estimated to be roughly 1,800 square feet.

SUMMARY OF COMPLETE TREATMENT ALTERNATIVES

Three basic alternatives for wastewater treatment have been developed: activated sludge, SBR, and TF/SC. For the most part, the portions of the plant other than the secondary treatment process are similar for each alternative. In this section, schematics, site plans, and design data are presented for each complete alternative.

ACTIVATED SLUDGE ALTERNATIVE

This alternative would utilize the same treatment processes used currently at the existing plant. However, much more flexibility and redundancy would be built in.

Plant Schematic

A schematic diagram of the activated sludge alternative is presented in Figure 6-4. As the diagram shows, part of the influent flow is pumped to the plant through force mains and part of the flow enters the plant by gravity. All flow would be combined at the headworks.

The schematic shows that the major processes include parallel units. This provides redundancy, allowing the plant to continue to operate when one unit is out of service.

Site Plan

A proposed layout for the activated sludge alternative is shown on Figure 6-5. Details of the layout will be revised as the design process proceeds, but the general placement of the major unit processes is somewhat fixed. For example, the aeration basins must be constructed away from the existing facilities to allow the existing plant to remain in service during construction. Once the aeration basin and headworks are completed, the existing aeration basin can be shut down, allowing construction of new secondary clarifiers in the location of the old aeration basin. Once the clarifiers are complete, the existing clarifiers can be removed, allowing construction of a new digester.

Ample space is provided for sludge loading and removal of screenings and grit. The access is designed to eliminate the need for trucks to back in and turn around. The final design for traffic patterns is partially dependent on whether the city can obtain a permit to construct a driveway across the creek to the west of the plant.

Potential future expansions for the unit processes are shown in dashed lines on Figure 6-5. Several options are shown for expanding the secondary process. The process could be expanded directly by increasing the volume of aeration basin. Alternatively, primary sedimentation could be added thus reducing the load to the aeration basins. As another alternative, a trickling filter could be added with or without the addition of primary sedimentation. Regardless of which expansion option is selected, the layout can accommodate more than a doubling of the capacity provided in this phase.

Design Data

The design data for all of the unit processes in the activated sludge alternative are presented in Table 6-2. The values are those projected for the design year 2020. Although future units are shown on the site plan, design data beyond the design year are not included because it is unclear how much more the population could expand given geographical constraints in the urban growth area.

Table 6-2. Design Data For Activated Sludge Alternative

Item	Value
Plant flow	
ADWF, mgd	1.9
Peak month, mgd	3.6
Peak day, mgd	5.1
PWWF, mgd	6.9
Plant load	
BOD average, ppd	5,300
BOD max month, ppd	7,000
SS average, ppd	3,800
SS max month, ppd	4,800
Influent Pumps	
Type: Self-priming[a]	
Number	3
Capacity each, mgd[b]	1.5
Screen	
Type: Fine-mesh in-channel	
Number	2
Opening size, inches	0.25
Capacity each, mgd	5.3

Item	Value
Emergency bypass bar rack	
Number	1
Opening size, inches	1
Capacity, mgd	6.9
Grit Removal	
Grit chamber: Induced vortex	
Number	2
Diameter, ft	10
Capacity each, mgd	7.0
Grit pump: Recessed impeller	
Grit separation: Cyclone	
Grit dewatering: Auger	
Aeration	
Basins	
Number	2
Width, ft	30
Water depth, ft	15
Length, ft	165
Volume each, 1000 gallons	555
Operating modes available:	
Plug flow, step feed, contact stabilization	
Anaerobic selector	
Process performance[c]	
MLSS, mg/L	2,400
F/M, lb BOD/lb MLVSS/day	0.34
Sludge age, days	4.2
HRT, hours	7.3
Blowers	
Type: Multistage centrifugal[a]	
Number	4
Capacity each, scfm	2,000
Secondary clarifiers	
Type: Flocculator, peripheral weir	
Number	2
Diameter, ft	66
Sidewater depth, ft	17
SOR at peak day, gpd/sq ft	745
SOR at PWWF, gpd/sq ft	1,000
RAS pumping (per clarifier)	
Number of pumps	2

Item	Value
Capacity each, gpm	600
Disinfection	
Type: Closed vessel, medium pressure[a]	
Number of trains	3
Capacity each, mgd	2.3
Lamps per train	8
Outfall	
Length	700
Diameter, inches	24
Diffuser length, ft	200
Number of diffuser ports	50
Sludge thickener (existing)	
Type: Gravity belt	
Number	1
Belt width, m	1
Capacity, lb/hr	800
Thickened sludge tank	
Number	1
Diameter, ft	16
Volume, gallons	22,000
Height, ft	15
Anaerobic digesters	
Type: Mesophilic, fixed submerged cover	
Number	2
Diameter, ft (exist/new)	30/36
Sidewater depth, ft (exist/new)	14/24
Volume, cubic ft (exist/new)	12,070/28,400
SRT at peak month, days	28
Digested sludge holding tank	
Number	1
Diameter, ft	19
Height, ft	15
Volume, gallons	33,000
Sludge dewatering[d]	
Type: Belt or centrifuge[a]	

Item	Value
• Number	1
Capacity, lb/hr	1,900
Facultative sludge lagoon[d]	
Number	1
Area, acres	1.9
Depth, ft	12
Loading, lbVSS/1000 sq ft/day	20

Notes: [a] Equipment type selection is preliminary for cost estimating purposes. Selection may change during predesign.

[b] Influent pump station receives flow from new interceptor only. All other flow is pumped to plant from collection system pump stations.

[c] At maximum month conditions.

[d] Either sludge dewatering or an FSL would be provided, not both. These solids handling options are preliminary selections subject to change during predesign.

TF/SC ALTERNATIVE

The trickling filter unit process would be new for the city. However, the solids contact portion operates on the same principles as an activated sludge plant. Overall operations would be similar to those for activated sludge, although simpler due to the stability of the trickling filter process.

Plant Schematic

A schematic diagram of the TF/SC alternative is presented in Figure 6-6. With the exception of the biological treatment, the schematic is similar to that of the activated sludge process shown in Figure 6-4. In this alternative, the plant flow is pumped over a trickling filter. Under most conditions, a single trickling filter pump would operate at constant speed. As plant flow varies, the amount of flow recycled over the trickling filter would vary accordingly. During high flow periods two pumps would operate.

Although only one trickling filter would be provided, redundancy is incorporated because the process can be operated as an activated sludge system with the solids contact tanks serving as aeration basins if the trickling filter is shut down. The other processes include parallel units for redundancy.

Site Plan

A proposed layout for the TF/SC alternative is shown on Figure 6-7. As with the activated sludge alternative, the general placement of the major unit processes is constrained by the site configuration and placement of existing process units. For example, the trickling filter and solids contact tanks must be constructed away from the existing facilities to allow the existing plant to remain in service during construction. Once the solids contact tank is completed, the existing

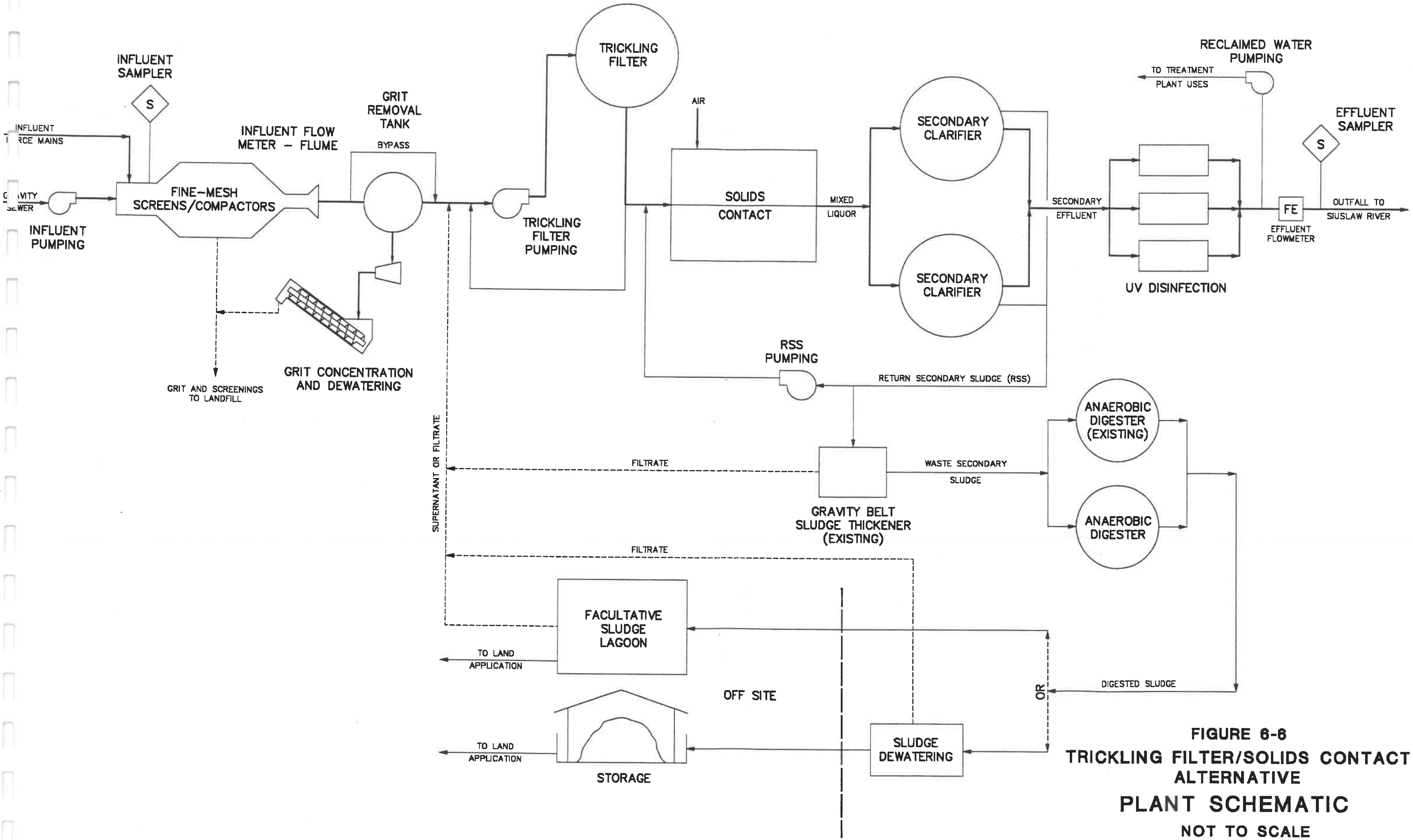

FIGURE 6-6
TRICKLING FILTER/SOLIDS CONTACT ALTERNATIVE
PLANT SCHEMATIC
NOT TO SCALE

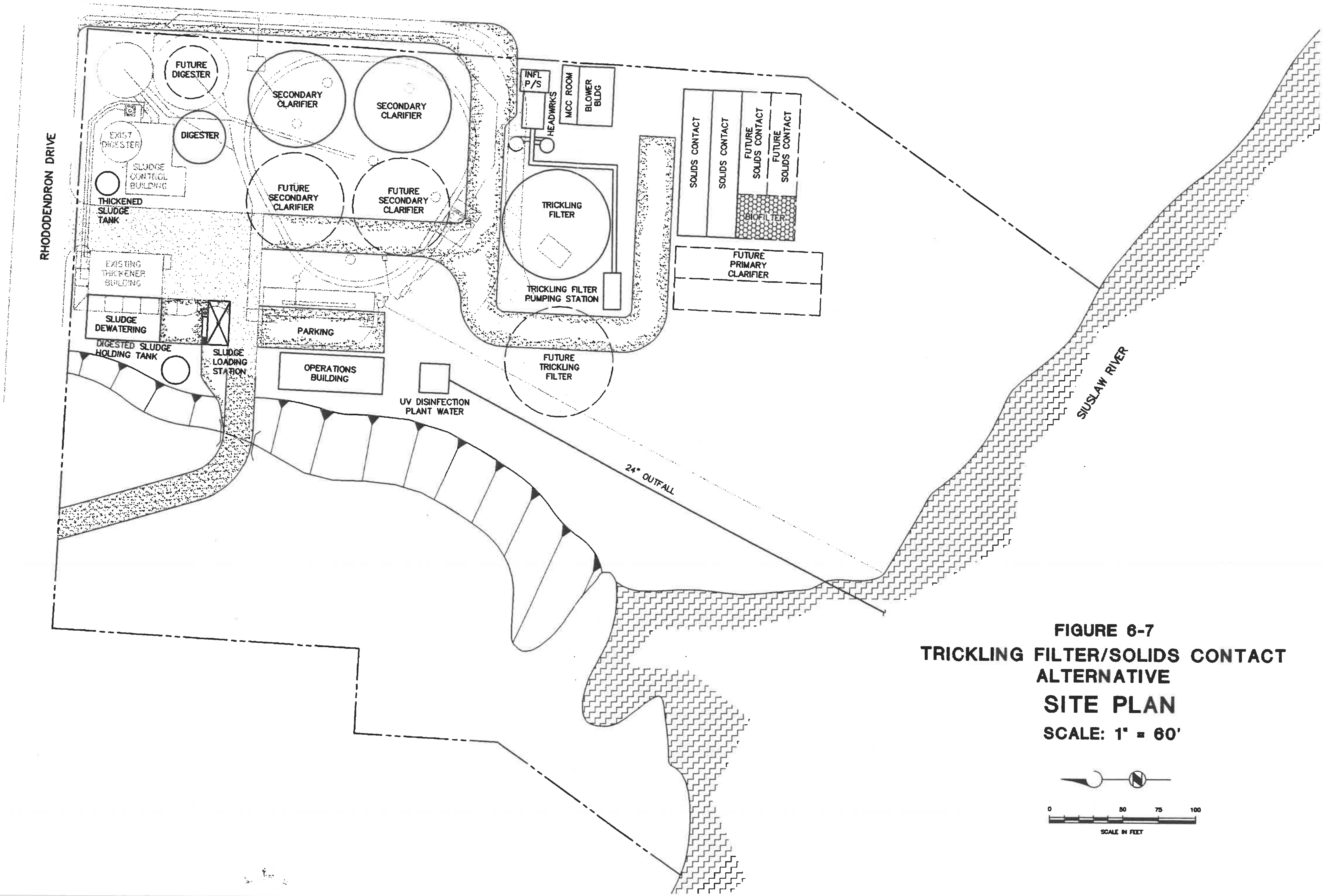

FIGURE 6-7
TRICKLING FILTER/SOLIDS CONTACT ALTERNATIVE
SITE PLAN
SCALE: 1" = 60'

Item	Value
Sludge dewatering[d]	
Type: Belt or centrifuge[a]	
Number	1
Capacity, lb/hr	1,900
Facultative sludge lagoon[d]	
Number	1
Area, acres	1.9
Depth, ft	12
Loading, lbVSS/1,000 sq ft/day	20

Notes: [a] Equipment type selection is preliminary for cost estimating purposes. Selection may change during predesign.

[b] Influent pump station receives flow from new interceptor only. All other flow is pumped to plant from collection system pump stations.

[c] Maximum month flow and load conditions assumed.

[d] Either sludge dewatering or an FSL would be provided, not both. These solids handling options are preliminary selections subject to change during predesign.

SBR ALTERNATIVE

Although the biological process for SBR is similar to activated sludge, the practical operation is quite different. Process changes are made by adjusting cycle times rather than by adjusting return sludge rates and tank volumes.

Plant Schematic

A schematic diagram of the SBR alternative is presented in Figure 6-8. In this alternative, the wastewater flows into the SBR and effluent is then decanted in batches to an equalization basin. It is then pumped through the disinfection system to the outfall. As plant flow varies, the cycle times would vary accordingly, as determined by the control module.

Two basins are provided. While one basin is quiescent for settling, the other accepts raw wastewater. Partial redundancy is provided in that the process can continue to operate to some degree with one basin out of service.

Site Plan

A proposed layout for the SBR alternative is shown on Figure 6-9. As with the other alternatives, the general placement of the major unit processes is constrained by the site configuration and placement of existing process units. For example, the SBR basins must be constructed away from

Item	Value
Sidewater depth, ft	16
SOR at peak day, gpd/sq ft	845
SOR at PWWF, gpd/sq ft	1,143
RAS pumping (per clarifier)	
Number of pumps	2
Capacity each, gpm	600
Disinfection	
Type: Closed vessel, medium pressure[a]	
Number of trains	3
Capacity each, mgd	2.3
Lamps per train	8
Outfall	
Length	700
Diameter, inches	24
Diffuser length, ft	200
Number of diffuser ports	50
Sludge thickener (existing)	
Type: Gravity belt	
Number	1
Belt width, m	1
Capacity, lb/hr	800
Thickened sludge tank	
Number	1
Diameter, ft	16
Volume, gallons	22,000
Height, ft	15
Anaerobic digesters	
Type: Mesophilic, fixed submerged cover	
Number	2
Diameter, ft (exist/new)	30/36
Sidewater depth, ft (exist/new)	14/24
Volume, cubic ft (exist/new)	12,070/28,400
SRT at peak month, days	28
Digested sludge holding tank	
Number	1
Diameter, ft	19
Height, ft	15
Volume, gallons	33,000

Item	Value
Opening size, inches	0.25
Capacity each, mgd	5.3
Emergency bypass bar rack	
Number	1
Opening size, inches	1
Capacity, mgd	6.9
Grit Removal	
Grit chamber: Induced vortex	
Number	2
Diameter, ft	10
Capacity each, mgd	7.0
Grit pump: Recessed impeller	
Grit separation: Cyclone	
Grit dewatering: Auger	
Trickling filter	
Number	1
Diameter, ft	75
Media depth, ft	16
Hydraulic load[c], gpm/sq ft	0.8
BOD load[c], ppd/1,000 cf	100
Interstage pumping	
Number of pumps	3
Capacity each, mgd	4.4
Aeration	
Basins	
Number	2
Width, ft	20
Water depth, ft	14
Length, ft	120
Volume each, 1,000 gallons	250
Blowers	
Type: Multistage centrifugal[a]	
Number	3
Capacity each, scfm	750
Secondary clarifiers	
Type: Flocculator, peripheral weir	
Number	2
Diameter, ft	62

aeration basin can be shut down, allowing construction of new secondary clarifiers in the location of the old aeration basin. Once the clarifiers are complete, the existing clarifiers can be removed allowing construction of a new digester.

Because the trickling filter is of much higher elevation than the other structures, it is placed as far north as possible on the currently vacant land. The more northerly location reduces the visibility from homes along the river, east and west of the plant. The aesthetics of the trickling filter are also improved by including a cover, which may be required for odor control.

Potential future expansions for the unit processes are shown in dashed lines on Figure 6-7. Two options are shown for expanding the secondary process. The process could be expanded directly by adding a trickling filter and increasing the volume of solids contact basin. Alternatively, primary sedimentation could be added, thus reducing the load to the secondary process. As with the activated sludge alternative, the layout can accommodate more than a doubling of the capacity provided in this phase.

Design Data

The design data for all of the unit processes in the TF/SC alternative are presented in Table 6-3. The values are those projected for the design year 2020. As under the activated sludge alternative, design data beyond the design year are not included.

Table 6-3. Design Data For TF/SC Alternative

Item	Value
Plant flow	
ADWF, mgd	1.9
Peak month, mgd	3.6
Peak day, mgd	5.1
PWWF, mgd	6.9
Plant load	
BOD average, ppd	5,300
BOD max month, ppd	7,000
SS average, ppd	3,800
SS max month, ppd	4,800
Influent Pumps	
Type: Self-priming[a]	
Number	3
Capacity each, mgd[b]	1.5
Screen	
Type: Fine-mesh in-channel	
Number	2

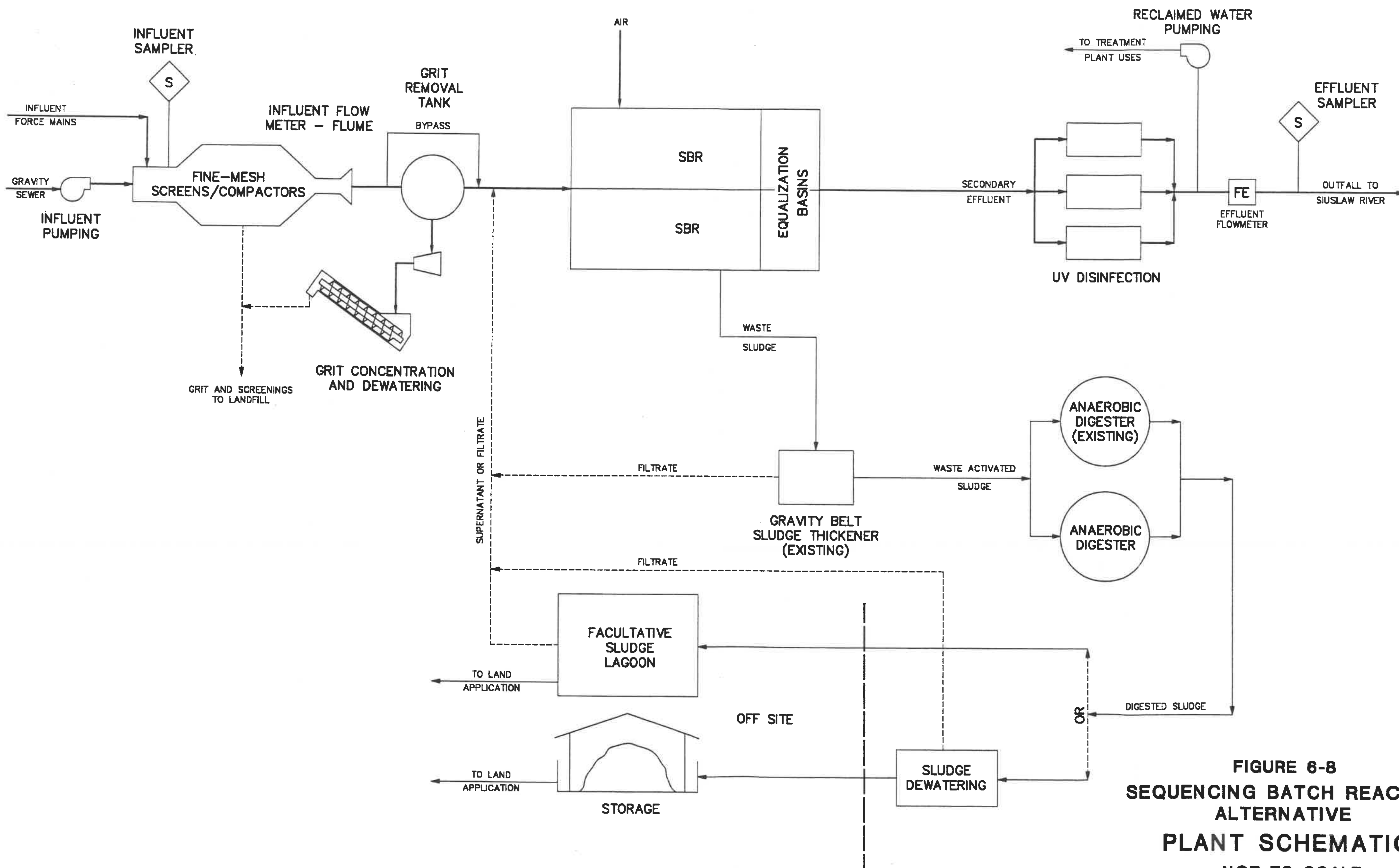

FIGURE 6-8
SEQUENCING BATCH REACTOR ALTERNATIVE
PLANT SCHEMATIC
NOT TO SCALE

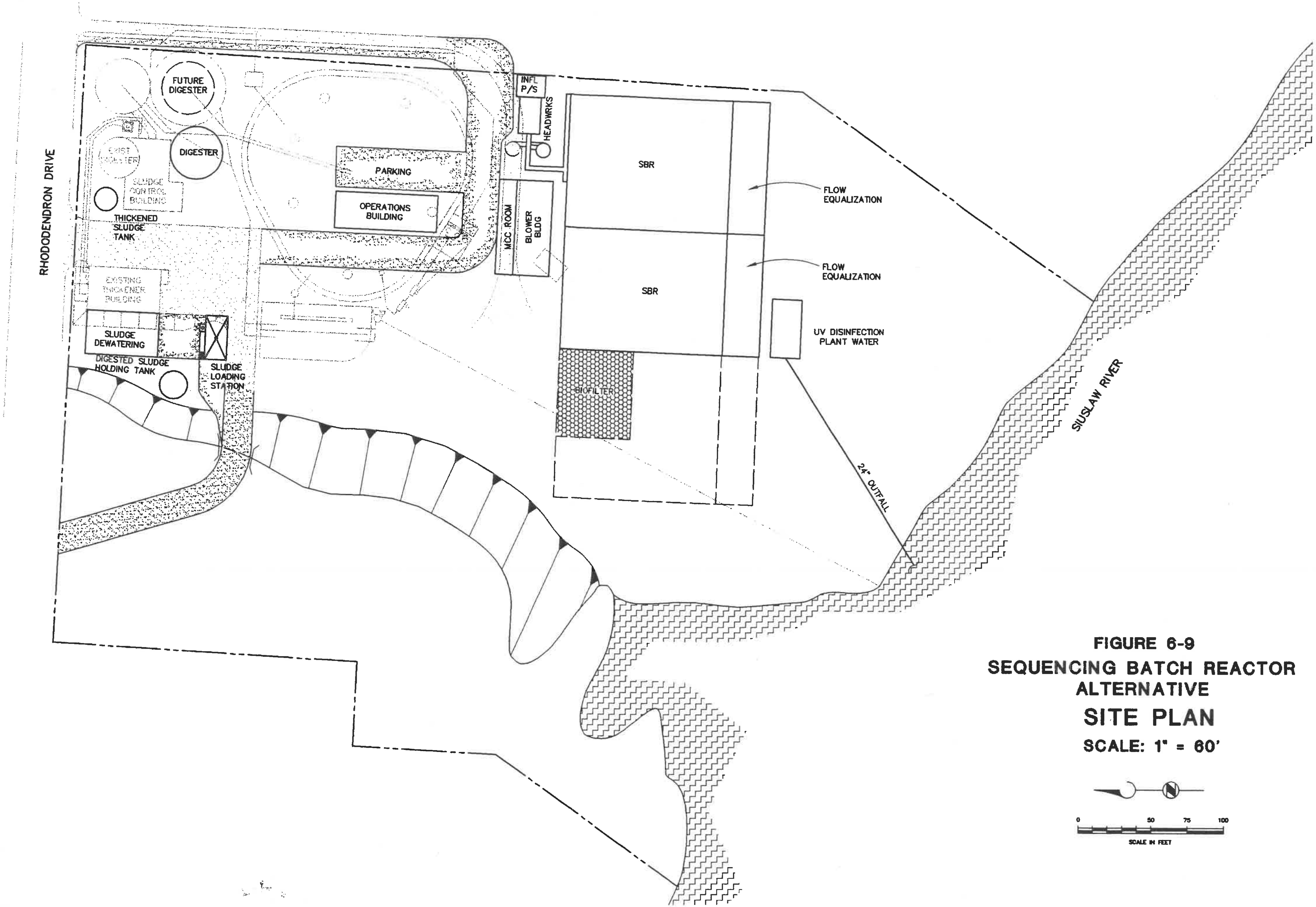

FIGURE 6-9
SEQUENCING BATCH REACTOR ALTERNATIVE
SITE PLAN
SCALE: 1" = 60'

the existing facilities to allow the existing plant to remain in service during construction. Once the SBR tanks and headworks are completed, the existing clarifiers can be removed allowing construction of a new digester.

Potential future expansions for the unit processes are shown in dashed lines on Figure 6-9. As with the activated sludge alternative, the layout can accommodate more than a doubling of the capacity provided in this phase.

Design Data

The design data for all of the unit processes in the SBR alternative are presented in Table 6-4. The values are those projected for the design year 2020. As under the other alternatives, design data beyond the design year are not included.

Table 6-4. Design Data For SBR Alternative

Item	Value
Plant flow	
ADWF, mgd	1.9
Peak month, mgd	3.6
Peak day, mgd	5.1
PWWF, mgd	6.9
Plant load	
BOD average, ppd	5,300
BOD max month, ppd	7,000
SS average, ppd	3,800
SS max month, ppd	4,800
Influent Pumps	
Type: Self-priming[a]	
Number	3
Capacity each, mgd[b]	1.5
Screen	
Type: Fine-mesh in-channel	
Number	2
Opening size, inches	0.25
Capacity each, mgd	5.3
Emergency bypass bar rack	
Number	1
Opening size, inches	1
Capacity, mgd	6.9

Item	Value
Grit Removal	
Grit chamber: Induced vortex	
Number	2
Diameter, ft	10
Capacity each, mgd	7.0
Grit pump: Recessed impeller	
Grit separation: Cyclone	
Grit dewatering: Auger	
Sequencing batch reactor	
Basins	
Number	2
Width, ft	92
Maximum water depth, ft	20
Bottom water level (decanted), ft	14
Length, ft	108
Volume each, 1000 gallons	1,500
Blowers	
Type: Multistage centrifugal[a]	
Number	3
Capacity each, scfm	3,700
Disinfection	
Type: Closed vessel, medium pressure[a]	
Number of trains	3
Capacity each, mgd	2.3
Lamps per train	8
Outfall	
Length	700
Diameter, inches	24
Diffuser length, ft	200
Number of diffuser ports	50
Sludge thickener (existing)	
Type: Gravity belt	
Number	1
Belt width, m	1
Capacity, lb/hr	800
Thickened sludge tank	
Number	1
Diameter, ft	16
Volume, gallons	22,000

Item	Value
Height, ft	15
Anaerobic digesters	
Type: Mesophilic, fixed submerged cover	
Number	2
Diameter, ft (exist/new)	30/36
Sidewater depth, ft (exist/new)	14/24
Volume, cf (exist/new)	12,070/28,400
SRT at peak month, days	28
Digested sludge holding tank	
Number	1
Diameter, ft	19
Height, ft	15
Volume, gallons	33,000
Sludge dewatering[c]	
Type: Belt or centrifuge[a]	
Number	1
Capacity, lb/hr	1,900
Facultative sludge lagoon[c]	
Number	1
Area, acres	1.9
Depth, ft	12
Loading, lbVSS/1,000 sq ft/day	20

Notes: [a] Equipment type selection is preliminary for cost estimating purposes. Selection may change during predesign.

[b] Influent pump station receives flow from new interceptor only. All other flow is pumped to plant from collection system pump stations.

[c] Either sludge dewatering or an FSL would be provided, not both. These solids handling options are preliminary selections subject to change during predesign.

CHAPTER 7

DEVELOPMENT OF SOLIDS MANAGEMENT OPTIONS

Solids management includes thickening, stabilization, and disposal of sludge produced in the biological treatment process. The quantity of solids produced in the biological treatment process is estimated at about 3,000 pounds per day. Assuming that the solids are 80 percent volatile and 55 percent reduced in the digestion process, this would result in production of about 1,600 pounds per day of digested sludge requiring disposal. Disposal is the most complex issue and the most difficult to resolve. The city currently has limited land available for sludge disposal. Although production of Class A sludge would increase disposal options, the additional cost or process requirements may not justify that option. In this discussion alternatives are broken down into two categories: production of Class A and Class B biosolids.

CLASS A BIOSOLIDS

As discussed in Chapter 5, sludge must meet Class A pathogen requirements to be land-applied without restriction. Although there are several methods of achieving Class A pathogen levels, many are better suited for larger plants or are prohibitively expensive. For this evaluation, Class A processes are categorized into two groups: processes utilized in conjunction with anaerobic digestion and the autothermophilic digestion process.

ANAEROBIC DIGESTION WITH ADDITIONAL TREATMENT

The plant currently uses anaerobic digestion to stabilize sludge, resulting in Class B biosolids. Under these alternatives, anaerobic digestion would continue to be used, with another process added to achieve Class A quality.

Anaerobic Digestion Improvements

To meet the increased loading projected for the future and to provide redundancy, a second anaerobic digester should be constructed and the existing digester should be refurbished. The new digester would be heated and mixed. A new boiler would be provided for digester and space heating. The boiler would operate on digester gas with oil as a standby fuel. The new digester would have either a fixed submerged or a floating cover. If a fixed cover were used, a separate digested sludge holding tank would be provided. Both digesters could then be fed continuously at a constant rate overflowing to the holding tank. Digested sludge could be withdrawn from the holding tank for hauling and/or dewatering as the operator's schedule dictated.

The existing digester will be emptied and cleaned when the new digester is operational. The upgrade would include new heating and mixing systems. Heat would be supplied by the boiler provided for the new digester. The inside of the tank would be thoroughly inspected and the appropriate structural repairs made.

lowest cost and most practical method of achieving Class A pathogen reduction with anaerobically digested sludge. Composting should continue to be considered if an appropriate site becomes available.

AUTOTHERMOPHILIC AEROBIC DIGESTION (ATAD)

ATAD is a single process that stabilizes the sludge, reduces the vector attraction, and reduces the pathogens to a Class A level. In this process, thickened sludge is aerobically digested at a high rate in two tanks in series. The heat generated by the reaction raises the sludge temperature to about 140 degrees F. The tanks are insulated to help maintain this temperature.

The process has a detention time of about 6 days. This is about one-third the detention time that anaerobic digestion requires. Consequently, the tanks would be about one-third the size. Like any aerobic digestion process, energy consumption is high. However, the short detention time reduces the significance of the energy consumption.

The quality of the digested sludge is poorer than that of anaerobically digested sludge. Odors are very strong unless it is dewatered. Dewatering is also more difficult than for anaerobically digested sludge.

The most significant concern with the process is odor. Although odor control would be included with the process, the city cannot accept the risk of odor caused by process upset or odor control malfunction. Several homes are located adjacent to the plant site.

The capital cost of ATAD is generally somewhat lower than that of anaerobic digestion. However, including the cost of odor control and the fact that a major portion of the anaerobic digestion process is already in place in Florence, the capital cost of ATAD would not be lower.

Based on the odor concerns and the fact that the process does not utilize the existing anaerobic digester, ATAD was eliminated from further consideration.

CLASS B BIOSOLIDS

The city's existing anaerobic digestion system currently produces Class B biosolids. The city is able to dispose of the sludge on land despite the restrictions imposed on disposal of Class B sludge. However, if the city is to continue this strategy, some action must be taken to meet the requirements of increasing sludge production in the coming years. Options include: continuing to apply liquid sludge on private land, applying liquid on dedicated land, dewatering sludge to increase storage capacity, and construction of a lagoon for storage and treatment.

LAND APPLICATION OF ANAEROBICALLY DIGESTED LIQUID SLUDGE

The city's existing program utilizes this strategy on private land. Continuing this program would be the least cost alternative for sludge disposal for now. However, as sludge production increases, more land will soon be required. Additional private land is difficult to find and will require hauling liquid sludge greater distances. Eventually, the costs may become higher than those for other alternatives. Furthermore, reliability is poor. During wet weather sludge cannot recommences, the storage provided by the FSL allows the operator to work around long periods of poor weather, equipment breakdowns, or other constraints to land disposal. Liquid sludge could be land-applied at the FSL site, eliminating the need for additional hauling.

Disadvantages of an FSL include potential odor and insect nuisance problems, land area requirements, cost of construction, and potential resistance from the public. A major portion of the cost is from the liner required to eliminate leakage into the groundwater. The liner would be particularly important in the Florence area because of the sandy soils. The land area requirements would be greater than for dewatered sludge storage. A large buffer area would also be required.

Although some sludge lagoons constructed in the past have presented significant odor problems, proper design minimizes the odor potential. Criteria for design include:

- Sufficient depth for an adequate aerobic zone above the sludge storage (anaerobic) zone.
- Proper loading rate.
- Consideration of wind direction in pond layout to minimize wave action.
- Sufficient buffer area.

The large amount of rainfall along the coast poses another disadvantage for an FSL in that allowance must be made for the rainwater entering the FSL. To account for the additional water, the lagoon must be made deeper and more supernatant must be hauled back to the treatment plant. To minimize the impact of rainwater accumulation, the lagoon should be made as small as possible. By phasing the construction, a smaller lagoon could be built now, with provisions for expansion when needed. Much of the supernatant could be hauled on the return trips from hauling sludge to the lagoon. The supernatant would not require additional hydraulic capacity at the plant because hauling could be suspended during peak flow periods. Supernatant could also be irrigated on the land adjacent to the lagoon during the summer. A larger site would be advantageous in that it could accommodate more irrigation of supernatant. Application of supernatant would be limited to agronomic rates and would require harvesting to remove the accumulated nutrients. With summertime irrigation, extra trips (in addition to sludge-hauling trips) for hauling supernatant would probably be unnecessary. Alternatively, if a site is found reasonably close to the wastewater collection system, supernatant could be returned via a pipeline.

The FSL option should continue to be considered. Even if it is not economical at this time, it may become more so in the future if sludge disposal sites become more scarce.

SUMMARY AND RECOMMENDATIONS

Composting is the only option for producing Class A biosolids that has not been eliminated. The cost and labor efforts should be evaluated and compared with the Class B options. Composting will cost more than the Class B options, but provides more flexibility in sludge disposal. All of the Class B options should be evaluated further. Most likely, the recommended solids handling program will include a combination of several of the Class B options.

These options are presented in Table 7-1 below with a listing of the major components of each option and the class of sludge produced.

Table 7-1. Solids Handling Options

Option	ATAD	Dewatering	Composting	FSL
Major components	ATAD Holding tank Dewater Storage Apply solid	Anaerobic digestion Dewater Storage Apply solid	Anaerobic digestion Dewater Compost Storage Apply/give away	Anaerobic digestion (Thicken) FSL Apply liquid Irrigate supernatant
Sludge class	A	B	A	B
Land area (process)	20 acres	20 acres	25 acres	50 acres
Land area (application)	50 acres	100 acres	25 acres	100 acres

A detailed evaluation of these options is presented in Chapter 8.

CHAPTER 8

EVALUATION OF ALTERNATIVES

In this chapter, the treatment alternatives developed in Chapters 6 and 7 are evaluated in detail. In this evaluation, both economic and noneconomic factors are considered.

ECONOMIC EVALUATION BACKGROUND

To make a valid comparison among alternatives, a present worth analysis is necessary in order to incorporate both capital and annual costs in the evaluation. In developing costs for the present worth analysis, many assumptions must be made to compensate for the lack of detail available during the facilities planning process. The analysis techniques and assumptions made are described below.

PRESENT WORTH ANALYSIS

In a present worth analysis, annual costs over the economic life of the alternative are brought from the future back to the present, discounted by an annual percentage rate called the discount rate. Once the annual costs are brought to the present as a single sum, they can be added to the capital cost to derive the total present worth. For this analysis, the discount rate is assumed at 8 percent. The analysis period, or economic life, is assumed to be 20 years. Salvage values, or the value at the end of the 20-year study period, are not considered in this analysis.

PRECISION OF COST ESTIMATES

The precision of a cost estimate is a function of the detail to which alternatives are developed and the techniques used in preparing the actual estimate. The American Association of Cost Engineers divides estimates into three basic categories:

1. Order-of-Magnitude Estimate. An order-of-magnitude estimate is made without detailed engineering data. Techniques such as cost-capacity curves, scale-up or scale-down factors, and ratios are used in developing this type of estimate. This type of estimate is normally accurate within +50 percent or -30 percent. That is, the final cost may be as much as 50 percent more or 30 percent less than the estimated amount. A relatively large contingency is normally included to reduce the probability of underestimating.

2. Budget Estimate. This estimate is prepared using process flow sheets, layouts, and equipment details. An estimate of this type is usually accurate within +30 percent and -15 percent.

3. Definitive Estimate. As the name implies, this estimate is prepared from well-defined engineering data, including construction plans and specifications. As a minimum, the data would include comprehensive plot plans and elevations, piping and instrument

diagrams, electrical diagrams, equipment data sheets and quotations, structural drawings, soil data and drawings, and a complete set of specifications. The definitive estimate is expected to be accurate within +15 percent and -5 percent.

The estimates presented in this document are order-of-magnitude estimates because the design has not been developed in sufficient detail for a more precise estimate. Although the final project cost may vary significantly from these estimates, the estimates are useful in evaluating alternatives because they are fairly accurate relative to each other.

BASIS FOR COSTS OVER TIME

Future changes in the costs of material, labor, and equipment will cause comparable changes in the costs presented in this analysis. However, because the relative economy of the alternatives should change only slightly with overall economic changes, the decisions based on the economic evaluation should remain valid.

Costs can be expected to undergo long-term changes in keeping with corresponding changes in the national economy. One of the best available indicators of these changes is the *Engineering News-Record* (ENR) construction cost index. It is computed from the prices for structural steel, Portland cement, lumber, and common labor, and is based on a value of 100 in the year 1913. Figure 8-1 shows the trend of the ENR index since 1980. The dashed portion of the line indicates expected future increases, based on the past trend.

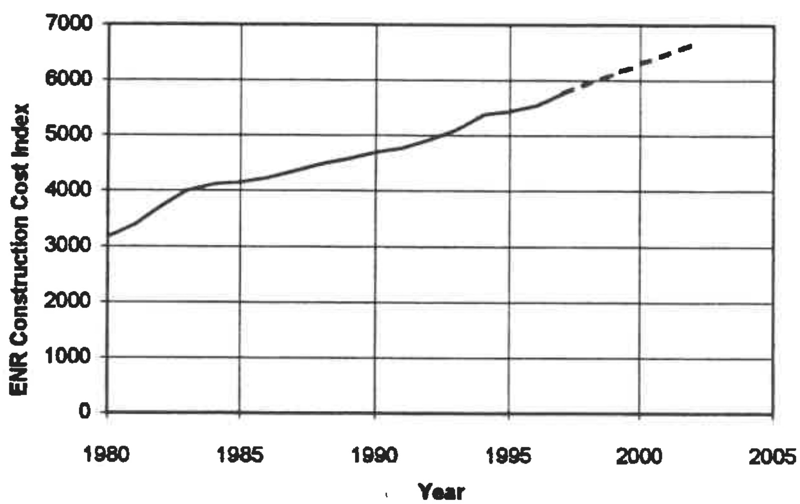

Figure 8-1. ENR Construction Cost Index Trend

The costs developed in this analysis are based on the current ENR 20-city index of 5800. The costs presented here may be related to those at any time in the past or future by applying the ratio of the then-prevailing cost index to 5800.

ENGINEERING AND ADMINISTRATIVE COSTS AND CONTINGENCIES

The cost of engineering services for major projects typically covers special investigations, a predesign report, surveying, foundation exploration, preparation of contract drawings and specifications, construction management, start-up services, the preparation of operation and maintenance manuals, and performance certifications. Depending on the size and type of project, engineering costs may range from 13 to 20 percent of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects and projects that involve extensive remodeling of existing plants.

The City of Florence has its own administrative costs associated with any major construction project. These include internal planning and budgeting, the administration of engineering and construction contracts, legal services, and liaison with regulatory and funding agencies. For a typical project of this size, the city's administrative costs will be about 4 percent of the contract cost. The total cost for engineering and administration is assumed to be 20 percent.

ECONOMIC COMPARISON OF LIQUID STREAM ALTERNATIVES

The three treatment alternatives developed in Chapter 6 are activated sludge, trickling filter/solids contact, and sequencing batch reactor. Construction cost breakdowns for each alternative are presented first, followed by annual costs. Then the construction and annual costs are combined in the present worth analysis.

CONSTRUCTION COSTS OF ALTERNATIVES

The construction costs for the alternatives, broken down by process area, are summarized in Table 8-1. Costs of engineering and administration are not included, as these costs do not affect the relative cost of one alternative compared to the others. Construction cost estimates for the components of these alternatives are based on costs of other similar installations, estimates of quantities of materials, and cost curves from various sources.

As indicated in the table, the construction costs for all three alternatives are very similar. Within the accuracy of this estimate, the cost totals for activated sludge and trickling filter/solids contact (TF/SC) are indistinguishable. The sequencing batch reactor (SBR) alternative is about 6 percent less costly. Although SBRs often have a significantly lower construction cost than the other alternatives in this flow range, only minor savings are indicated in this estimate. The unusually high cost of the SBR system is a result of the abnormally high biochemical oxygen demand loading at the Florence plant. The SBR, which is an extended aeration process, must utilize exceptionally large aeration basins and aeration equipment to treat the heavy organic load.

Table 8-1. Construction Cost Breakdowns for Liquid Stream Treatment Alternatives

Item		Costs, $1,000		
		A/S	TF/SC	SBR
Contractor indirects		469	469	469
Influent pumping		368	368	370
Yard development		384	373	373
Headworks		773	773	773
Odor control		237	237	237
Trickling filter		0	1,145	0
Aeration basins		866	434	1,631
Blower building		553	286	420
Secondary clarifiers		1,381	1,247	0
Yard piping		341	341	256
Flow equalization		0	0	160
Electrical/instrumentation		1,680	1,470	1,848
Disinfection		692	692	692
Outfall		558	558	558
Operations building		287	287	287
Subtotal		8,589	8,680	8,073
Bond @	1%	86	87	81
Contingency @	15%	1,288	1,302	1,211
Total construction cost		9,963	10,069	9,365

ANNUAL COSTS OF ALTERNATIVES

Estimated annual costs and the present worth of the annual costs are summarized in Table 8-2. Annual costs include electrical power consumption, labor, and chemical use. The major power-consuming processes are aeration and ultraviolet disinfection. Labor costs are estimated by assuming a number of shifts required to operate the plant and perform routine maintenance. Because the alternatives do not use chlorine for disinfection, chemical use is zero. Polymer use for sludge thickening is included in the solids handling cost estimate.

Table 8-2. Annual Costs for Liquid Stream Treatment Alternatives

Item	Costs, $1,000		
	A/S	TF/SC	SBR
Power, $1,000[a]	74	52	104
Labor[b]	374	331	331
Chemicals	0	0	0
Total annual cost	448	383	435
Present worth of annual cost[c]	4,394	3,755	4,271

Notes: [a] Power assumed at $0.05 per kWh.

[b] Labor assumed at $35 per hour, with supervisor at $45/hour.

[c] Present worth based on discount rate of 8% and 20 year life.

As expected, the low electrical power and labor requirements for TF/SC result in the lowest annual cost for this alternative. The computerized controls and lack of secondary clarifiers for the SBR result in low labor costs, but the extended aeration process of the SBR system consumes the highest amount of electrical power, resulting in a high annual cost. The efficient activated sludge aeration process uses less electrical power than the SBR. However, labor costs are higher because the significant flexibility offered by the activated sludge process requires more operator attention.

PRESENT WORTH COST OF LIQUID STREAM ALTERNATIVES

The total present worth cost of each alternative incorporates both the construction cost and the annual costs. By determining the present worth of the annual costs, they can be added to the construction cost, giving one value to form the basis for the economic comparison of the alternatives. These costs are presented in Table 8-3. The difference in total present worth between the lowest cost and highest cost alternative is only 5 percent. This difference is insignificant within the accuracy of this estimate.

Table 8-3. Present Worth Cost of Liquid Stream Alternatives

Item	Costs, $1,000		
	A/S	TF/SC	SBR
Construction cost	10,050	10,127	9,451
Present worth of annual cost[a]	4,394	3,755	4,271
Total present worth[a]	14,444	13,882	13,722

Note: [a]Present worth based on 8 percent discount rate over 20 years.

EVALUATION AND RANKING OF LIQUID STREAM ALTERNATIVES

The liquid stream treatment alternatives are activated sludge, TF/SC, and SBR. A brief evaluation of the alternatives relative to each of the ranking criteria, followed by the selection recommendation, is presented below.

ENVIRONMENTAL IMPACTS

Most of the environmental impact criteria are affected equally by each alternative. For example, all three alternatives would have an equivalent effect on water quality, air quality, solid waste generation, and historic preservation. Energy consumption is one criterion which distinguishes the alternatives. The SBR alternative consumes twice as much energy as TF/SC. Activated sludge energy consumption is midway between the other alternatives. The SBR alternative also requires more land closer to the river than do the other alternatives. For this criterion, activated sludge and TF/SC are ranked equal, with SBR ranked lower.

EASE OF IMPLEMENTATION

This factor refers to the ease of construction and start-up while maintaining the existing process in operation. The SBR alternative has a slightly greater ease of implementation because the entire process would be constructed away from the existing plant. However, the other alternatives present no significant difficulty because the headworks and biological treatment units would be constructed away from the existing plant. Once these units are in service, secondary clarifiers could be constructed without difficulty in the location of the existing aeration basin. Therefore, this factor is not considered to favor one alternative over another.

EASE AND RELIABILITY OF OPERATION

TF/SC and SBR are ranked equivalent with respect to ease of operation. Activated sludge requires more operator attention, particularly when flow and load conditions change. However, this is not considered a significant disadvantage because activated sludge is the process currently utilized and best understood by the city staff.

TF/SC is considered the most reliable process. The trickling filter handles flow and load variations well and is a simple process requiring little attention. The process is not dependent on continuous functioning of complex components. SBR is considered less reliable because the entire process relies on proper operation of the computerized control system, automatic valves, and the decant system. Failure of any of these components could have a major impact on treatment. Activated sludge is nearly as reliable as TF/SC. It is not vulnerable to failure of a single component, but does require more operator attention to optimize treatment. With respect to reliability, activated sludge and TF/SC are ranked equally, with SBR ranked lower.

PERMITS AND REGULATORY ASPECTS

The three alternatives are well-known and accepted by the regulatory agencies. All three alternatives are ranked equally with respect to this criterion.

FLEXIBILITY

Flexibility refers to both the flexibility in operation and future expansion. Activated sludge is the most flexible in operation. There are several modes of operation available to optimize treatment under various conditions. Sludge reaeration and step feed modes protect the solids inventory during peak flow periods. Biological selector modes can be used to improve sludge settling characteristics during low flow periods. Basins can be removed from service as flows and loads permit. SBR is also rather flexible. The timing of the sequence can be adjusted easily to optimize treatment under varying conditions. However, basins can not generally be removed from service on a long-term basis. TF/SC is less flexible; the entire filter is either used or bypassed. However, operational flexibility is less necessary because of the stability of the process. Activated sludge is ranked the highest in operational flexibility, with TF/SC and SBR ranked second.

With respect to construction and future expansion, activated sludge is the most flexible. Basins of the optimum size can be added in phases as increasing loads demand. For TF/SC and SBR, an entire new unit must be added when loads demand, resulting in paying for more excess capacity for the early years of the design period. This disadvantage is more significant for TF/SC because the excess capacity can lead to decreased treatment efficiency. Activated sludge is ranked the highest, with SBR second and TF/SC third.

AESTHETICS

The aesthetics criterion refers to visual and other effects (noise, odor, traffic) on nearby residents and the public. With respect to nonvisual effects, all three alternatives are considered equivalent, assuming that the trickling filter would be covered and provided with odor control; effects will be minimal. With respect to visual effects, activated sludge is ranked the highest. TF/SC is ranked second as a result of the high profile of the trickling filter. The top of the filter would be about 18 feet above grade with the cover reaching several feet above that. SBR is ranked third because the relatively large tanks would be situated close to the river. This location is visible from several homes as well as condominiums upstream along the river. The tanks would also have a greater impact on the view of river users and people in the sand dunes.

ECONOMICS

The budgetary cost estimates for the three alternatives were presented above and summarized in Table 8-3. The 1.5 percent difference between the present worth costs of TF/SC and SBR is insignificant within the accuracy of the estimate. The 6 percent difference between the costs of activated sludge and SBR is also of little significance. On the economic basis, TF/SC and SBR are ranked first, with activated sludge ranked lower.

SELECTION OF RECOMMENDED ALTERNATIVE

The rankings of the alternatives with respect to the criteria discussed above are summarized in Table 8-4. Criteria for which all three alternatives are considered equal are not included in the table.

Table 8-4. Summary of Treatment Alternative Rankings

	Alternative Ranking		
Criteria	A/S	TF/SC	SBR
Environmental impact	1	2	3
Reliability	1	1	2
Flexibility in expansion	1	3	2
Flexibility in operation	1	2	1
Aesthetics	1	2	3
Economics	2	1	1

As the table shows, the activated sludge alternative is ranked first in every category except cost. Although cost is considered one of the most important criteria, the cost differential between the alternatives is so small that ranking on the basis of cost should not be given much weight. Within the accuracy of the budgetary cost estimates, the difference between the alternatives is almost insignificant.

From a cost standpoint, a more significant criterion is flexibility in expansion. Providing more flexibility in expansion will afford the city definite cost savings in the future. Activated sludge provides the most flexibility.

Reliability and aesthetics are also considered very important. Past violations of the discharge permit have caused controversy within the city. It is crucial to the city that violations do not occur in the future. Providing the most reliable plant will ensure that violations of the permit do not occur. Aesthetics are important because the plant is located close to residences and recreational areas. Tourism represents a significant portion of the city's economy. A plant with the least aesthetic impact will have the least effect on tourism and will be the most easily accepted by stakeholders in the community.

Any of the three alternatives would provide a reliable, operator-friendly plant for a reasonable cost. Taking into account all the factors discussed above, activated sludge would provide the best fit for the city's needs. Consequently, activated sludge is the recommended alternative.

ECONOMIC COMPARISON OF SOLIDS HANDLING OPTIONS

Four major options were developed in Chapter 7 for detailed evaluation. An economic evaluation of these options are presented below.

CONSTRUCTION COSTS OF OPTIONS

The construction costs for the options are summarized in Table 8-5.

Construction cost estimates for the solids handling options have been developed in the same manner as described above for the liquid stream treatment alternatives. The price of land is assumed at 3,000 dollars per acre, based on preliminary investigations into availability of land.

The facultative sludge lagoon (FSL) is the lowest cost option by about 20 percent. Excluding the cost of land, composting is the most expensive option. However, because it is assumed that compost would require 25 percent as much land for application as would dewatered sludge, the total cost for composting is less than for dewatering. The reduced land requirement for compost results from the lack of restrictions on application of the Class A material, and the assumption that about half of the finished compost could be given to the public.

Table 8-5. Construction Costs for Solids Handling Options

Item	Costs, $1,000			
	ATAD	Dewatering	Composting	FSL
ATAD equipment, tanks	1,150	--	--	--
ATAD foundation (includes piles)	78	--	--	--
Feed and discharge pumps, piping	70	--	--	--
Convert exist digester to holding tank	100	--	--	--
Anaerobic digestion and holding tanks	--	1,483	1,483	1,483
Sludge dewatering	1,001	1,001	1,001	--
Dry box truck	60	60	60	--
Dewatered sludge storage	475	475	475	--
Manure spreader	72	72	72	--
Loader	100	100	100	--
Blower, air piping	--	--	10	--
Shredder	--	--	52	--
Chipper	--	--	30	--
Tank truck	--	--	--	100
Dredge	--	--	--	50
FSL	--	--	--	460
Supernatant irrigation system	--	--	--	50
Access road	50	50	70	100
Subtotal	3,156	3,241	3,353	2,243
Contingency @ 15 percent	473	486	503	336
Land	210	360	150	450
Total	3,839	4,087	4,006	3,029

ANNUAL COSTS OF OPTIONS

The estimated annual costs for the four solids handling options are summarized in Table 8-6. The assumption of round-trip hauling distance is critical to this analysis. As hauling distance is increased, the cost of the FSL option increases more rapidly because this option involves hauling liquid sludge, requiring more trips. Refer to the present worth comparison below for an estimate of the hauling distance at which FSL option and the dewatering option would break even.

Autothermophilic aerobic digestion (ATAD) is the only option with significant electrical power costs. ATAD is an aeration process which consumes a relatively large amount of energy. The other processes consume very little energy. Anaerobic digestion, on the other hand, produces digester gas that will be used for building heating, actually saving energy.

The most significant expense for the composting option is the labor required to mix the sludge with amendment, turn the piles, and screen the materials. It is assumed that the finished compost will be in sufficient demand by the public and that only half of the material will require land application.

Table 8-6. Annual Costs for Solids Handling Options

Item	Costs, $1,000			
	ATAD	Dewatering	Composting	FSL
Electrical power[a]	26	-2	2	2
Chemicals (polymer)	15	15	15	9
Labor for process[b]	38	38	93	6
Hauling to storage[c]	16	16	16	40
Application	21	21	11	53
Total annual cost	116	88	137	110
Present worth of annual cost[d]	1,140	865	1,341	1,082

Notes: [a] Electricity assumed at $0.05 per kWh. All options except ATAD include a $5,000 credit for anaerobic digester gas production.

[b] Labor assumed at $35 per hour.

[c] Hauling distance assumed at 20-mile round-trip.

[d] Present worth based on discount rate of 8 percent over 20-year period.

PRESENT WORTH COST OF SOLIDS HANDLING OPTIONS

The total present worth is the sum of the capital cost and the present worth of the annual costs. These costs are summarized in Table 8-7. ATAD and dewatering have about the same life cycle cost. Composting is more costly by about 7 percent. The FSL option is substantially less expensive in both capital and annual costs. The total present worth is about 80 percent that of the next most economical option. As mentioned above, this analysis is sensitive to hauling distance. As hauling distances become greater, the dewatering options become more favorable. Based on the assumptions made in this analysis, dewatering would become cost-effective if the distance to the application sites exceed 60 miles. This break-even distance could vary somewhat, depending on actual sludge production, truck capacity, labor rates, and other factors. However, for distances as short as 10 or 20 miles, the FSL is clearly the most cost-effective option. An additional economic advantage to the FSL option is that no land application would be required for the first two years of operation.

Table 8-7. Total Present Worth Costs for Solids Handling Options

Item	Costs, $1,000			
	ATAD	Dewatering	Composting	FSL
Construction cost	3,839	4,087	4,006	3,029
Present worth of annual cost	1,140	865	1,341	1,082
Total present worth[a]	4,979	4,952	5,347	4,111

Notes: [a] Present worth based on 8 percent discount rate and 20-year study period.

In this chapter the liquid and solids treatment alternatives developed previously are evaluated and ranked on the basis noneconomic factors as well as present worth cost. Noneconomic factors considered in this evaluation are environmental impacts, ease of implementation, ease and reliability of operation, regulatory aspects, flexibility, and aesthetics. The highest ranked alternatives are selected for the recommended plan.

EVALUATION AND RANKING OF SOLIDS HANDLING OPTIONS

The solids handling options are ATAD, dewatered digested sludge, composted digested sludge, and FSL. A brief evaluation of the alternatives relative to each of the ranking criteria, followed by the selection recommendation, is presented below.

ENVIRONMENTAL IMPACTS

Each of the options has fairly similar environmental impacts. They all require land for storage and application. The FSL would require more land for storage and fuel consumption for sludge hauling. The ATAD option would consume more energy than the other options. It would have a continuous power draw of about 60 horsepower as opposed to less than 5 horsepower for the other options.

EASE OF IMPLEMENTATION

The FSL option would have the greatest ease of implementation because all construction would take place at a remote site. The other three options would require a new dewatering facility at the existing treatment plant. Construction of this facility would cause some additional inconvenience to the plant operators. Construction of ATAD would cause even more disruption as the entire sludge digestion system at the plant would be replaced.

EASE AND RELIABILITY OF OPERATION

The FSL and dewatering options would have the greatest ease and reliability of operation. ATAD and composting would represent new processes for the operators. These processes are sensitive to temperature and other conditions and must be monitored closely. Failure of the composting process would not represent a major problem; sludge could still be applied to land, subject to Class B restrictions. An upset to the ATAD process would represent a major problem because the treatment plant would then be without any sludge stabilization process. Significant odors could result at the plant.

COMPLEXITY

ATAD and composting are more complex than simple dewatering or an FSL. However, ATAD is no more complex than anaerobic digestion, which is currently in use at the treatment plant. Although composting requires substantial labor and monitoring, it is a fairly simple process. The options are considered equivalent with respect to this criterion.

REGULATORY ASPECTS

The four options are well-known and accepted by the regulatory agencies. All four options are ranked equally with respect to this criterion.

FLEXIBILITY

The FSL option provides the greatest flexibility. An FSL provides about 2 years of storage, during which time the land application program can be developed. Liquid sludge can generally be applied to more types of sites than dewatered sludge. Furthermore, if sites which require dewatered sludge are obtained in the future, dewatering can be added at the FSL site. On the other hand, if dewatering is provided now and sites which accept dewatered sludge are not available, the cost of the dewatering facility cannot be recovered. Composting provides some added flexibility in that the product is more desirable and could be given to the public or used on city property in town.

AESTHETICS

ATAD would have the greatest aesthetic impact as a result of the odor potential at the treatment plant. Although odor control would be included, the potential for occasional problems exists as a result of process upset, odor control equipment failure, and exposure of sludge during transfer operations. Furthermore, the finished sludge would produce more odor than would sludge from the other options. An FSL would have an aesthetic impact, but it is assumed that the site would be remote and would have sufficient buffer to minimize the effect. Odor from an FSL is generally rather faint and musty. The aerobic layer on top of the lagoon prevents foul odors from escaping. Storage of dewatered solids would have a similar impact, but of a lesser extent. Composting could produce substantial odor. Remoteness of the site would be most crucial to this option.

ECONOMICS

The budgetary cost estimates for the four options were presented above and summarized in Table 8-7.

The FSL option represents considerable savings in capital cost. The cost is about 20 percent lower than that of ATAD, and 25 percent lower than the other options. The annual cost is lower than for all except dewatering. As discussed earlier in this chapter, the annual cost is sensitive to hauling distance. If an application site were located adjacent to the FSL, the annual cost would be less than for dewatering. Hauling distance would have to exceed 60 miles before the total present worth cost of the FSL would exceed that of the dewatering option.

SELECTION OF RECOMMENDED OPTION

The rankings of the options with respect to the criteria discussed above are summarized in Table 8-8. Criteria for which all three alternatives are considered equal are not included in the table. As discussed above, the FSL option has a substantially lower cost than the other options. Unless there were other overriding factors, cost alone would be a sufficient basis for recommending the

FSL option. In addition, the FSL is ranked highest in several other categories, and has no ranking lower than second. If sludge hauling distance were greater than 60 miles, dewatering would be more economical than an FSL and could be recommended. However, based on initial investigations, it appears that obtaining suitable land much closer to Florence is realistic. Therefore, the recommended option is an FSL.

Selecting the recommended option does not preclude the city from pursuing an additional option such as composting for producing Class A biosolids. A pilot composting operation could be set up at the FSL site. The operation could be expanded or eliminated depending on the results of the pilot study. The FSL would provide sufficient storage to allow the city to defer the purchase of additional land for sludge application for about 2 years. This would give the city time to evaluate the success of the Class A biosolids program before additional land were purchased for sludge application.

Table 8-8. Summary of Solids Handling Option Rankings

Criteria	Alternative ranking			
	ATAD	Dewatering	Composting	FSL
Environmental impact	2	1	1	2
Ease of implementation	3	2	2	1
Reliability	3	1	2	1
Flexibility	3	2	1	1
Aesthetics	3	1	2	2
Economics	3	2	4	1

CHAPTER 9

RECOMMENDED PLAN

Based on the evaluation in Chapter 8, activated sludge was selected as the recommended plan for liquid stream treatment. Anaerobic digestion with facultative sludge lagoon storage of digested sludge was recommended for solids handling. In this chapter, the recommended improvements for the entire wastewater system are summarized and the total costs are presented.

PLANT SCHEMATIC

A schematic diagram of the recommended plan is presented in Figure 9-1. All the major processes include parallel units, which provide redundancy. The flow lines show that digested sludge can be transported directly off-site or it can be thickened prior to removal. It can also be applied directly on land or hauled to the facultative sludge lagoon (FSL) for storage and further stabilization.

SITE PLAN

A proposed site plan for the recommended activated sludge plant is shown in Figure 9-2. An artist's rendition of an oblique aerial view of the site follows in Figure 9-3.

On Figure 9-2, potential future units are shown only to indicate the flexibility in expansion. Whether primary clarifiers, a trickling filter, more aeration basins, or some combination are added will be decided in the future, depending upon factors such as regulatory requirements, economics, and operator preference.

Layout details will be revised and further refined during the design phase, but the locations of the major unit processes are generally fixed. The headworks and aeration basins must be constructed away from the existing unit processes to allow the existing plant to continue to operate during construction. Once those units are completed, the secondary clarifiers can be constructed in the location of the existing aeration pond.

A layout for the FSL site has not yet been developed because it is dependent on the size, shape, and topography of the land. It is important that a suitable site be obtained as soon as possible to allow the design process to start. An artist's rendition of a typical FSL site is shown in Figure 9-4.

DESIGN DATA

The design data for the recommended plan are presented in Table 9-1. The values are those projected for the design year 2020. Proposed future units are not included.

Table 9-1. Design Data For Activated Sludge Plant

Item	Value
Plant flow	
ADWF, million gallons per day (mgd)	1.9
Peak month, mgd	3.6
Peak day, mgd	5.1
PWWF, mgd	6.9
Plant load	
BOD average, ppd	5,300
BOD max month, ppd	7,000
SS average, ppd	3,800
SS max month, ppd	4,800
Influent Pumps	
Type: Self-priming[a]	
Number	3
Capacity each, mgd[b]	1.5
Screen	
Type: Fine-mesh in-channel	
Number	2
Opening size, inches	0.25
Capacity each, mgd	5.3
Emergency bypass bar rack	
Number	1
Opening size, inches	1
Capacity, mgd	6.9
Grit Removal	
Grit chamber: Induced vortex	
Number	2
Diameter, ft	10
Capacity each, mgd	7.0
Grit pump: Recessed impeller	
Grit separation: Cyclone	
Grit dewatering: Auger	
Aeration	
Basins	
Number	2
Width, ft	30
Water depth, ft	15
Length, ft	165

Item	Value
Volume each, 1,000 gallons	555
Operating modes available:	
Plug flow, step feed,	
contact stabilization	
Anaerobic selector	
Process performance[c]	
MLSS, mg/L	2,400
F/M, lb BOD/lb MLVSS/day	0.34
Sludge age, days	4.2
HRT, hours	7.3
Blowers	
Type: Multistage centrifugal[a]	
Number	4
Capacity each, scfm	2,000
Secondary clarifiers	
Type: Flocculator, peripheral weir	
Number	2
Diameter, ft	66
Sidewater depth, ft	17
SOR at peak day, gpd/sq ft	745
SOR at PWWF, gpd/sq ft	1,000
RAS pumping (per clarifier)	
Number of pumps	2
Capacity each, gpm	600
Disinfection	
Type: Closed vessel, medium pressure[a]	
Number of trains	3
Capacity each, mgd	2.3
Lamps per train	8
Outfall	
Length	700
Diameter, inches	24
Diffuser length, ft	200
Number of diffuser ports	50
Sludge thickener (existing)	
Type: Gravity belt	
Number	1
Belt width, m	1
Capacity, lb/hr	800

Item	Value
Thickened sludge tank	
Number	1
Diameter, ft	16
Volume, gallons	22,000
Height, ft	15
Anaerobic digesters	
Type: Mesophilic, fixed submerged cover	
Number	2
Diameter, ft (exist/new)	30/36
Sidewater depth, ft (exist/new)	14/24
Volume, cubic ft (exist/new)	12,070/28,400
SRT at peak month, days	28
Digested sludge holding tank	
Number	1
Diameter, ft	19
Height, ft	15
Volume, gallons	33,000
Odor control biofilter	
Area, sq ft	3,000
Depth, ft	3
Loading rate, cfm/sq ft	2
Air flow rate, cfm	6,000
Facultative sludge lagoon	
Number	1
Area, acres	1.9
Depth, ft	12
Loading, lbVSS/1,000 ft[b]/day	20

Notes:
[a] Equipment type selection is preliminary for cost estimating purposes. Selection may change during predesign.
[b] Influent pump station receives flow from new interceptor only. All other flow is pumped to plant from collection system pump stations.
[c] At maximum month conditions.

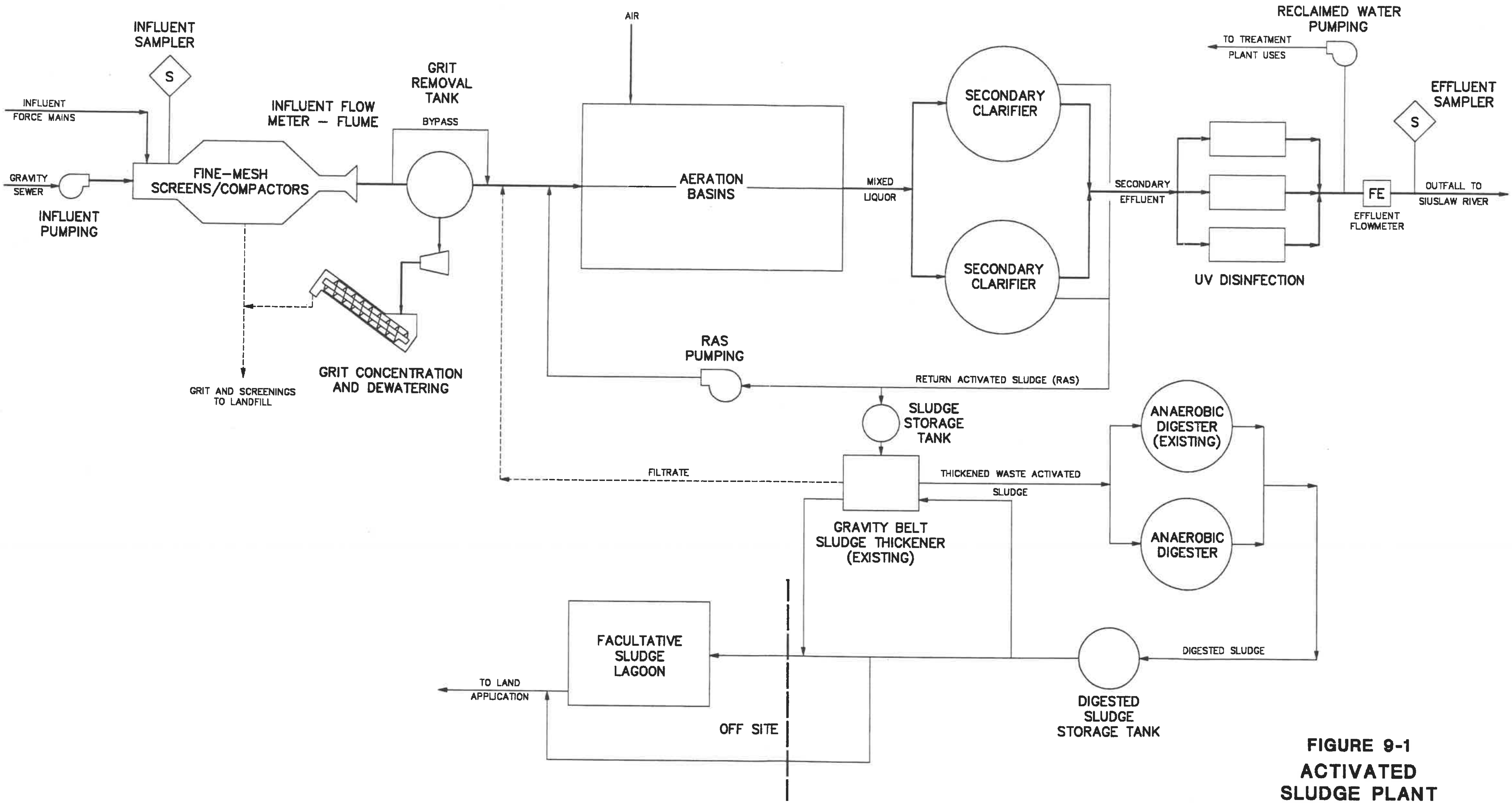

FIGURE 9-1
ACTIVATED SLUDGE PLANT
PLANT SCHEMATIC
NOT TO SCALE

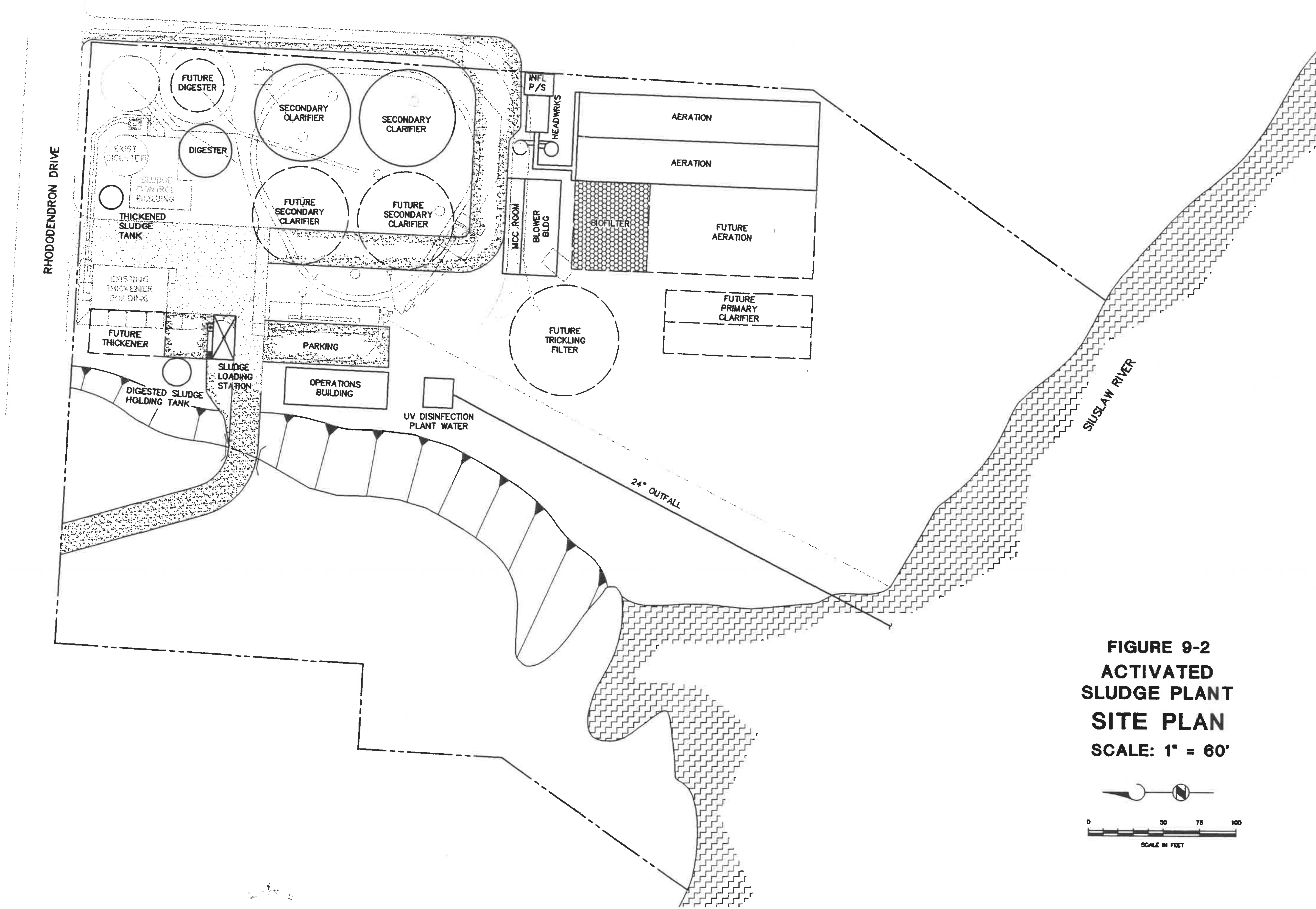

FIGURE 9-2
ACTIVATED SLUDGE PLANT
SITE PLAN
SCALE: 1" = 60'

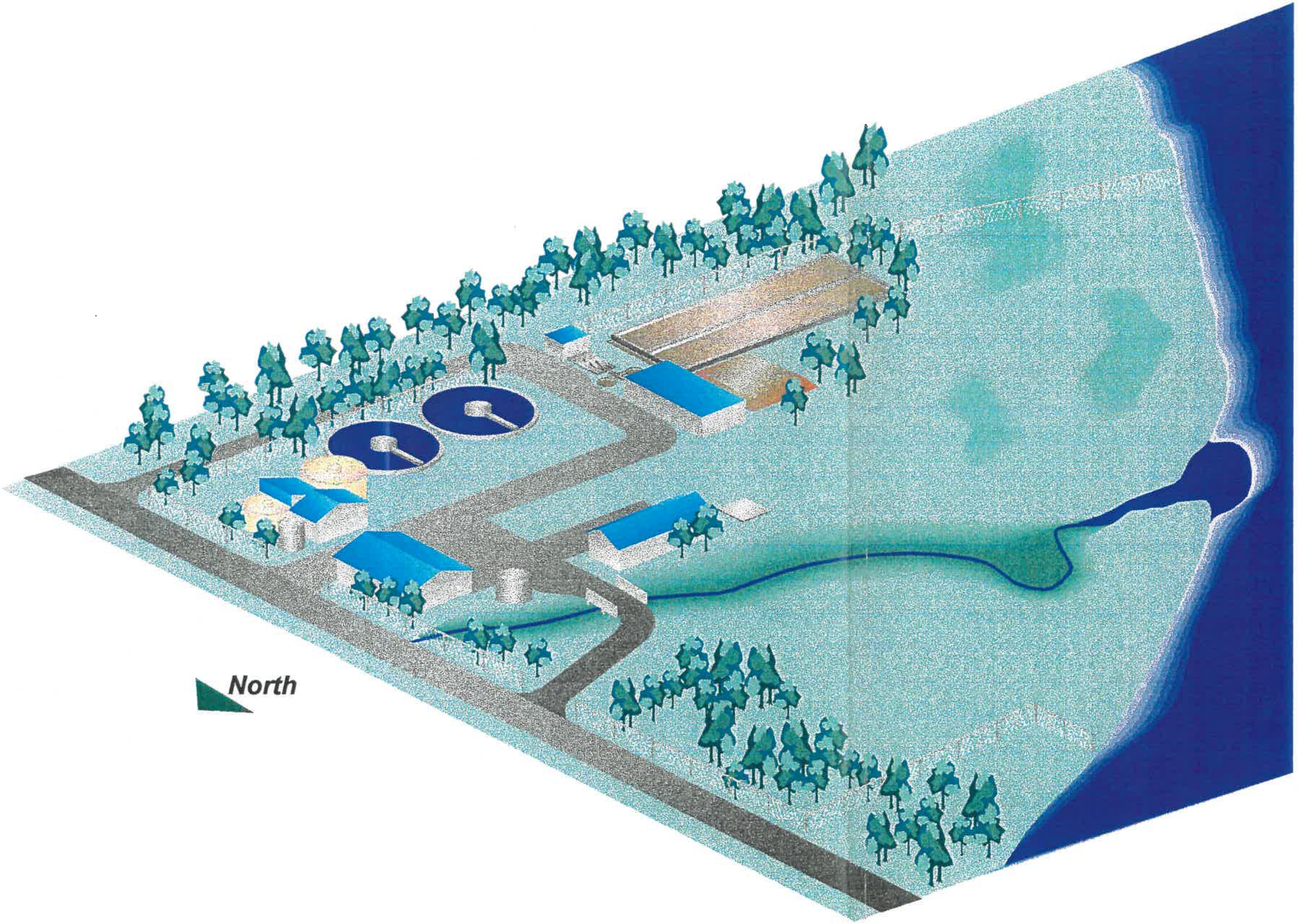

FIGURE 9-3
ARTIST'S RENDITION OF RECOMMENDED PLAN

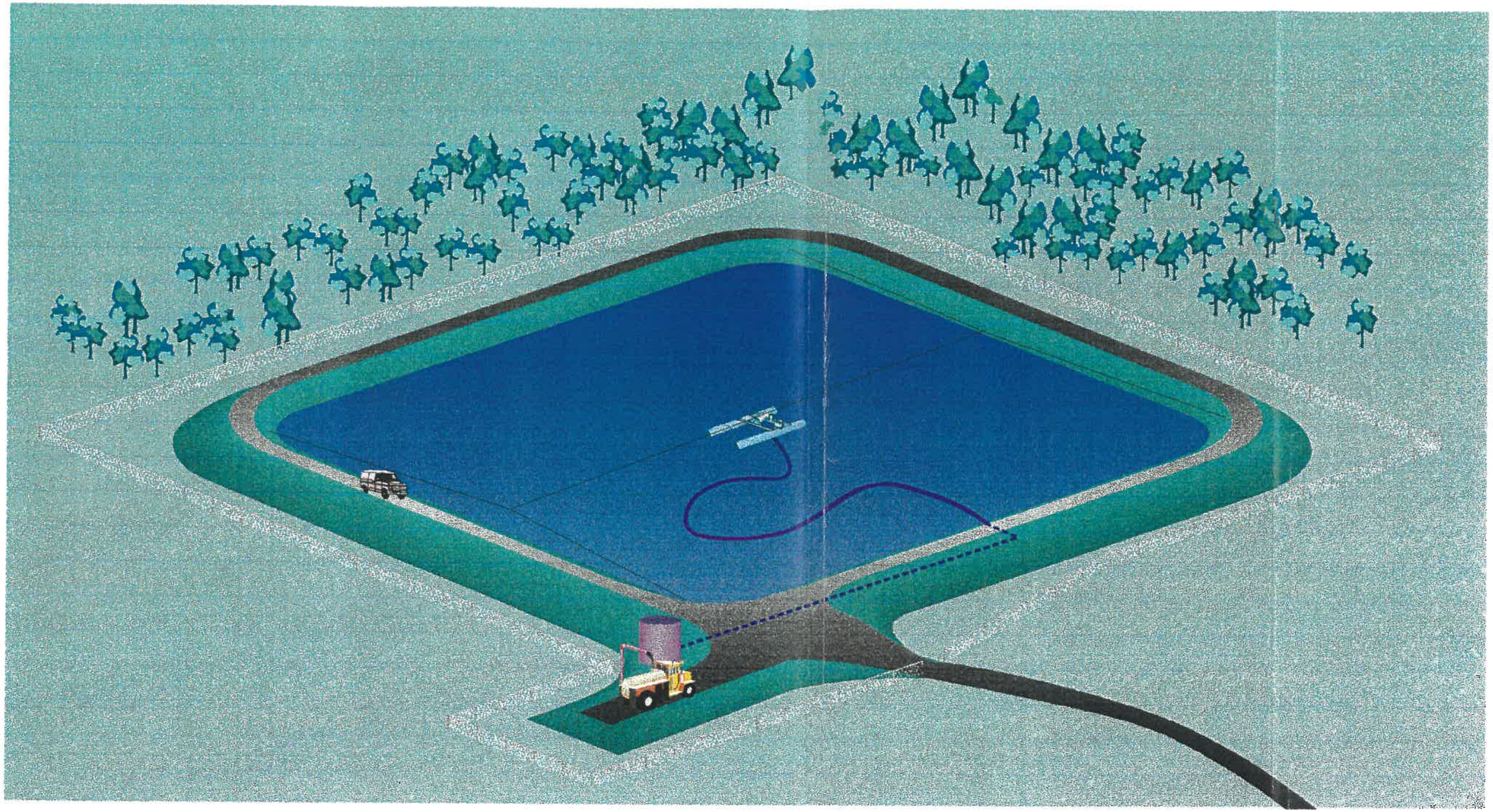

FIGURE 9-4
ARTIST'S RENDITION OF TYPICAL FSL SITE

CAPITAL COST

Table 9-2 below summarizes the total project costs for the liquid stream treatment and solids handling portions of the project, as well as the collection system improvements. Costs for the collection system improvements, developed in Chapter 3, are also summarized here.

Table 9-2. Estimated Capital Costs for Recommended Plan

Item	Costs, $1,000
Liquid stream treatment	
Contractor indirects	469
Influent pumping	368
Yard development	384
Headworks	773
Odor control	237
Aeration basins	866
Blower building	628
Secondary clarifiers	1,381
Yard piping	341
Electrical/instrumentation	1,680
Disinfection	692
Outfall	558
Operations building	287
Subtotal, treatment plant	8,664
Solids Handling	
Anaerobic digestion	1,483
Tank truck	100
FSL	460
Dredge	50
Access road	100
Supernatant irrigation system	50
Subtotal, solids handling	2,243
Collection system	
Gravity interceptor	1,497
Force mains	493
Pump stations	150
Subtotal, collection system	2,140
Subtotal, total project	13,047
Bond at 1 percent	130
Contingency at 15 percent	1,957
Subtotal	15,135
Engineering, admin. at 20 percent	3,027
Subtotal	18,161
Land	450
Total project cost	18,611

PHASING OPPORTUNITIES

Phasing the construction could allow some costs to be deferred to the future. Because phasing incurs costs associated with multiple design and construction contracts, additional mobilization, and loss of economy of scale, an item should be deferred about 10 years to make phasing worthwhile. An exception would be individual pieces of mechanical equipment such as a blower or pump; these items would be worth deferring even a few years.

Components of this project that may have phasing potential are discussed briefly below.

- Collection system. The upper portions of the new interceptor, including the pump stations and force mains will not be necessary until those areas are developed. At this time, only the lower portion, which provides relief to the Ivy Street pump station, is necessary.
- Influent pumping. It may be possible to provide two pumps now and add the third later.
- Aeration. Four blowers will not be necessary for several years. Two or three would be sufficient at first. Likewise, some of the diffusers can be installed in the future. Although the aeration basins will have excess capacity at first, it is unlikely that phasing the construction of the basins would be worthwhile. Adding on to the basins is a major project with significant mobilization costs and potential disruption to plant operation.
- Disinfection. Although the entire structure would be built initially, some of the actual UV modules could be installed later.

COLLECTION SYSTEM IMPROVEMENTS

The flow modeling of the collection system presented in Chapter 3 shows that large sections of the existing system are inadequate to handle the expected future flows. In addition, new development in the northern part of the Urban Growth Boundary will require a major expansion of the collection system in that area. Adding a major interceptor from the northern end of the system to the treatment plant could alleviate the capacity problems within the system and handle the increased flows from newer developments to the north. The proposed interceptor was evaluated using the computer model.

NEW INTERCEPTOR

The route of the interceptor was selected in conjunction with city staff to take advantage of publicly-owned and undeveloped land. Topography was also considered in order to maintain a reasonable slope while minimizing excavation requirements. Modeling was based on the flows the interceptor would need to carry under two conditions:

- Carrying the flow from the newly developed basins in the north.
- Carrying the flow from the new basins in the north plus flow diverted from the eastern portion of the existing system to relieve the overload there. The diversion is located at Oak and 31st Streets.

The design parameters for the proposed interceptor were developed through an iterative modeling process. The detailed parameters for each section of the interceptor are summarized in Appendix A. The length of the interceptor is approximately 5.5 miles. Topography requires that the upper third of the interceptor (Node 6045 to Node 6040) utilize pump stations and pressure mains. These components can be seen on the map of the collection system model in Appendix A. The middle portion of the interceptor (Node 6040 to Node 6060) would consist of an 18-inch gravity main. The remainder of the interceptor (Node 6060 to Node 6085) would consist of a 24-inch gravity main. The capacity of the proposed interceptor and the calculated flows for the maximum flows under buildout conditions are shown in Figure 9-5. Flows are shown with and without the 31st Street diversion. In either case, the interceptor is more than adequate to handle the flows. The figure shows a dramatic increase in capacity downstream of node 6075. This is a result of the steep slope in this area.

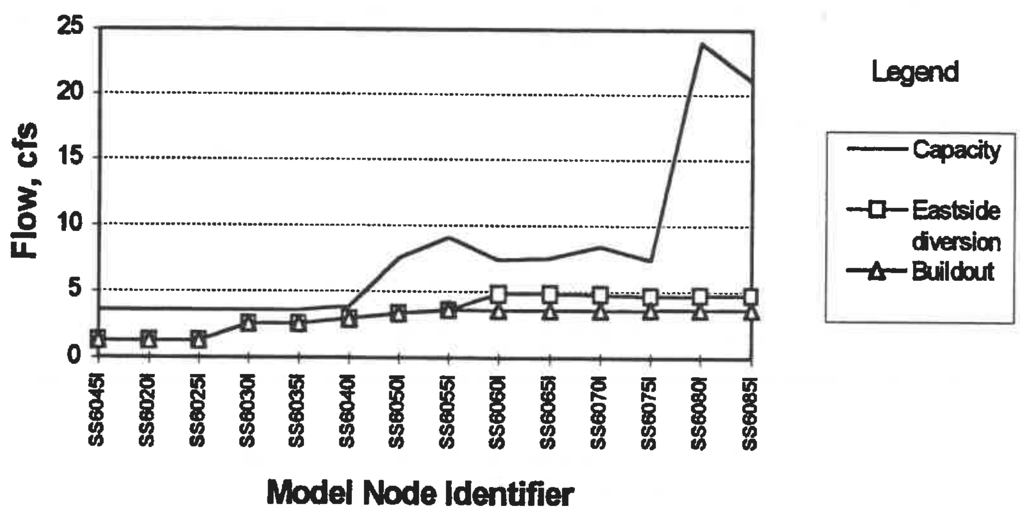

Figure 9-5. Capacity and Flows Through Proposed Interceptor

Although an 18-inch interceptor may be adequate to handle the projected flows, a 24-inch pipeline is recommended in the lower section to ensure that the interceptor does not become overloaded in the future. Because the life span of plastic sewer pipe can be as much as 100 years, the interceptor will continue to be in service far beyond the 20-year planning period. Consequently, it is recommended that the pipe be sized for the maximum flow should growth continue at the current rate over the long term. This will avoid the expensive and disruptive process of sewer replacement in city streets 20 or 30 years from now. The additional cost of providing 24-inch pipe instead of 18-inch pipe is much less than the cost of constructing a new sewer. The cost differential is conservatively estimated at about 35 dollars per foot, for a total of about $400,000.

PUMP STATIONS FOR NEW INTERCEPTOR

As discussed previously, the topography in the area of the upper portion of the interceptor makes construction of a gravity sewer impractical. Consequently, two pump stations are required in this area. These would probably be duplex self-priming stations similar to most of the others in the collection system. The capacity of each station would be about 1.3 cubic feet per second (cfs), or 0.84 mgd.

Because the interceptor would enter the treatment plant as a gravity sewer, a lift station would be required to pump the wastewater up into the headworks. As shown on Figure 9-5, the maximum flow entering the pump station would be about 4.7 cfs, or 3 mgd. The pump station would have three variable-speed pumps. The capacity would be 3 mgd with one pump out of service. It is estimated that the sewer would be about 10 feet deep at the plant, allowing the use of self-priming pumps. If, during detailed design, it is found that substantially greater depth is required, self-priming pumps may not be practical. In this case, submersible or vertical turbine solids handling pumps could be used.

COSTS

There are several components in the proposed interceptor project. These include gravity sewers, pressure mains, and pump stations. Budgetary unit costs for these components are presented in Table 9-3.

Table 9-3. Estimated Unit Costs for Interceptor Components

Item	Unit cost
Duplex package pump station at upper end	$100,000
Pressure main under pavement	$55/LF
18-inch sewer, no pavement	$50/LF
24-inch sewer, no pavement	$85/LF
24-inch sewer under pavement	$100/LF

Based on these unit costs, the entire interceptor project is estimated to cost approximately $3 million, including construction contingency and costs for engineering and contract administration. These costs do not include the influent pump station at the treatment plant; it is included in the estimate for the treatment plant improvements.

SCHEDULE

The improvements described above represent the ultimate facility required under buildout conditions. However, the pump stations and piping in the northern portion of the UGB will not be necessary until development takes place in that area and it is decided to provide wastewater service to the area. As discussed in Chapter 3, the collection system is not currently overloaded, with the exception of the Ivy Street pump station. Additional modeling should be performed based on expected scenarios of near-term development to determine when various portions of the interceptor will be necessary. Because of the overloaded condition of the Ivy Street pump station, a small portion of the interceptor should be constructed as soon as possible to relieve flows to that pump station. Details of this portion of the project are discussed below.

INTERIM PROJECT

Because the Ivy Street pump station is overloaded, resulting in bypassing of wastewater during wintertime high flows, an interim project has been identified to eliminate further bypasses as soon as possible. This project could be completed on a faster schedule than the overall facility improvements, targeting completion before high flows are experienced in the winter of 1998.

The interim improvements would include a temporary pump station and pressure sewer at 8th Street to divert flow from the Ivy Street pump station. A section of the proposed interceptor would be constructed from the pressure sewer discharge on 8th Street to the treatment plant, a distance of about 1200 feet. Additionally, the influent pump station at the treatment plant would be constructed. The only portions of these improvements that would not be retained in the long-term improvements are the 8th Street pump station and pressure sewer. However, the pump station could be relocated to be used elsewhere in the collection system. The cost of these collection system improvements (excluding the influent pump station) is estimated to be about $250,000. As shown in Table 9-2, the cost of the influent pump station is estimated at about $370,000 plus contingency, administration, and engineering costs.

APPENDIX A

COLLECTION SYSTEM MODEL SUMMARY

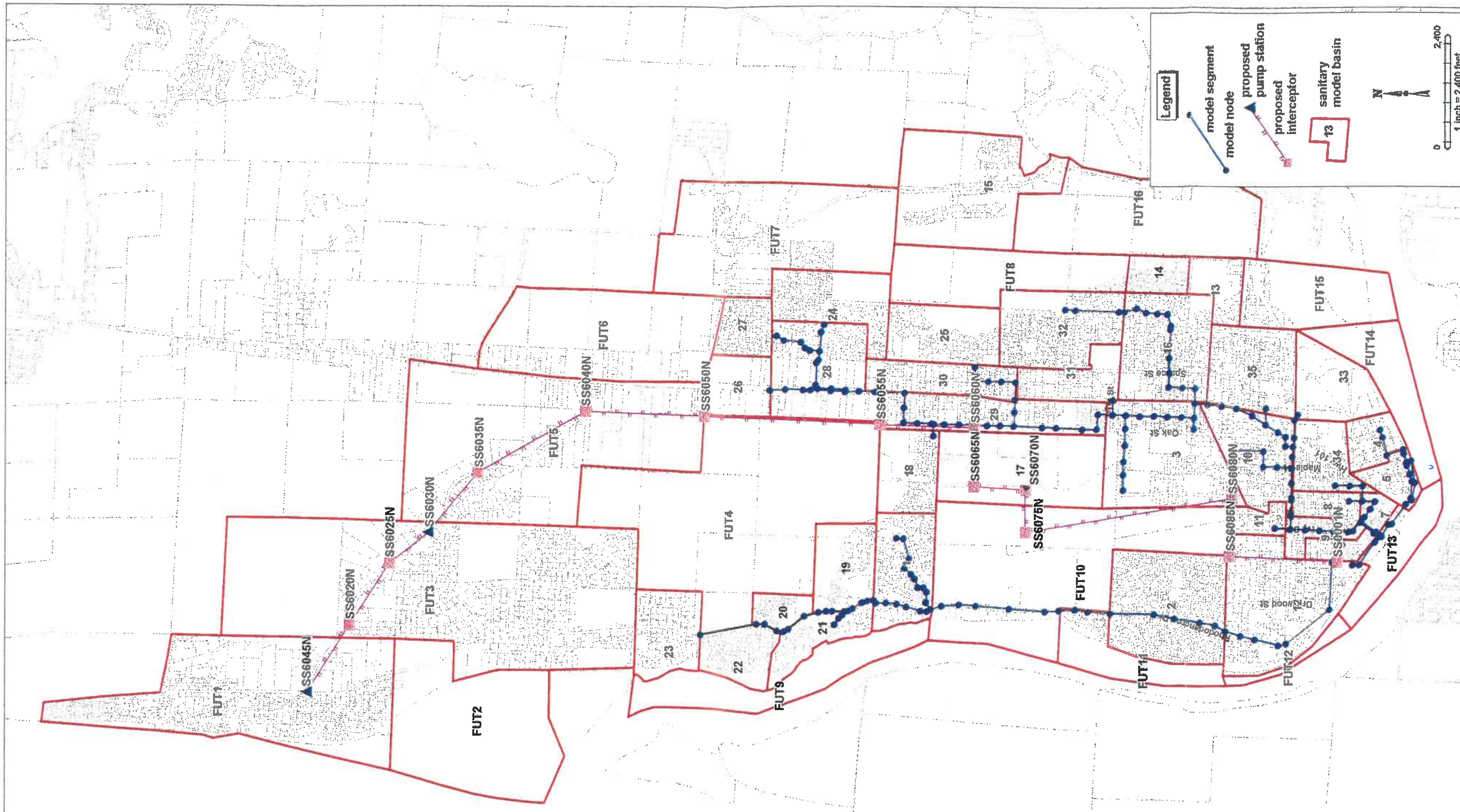

Figure A-1
Florence Collection System Model Map

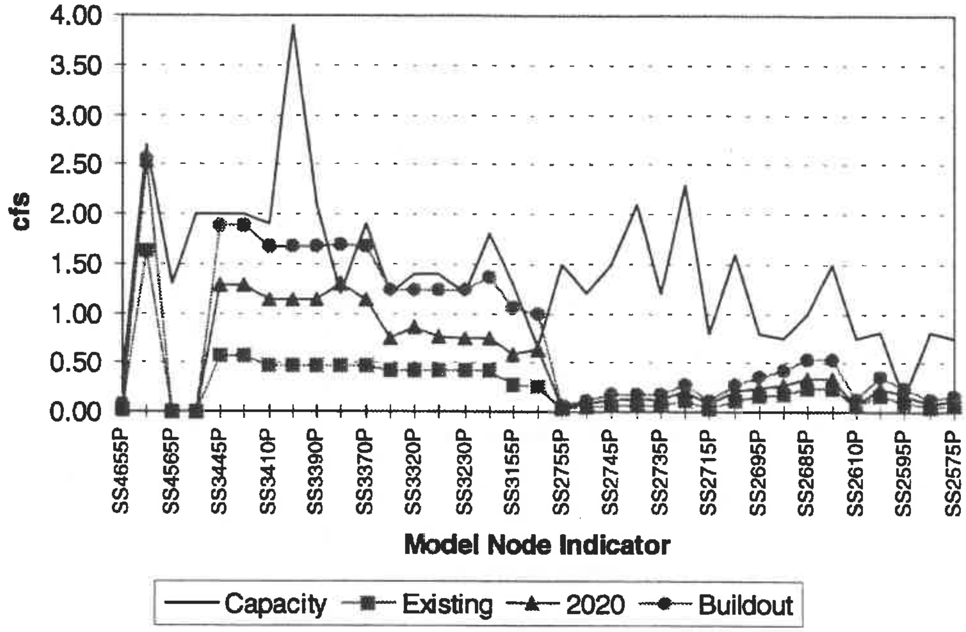

Figure A-2. Collection System Capacity and Wet Weather Flows, Nodes 4655-2575

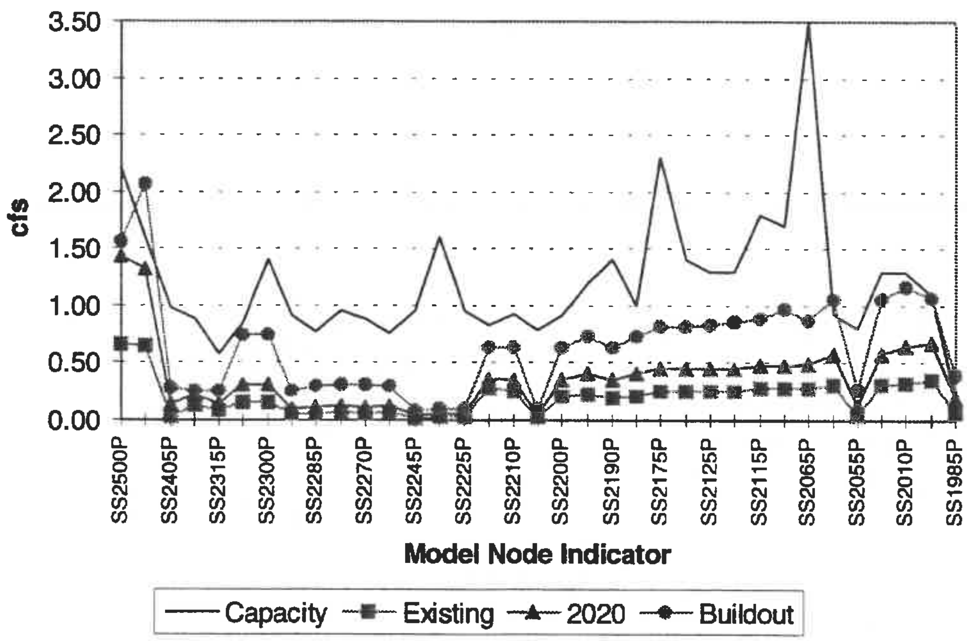

Figure A-3. Collection System Capacity and Wet Weather Flows, Nodes 2500-1985

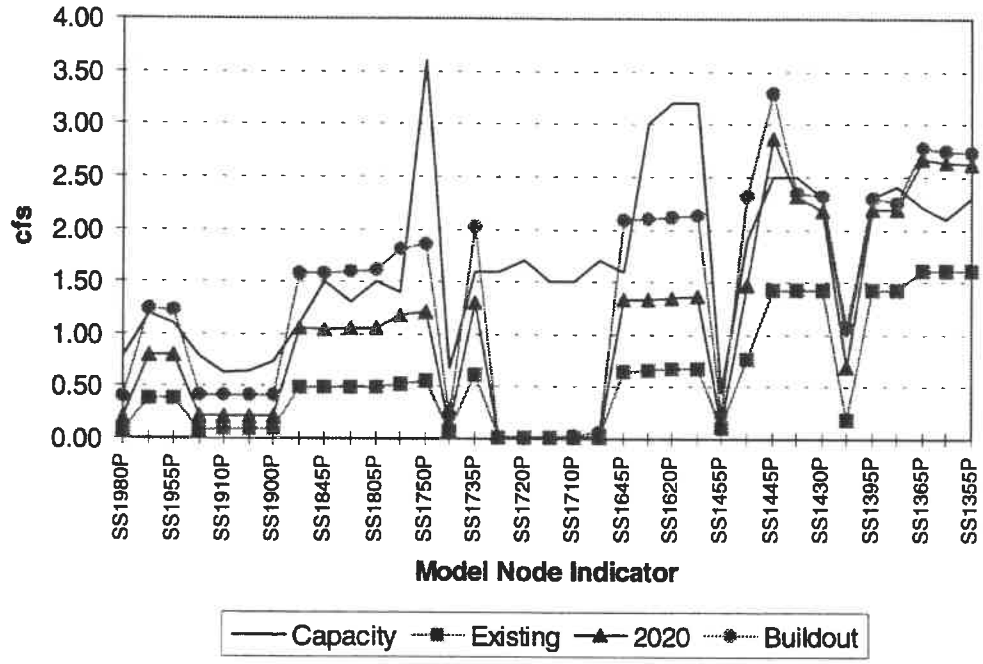

Figure A-4. Collection System Capacity and Wet Weather Flows, Nodes 1980-1355

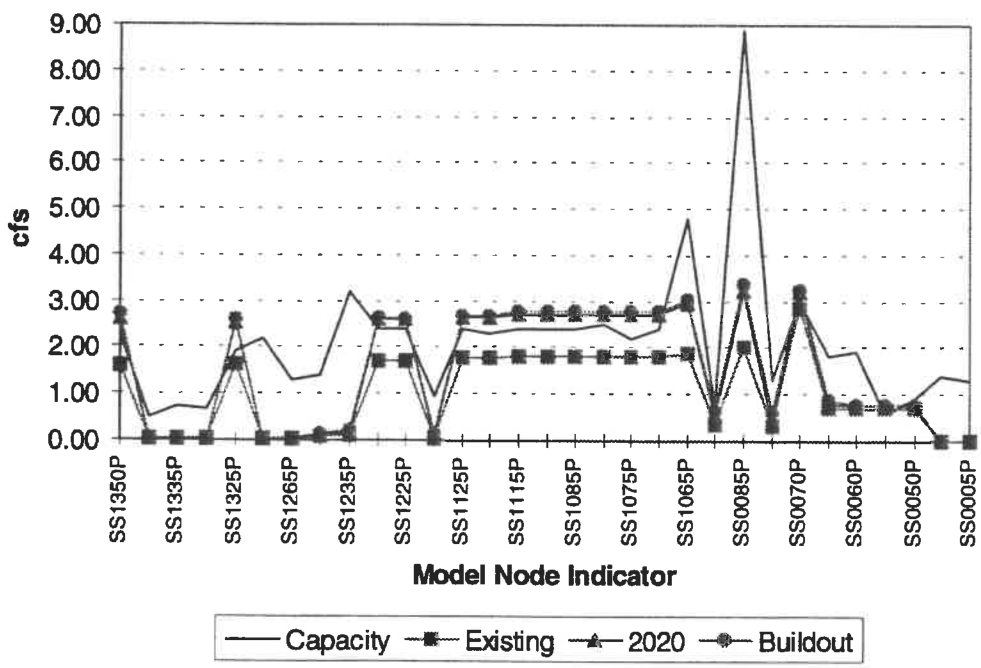

Figure A-5. Collection System Capacity and Wet Weather Flows, Nodes 1350-0005

Table A-1. Interceptor Design Parameters

Upstream Node	Upstream Ground Elevation (ft)	Upstream Ground Cover (ft)	Upstream Invert Elevation (ft)	Pipe Length (ft)	Slope (ft/ft)	Pipe Diameter (inches)	Mannings n	Design Capacity (cfs)	Design Velocity (fps)
SS6045N	106.4	5	99.9	1769	0.0012	18	0.013	3.64	2.1
SS6020N	104.3	5	97.8	1640	0.0012	18	0.013	3.64	2.1
SS6025N	102.3	5	95.8	1100	0.0012	18	0.013	3.64	2.1
SS6030N	101.0	5	94.5	1733	0.0012	18	0.013	3.64	2.1
SS6035N	98.9	5	92.4	2745	0.0012	18	0.013	3.64	2.1
SS6040N	95.6	5	89.1	2644	0.0014	18	0.013	3.87	2.2
SS6050N	92.0	5	85.5	3942	0.0051	18	0.013	7.48	4.2
SS6055N	72.0	5	65.5	2110	0.0076	18	0.013	9.14	5.2
SS6060N	56.0	4.5	49.5	1304	0.0011	24	0.013	7.41	2.4
SS6065N	59.6	9.5	48.1	1171	0.0011	24	0.013	7.54	2.4
SS6070N	54.8	6	46.8	937	0.0014	24	0.013	8.42	2.7
SS6075N	56.0	8.5	45.5	4689	0.0011	24	0.013	7.38	2.4
SS6080N	53.0	10.5	40.5	1286	0.0113	24	0.013	24.01	7.6
SS6085N	38.0	10	26.0	2439	0.0086	24	0.013	20.99	6.7

APPENDIX B

DISCHARGE PERMIT AND MUTUAL ORDER AND AGREEMENT

Expiration Date: 7-31-97
Permit Number: 100934
File Number: 30058

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
811 S.W. Sixth Avenue, Portland, OR 97204
Portland, OR 97204
Telephone: (503) 229-5696

Issued pursuant to ORS 468B.050 and The Federal Clean Water Act

ISSUED TO:

City of Florence
P.O. Box 340
Florence, OR 97439

PLANT TYPE AND LOCATION:

Activated Sludge
Rhododendron Drive
Florence, OR 97439

Treatment System Class: II
Collection System class: II

EPA REFERENCE NO: OR-002074-5

SOURCES COVERED BY THIS PERMIT:

Type of Waste	Outfall Number	Outfall Location
Domestic Sewage	001	R.M. 4.1

RECEIVING SYSTEM INFORMATION:

Basin: Mid Coast
Sub-Basin: Siuslaw
Stream: Siuslaw River
Hydro Code: 12C-SIUS 4.1 D
County: Lane

Issued in response to Application No. 998778 received January 24, 1989.

This permit is issued based on the land use findings in the permit record.

Lydia R. Taylor
Lydia R. Taylor, Administrator

JUL 14 1992
Date

PERMITTED ACTIVITIES

Until this permit expires or is modified or revoked, the permittee is authorized to construct, install, modify, or operate a wastewater collection, treatment, control and disposal system and discharge to public waters adequately treated wastewaters only from the authorized discharge point or points established in Schedule A and only in conformance with all the requirements, limitations, and conditions set forth in the attached schedules as follows:

Each other direct and indirect discharge to public waters is prohibited.

This permit does not relieve the permittee from responsibility for compliance with any other applicable federal, state, or local law, rule, standard, ordinance, order, judgment, or decree.

SCHEDULE A

1. Waste Discharge Limitations not to be Exceeded After Permit Issuance.

a. Outfall Number 001 (Sewage Treatment Plant Discharge)

(1) May 1 - October 31:

Parameter	Average Effluent Concentrations Monthly	Weekly	Mass Load Limits * Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
BOD_5	20 mg/l	30 mg/l	125	188	250
TSS	20 mg/l	30 mg/l	125	188	250
FC per 100 ml	200	400			

(2) November 1 - April 30:

Parameter	Average Effluent Concentrations Monthly	Weekly	Mass Load Limits * Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
BOD_5	30 mg/l	45 mg/l	188	281	376
TSS	30 mg/l	45 mg/l	188	281	376
FC per 100 ml	200	400			

(3)

Other Parameters (year-round)	Limitations
pH	Shall be within the range 6.0-9.0.
BOD_5 and TSS Removal Efficiency	Shall not be less than 85 percent monthly average.

* Mass load limits based on the average dry weather design flow to the facility of 0.75 MGD.

(4) Notwithstanding the effluent limitations established by this permit, no wastes shall be discharged and no activities shall be conducted which violate Water Quality Standards as adopted in OAR 340-41-245 except in the defined mixing zone:

That portion of the Siuslaw River within a radius of 100 feet from the point of discharge.

SCHEDULE B

1. Minimum Monitoring and Reporting Requirements.
(unless otherwise approved in writing by the Department)

a. Influent

Item or Parameter	Minimum Frequency	Type of Sample
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab

b. Outfall Number 001 (Discharge from the sewage treatment plant)

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD)	Daily	Measurement
Flow Meter Calibration	Annual	Verification
BOD_5	2/Week	Composite
TSS	2/Week	Composite
pH	3/Week	Grab
Fecal Coliform	Weekly	Grab
Quantity Chlorine Used	Daily	Measurement
Chlorine Residual	Daily	Grab
Average Percent Removed (BOD_5 and TSS)	Monthly	Calculation

c. Sludge Management

Item or Parameter	Minimum Frequency	Type of Sample
Sludge analysis including: Total solids (% dry wt.) Volatile solids (% dry wt.) Sludge nitrogen NH_3-N; NO_3-N; & TKN-N (% dry wt.) Phosphorus-P (% dry wt.) Potassium (% dry wt.) Sludge metals content for Cd, Cu, Ni, Pb, & Zn (mg/kg) pH (standard units)	Annually	Composite sample to be representative of the product to be land applied from the digester withdrawal line. (See note 1/)

Record of % volatile solids reduction accomplished through digestion	Monthly	Calculation (See note 2/)
Record of locations where sludge is applied on land (Site location map to be maintained at treatment facility for review upon request by DEQ)	Each occurrence	Date, volume & locations where sludges were applied recorded on site location map.

Notes:

1/ Composite samples from the digester withdrawal line shall consist of at least 6 aliquots of equal volume collected over a 24 hour period and combined.

2/ Calculation of the % volatile solids reduction is to be based on comparison of a representative grab sample of total and volatile solids entering the digester (a weighted blend of the primary and secondary clarifier solids) and a representative composite sample of sludge solids exiting the digester withdrawal line (as defined in note 1/ above).

2. Reporting Procedures

Monitoring results shall be reported on approved forms. The reporting period is the calendar month. Reports must be submitted to the Department by the 15th day of the following month.

State monitoring reports shall identify the name, certificate classification and grade level of each principal operator designated by the permittee as responsible for supervising the wastewater collection and treatment systems during the reporting period. Monitoring reports shall also identify each system classification as found on page one of this permit.

Monitoring reports shall also include a record of the quantity and method of use of all sludge removed from the treatment facility and a record of all applicable equipment breakdowns and bypassing.

SCHEDULE C

Compliance Schedules and Conditions

1. By no later than August 31, 1992, the permittee shall submit a sludge management plan in accordance with Oregon Administrative Rule 340, Division 50, "Disposal of Sewage Treatment Plant Sludge and Sludge Derived Products Including Septage". Upon approval of the plan by the Department, the plan shall be implemented by the permittee.

2. By no later than December 31, 1992, the permittee shall submit to the Department a report which either identifies known sewage bypass locations and a plan for estimating the frequency, duration and quantity of sewage bypassing treatment, or certifies that there are no bypasses. If known sewage bypass locations are identified, the report shall also provide a schedule to eliminate the bypass(es).

3. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by January 15 of each year which details sewer collection maintenance activities that have been done in the previous year and outlines those activities planned for the following year.

4. The permittee is expected to meet the compliance dates which have been established in this schedule. Either prior to or no later than 14 days following any lapsed compliance date, the permittee shall submit to the Department a notice of compliance or noncompliance with the established schedule. The Director may revise a schedule of compliance if he determines good and valid cause resulting from events over which the permittee has little or no control.

SCHEDULE D

Special Conditions

1. All sludge shall be managed in accordance with a sludge management plan approved by the Department of Environmental Quality. No substantial changes shall be made in sludge management activities which significantly differ from operations specified under the approved plan without the prior written approval of the Department.

2. The permittee shall comply with Oregon Administrative Rules (OAR), Chapter 340, Division 49, "Regulations Pertaining To Certification of Wastewater System Operator Personnel" and accordingly:

   a. The permittee shall have its wastewater system supervised by one or more operators who are certified in a classification and grade level (equal to or greater) that corresponds with the classification (collection and/or treatment) of the system to be supervised as specified on page one of this permit.

      Note: A "supervisor" is defined as the person exercising authority for establishing and executing the specific practice and procedures of operating the system in accordance with the policies of the permittee and requirements of the waste discharge permit. "Supervise" means responsible for the technical operation of a system, which may affect its performance or the quality of the effluent produced. Supervisors are not required to be on-site at all times.

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 2.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified in the proper classification and at grade level I or higher.

   c. The permittee is responsible for ensuring the wastewater system has a properly certified supervisor available at all times to respond on-site at the request of the permittee and to any other operator.

   d. The permittee shall notify the Department of Environmental Quality in writing within thirty (30) days of replacement or redesignation of certified operators responsible for supervising wastewater system operation. The notice shall be filed with the Water Quality Division, Operator Certification Program (see address on page one). This requirement is in addition to the reporting requirements contained under Schedule B of this permit.

3. The permittee shall notify the DEQ Salem Office (phone: 378-8240) of any malfunction so that corrective action can be coordinated between the permittee and the Department.

BEFORE THE ENVIRONMENTAL QUALITY COMMISSION

OF THE STATE OF OREGON

IN THE MATTER OF: CITY OF FLORENCE Permittee,	))))))	MUTUAL AGREEMENT AND ORDER No. WQMW-WR-96-056 LANE COUNTY

WHEREAS:

1. On July 14, 1992, the Department of Environmental Quality (Department or DEQ) issued a National Pollutant Discharge Elimination System (NPDES) Permit Number 100934 (Permit) to the City of Florence (Permittee). The Permit authorizes the Permittee to construct, install, modify or operate wastewater collection, treatment, control and disposal facilities and discharge adequately treated wastewater into the Siuslaw River, waters of the state, in conformance with the requirements, limitations and conditions set forth in the Permit. The Permit expires on July 31, 1997.

2. Condition 1 of Schedule A of the Permit specifies certain wastewater discharge limits for the Permittees facilities. During the time period the Permit has been in effect, the Permittee has not consistently met these discharge limits and probably cannot meet them in the future if the treatment facilities and collection system remain unchanged.

3. On January 2, 1996, the Department issued a Notice of Noncompliance (NON) notifying the Permittee of violations of the Permit. The following violation was cited:

a. Exceeding concentration and mass load limits for Total Suspended Solids (TSS) and the fecal coliform limit. This occurred during a period of heavy rainfall which resulted in sewage bypassing treatment and raw sewage overflows from the Ivy Street pump station. Violations were documented on December 10, 29 and 30, 1995, and from February 6 - 29, 1996.

4. The Permittee's wastewater treatment facility (WWTF) has an average dry weather design flow of 0.75 million gallons per day (MGD). During periods of precipitation, the wastewater collection system receives large amounts of inflow and infiltration (I/I), mostly in the form of infiltration. During these events, flows to the WWTF are typically above 0.9 MGD and have reached

over 1.5 MGD with instantaneous peak flows of 2.5 MGD. These excessive flows result in raw sewage overflows from the Ivy Street pump station, washout of solids from the clarifier, and insufficient detention time of wastewater in the chlorine contact chamber.

5. The WWTF has reached and/or exceeded its hydraulic capacity resulting in the circumstances described in Paragraph 4. The City of Florence is experiencing rapid growth, and any additional connections will increase the hydraulic loading to the WWTF and will likely result in additional violations of the Permit limits and water quality standards in the receiving stream.

6. The Permittee has in the past requested permission to land apply biosolids from the anaerobic digester during periods when runoff from the land application site may occur which is in violation of Permittee's approved sludge management plan required by OAR Chapter 340, Division 50. These requests are made as a result of insufficient storage capacity in the digester. This lack of capacity also causes process control problems with regard to insufficient wasting of solids from the clarifier. The current biosolids handling facilities do not provide enough residence time and are likely inadequate to meet the requirements of 40 CFR 503 and therefore, until an upgrade to the facilities is completed, the requirements may be violated.

7. In accordance with OAR 340-41-445, toxic substances shall not be introduced into waters of the state that exceed in-stream numerical standards or will adversely affect beneficial uses. The Permittee uses chlorine, which is a toxic substance, to disinfect wastewater. The chlorine standard may be exceeded outside the Permittee's mixing zone in the Siuslaw River. Prior to Department approval of any proposed treatment and disposal alternatives, the Permittee will be required to demonstrate that the proposed facilities will meet all discharge standards and will not violate in-stream water quality standards including the chlorine toxicity standard.

8. The Department and the Permittee recognize that until the sewerage facilities are upgraded and the Permittee completes the actions required in this MAO, the Permittee will continue, at times, to violate the effluent limitations of the Permit. The Permittee will also continue to bypass and/or overflow raw or partially treated sewage to the receiving stream.

9. The Permittee is presently capable of treating its effluent so as to meet the following interim effluent limitations unless influent flows are above the design flow of 0.75 MGD in which case sewage may bypass partial treatment and there may be overflows from the Ivy Street pump station:

Outfall Number 001

A. (1) Year Round

Parameter	Average Effluent Concentrations Monthly	Average Effluent Concentrations Weekly	Effluent Loadings Monthly Average lb/day	Effluent Loadings Weekly Average lb/day	Effluent Loadings Daily Maximum lbs
BOD_5 -	30	45	188	281	376
TSS	30	45	188	281	376

B. During those times when the daily flow exceeds 0.75 MGD, the daily maximum Biochemical Oxygen Demand (BOD) and TSS mass load limitations shall not apply. The WWTF shall be operated as effectively as possible during those times. Also, during those occurrences, the BOD/TSS concentration values obtained for that day will not be used in calculating the monthly average or weekly average effluent concentrations or BOD and TSS percent removal efficiency; and the daily maximum mass load value obtained for that day will not be used for calculating the monthly average or weekly average effluent mass loadings. The fecal coliform colonies per 100 milliliters (ml) results obtained for that day will not be used in calculating the monthly geometric mean or weekly geometric mean.

C. During those times when flows to the Ivy Street pump station exceed 1.0 MGD, overflows of raw sewage will be allowed from the pump station into the Siuslaw River in accordance with the Notification and Response Plan and requirements referred to in Paragraph 11.A(1) and 11.A(2).

10. The Department and Permittee further recognize that the Environmental Quality Commission has the power to impose a civil penalty and to issue an abatement order for violations of conditions of the Permit. Therefore, pursuant to ORS 183.415(5), the Department and Permittee wish to resolve the past and future violations referred to in Paragraphs 2 - 8 by this MAO. This MAO is not

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

NOW THEREFORE, it is stipulated and agreed that:

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

A. Requiring Permittee to comply with the following schedule:

(1) By no later than 30 days after this MAO is signed, the City shall post a sign at the location of the Ivy Street pump station outfall informing the public that raw sewage overflows occasionally occur into the Siuslaw River at that point during the winter. The sign shall remain posted until the City achieves compliance with the Permit.

(2) By no later than 90 days after this MAO is signed, the Permittee shall submit to DEQ for approval a draft Notification and Response Plan describing procedures for notification of the Department and the public for overflows, bypasses and other plant malfunctions. Within 30 days of receiving DEQ comments on the draft, the Permittee shall submit the final Notification and Response Plan for approval. The Permittee shall implement the Plan upon approval. The Plan should include procedures for notifying the public during periods when untreated sewage is discharged. At a minimum, this shall include notifying local radio stations and the nearest newspaper with general circulation of the amount of days that raw sewage was bypassed each month and the gallonage on each day. It shall also contain provisions for posting of the Ivy Street Pump Station, and the overflow location on the Siuslaw River. Sample collection procedures upstream and downstream of the overflow point and sewage treatment plant shall be outlined.

(3) By no later than 3 (three) months after this MAO is signed, Permittee shall retain a consultant to prepare the proposed draft facilities plan report (FPR) for wastewater treatment plant upgrades.

(4) 4/9 By no later than 9 (nine) months after retaining a consultant, Permittee shall submit a draft facilities plan (FPR) report for upgrading the existing WWTF. The FPR should include an evaluation of sewage collection, treatment and disposal system alternatives for complying with minimum federal secondary treatment standards; all appropriate surface water quality standards,

(as specified in OAR Chapter 340, Division 41, Table 20); DEQ minimum design criteria, (as specified in OAR 340-41-455 (1)(a)); groundwater quality protection regulations, (as specified in OAR 340-40-030); and applicable biosolids regulations listed in 40 CFR 503, and OAR Chapter 340, Division 50. The FPR shall include an evaluation of the mixing zone to demonstrate that all permit limits and water quality standards can be met at the existing outfall location. The evaluation of alternatives shall also include a cost-effective I/I analysis.

(5) By no later than 3 (three) months after the Department provides written comments on the draft FPR, the Permittee shall submit an approvable final FPR.

(6) By no later than 6 (six) months following Department approval of the FPR, Permittee shall submit a preliminary design report.

(7) By no later than 6 (six) months after Department approval of the preliminary design report, the Permittee shall submit for DEQ approval draft Plans and Specifications for upgrading/expanding the WWTF and/or completion of all cost-effective I/I work identified in the approved FPR.

(8) By no later than 3 (three) months after Department provides written comments on the plans and specifications, Permittee shall submit approvable engineering plans and specifications for construction of necessary improvements.

(9) By no later than 6 (six) months after approval of the plans and specifications, Permittee shall award construction contracts for completion of necessary improvements.

(10) By no later than 16 (sixteen) months following award of the construction contract, the Permittee shall complete the necessary upgrades/expansion to the WWTF and any required work on the collection system.

(11) By no later than 3 (three) months after completion of facility upgrades, the Permittee shall attain operational level to comply with all established Permit waste discharge limitations and all water quality standards.

B. Requiring Permittee to meet the interim effluent limitations set forth in Paragraph 9.A above until completion of necessary corrective actions as required by the schedule specified in

Paragraph 11.A. The WWTF shall be operated as effectively as practicable to minimize the discharge of pollutants.

C. Requiring Permittee, upon receipt of a written notice from the Department for any violations of the MAO, to pay the following civil penalties:

(1) $250 for each day of each violation of the compliance schedule referred to in Paragraph 11.A.

(2) $500 for each violation of an interim monthly average waste discharge limitation set forth in Paragraph 11.B.

(3) $100 for each violation of each interim weekly average or daily maximum waste discharge limit set forth in Paragraph 11.B and any other condition of this MAO.

12. If any event occurs that is beyond Permittee's reasonable control and that causes or may cause a delay or deviation in performance of the requirements of this MAO, Permittee shall immediately notify the Department verbally of the cause of delay or deviation and its anticipated duration, the measures that have been or will be taken to prevent or minimize the delay or deviation, and the timetable by which Permittee proposes to carry out such measures. Permittee shall confirm in writing this information within five (5) working days of the onset of the event. It is Permittee's responsibility in the written notification to demonstrate to the Department's satisfaction that the delay or deviation has been or will be caused by circumstances beyond the control and despite due diligence of Permittee. If Permittee so demonstrates, the Department shall extend times of performance of related activities under this MAO as appropriate. Circumstances or events beyond Permittee's control include, but are not limited to acts of nature, unforeseen strikes, work stoppages, fires, explosion, riot, sabotage, or war. Increased cost of performance or consultant's failure to provide timely reports may not be considered circumstances beyond Permittee's control.

13. Regarding the violations set forth in Paragraphs 2 - 8 above, which are expressly settled herein without penalty, Permittee and the Department hereby waive any and all of their rights to any and all notices, hearing, judicial review, and to service of a copy of the final MAO herein. The

Department reserves the right to enforce this MAO through appropriate administrative and judicial proceedings.

14. The terms of this MAO may be amended by the mutual agreement of the Department and Permittee.

15. The Department and Permittee may mutually agree to amend the compliance schedule and conditions of this MAO upon finding that such modification is necessary because of changed circumstances or to protect the public health and environment. The Department may amend the compliance schedule and or conditions of the MAO upon the Permittee's repeated failure or refusal to comply with the terms and conditions of the MAO. The Department shall provide the Permittee a minimum of thirty days written notice prior to issuing an Amended Order modifying any compliance schedules or conditions. If the Permittee contests the Amended Order, the applicable procedures for conduct of contested cases in such matters shall apply.

16. This MAO shall be binding on the parties and their respective successors, agents, and assigns. The undersigned representative of each party certifies that he or she is fully authorized to execute and bind such party to this MAO. No change in ownership or corporate or partnership status relating to the facility shall in any way alter Permittee's obligations under this MAO, unless otherwise approved in writing by DEQ.

17. Unless otherwise directed in writing by the Department, all reports, notices and other communications required under or relating to this MAO should be directed to Julie Berndt, DEQ Western Regional Office, 1102 Lincoln Street, Eugene, Oregon 97401; phone number (503) 686-7838 ext. 234. Permittee contact is Rick Mumpower, PO Box 340, Florence, Oregon 97439; phone number (503) 997-2611.

18. Permittee acknowledges that it has actual notice of the contents and requirements of the MAO and that failure to fulfill any of the requirements hereof would constitute a violation of this MAO and subject Permittee to payment of civil penalties pursuant to Paragraph 11.C. above.

19. Any stipulated civil penalty imposed pursuant to Paragraph 11.C. shall be due upon written demand. Stipulated civil penalties shall be paid by check or money order made payable to the

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

20. Providing Permittee has paid in full all stipulated civil penalties pursuant to Paragraph 19 above, this MAO shall terminate 60 days after Permittee demonstrates full compliance with the requirements of the schedule set forth in Paragraph 11.A. above.

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CITY OF FLORENCE

4/2/96
Date

Kenneth D. Hobson

City Manager, City of Florence

DEPARTMENT OF ENVIRONMENTAL QUALITY

4/9/96
Date

Langdon Marsh, Director

FINAL ORDER

IT IS SO ORDERED:

ENVIRONMENTAL QUALITY COMMISSION

4/9/96
Date

Langdon Marsh, Director
Department of Environmental Quality
Pursuant to OAR 340-11-136(1)

APPENDIX C

OUTFALL MIXING ZONE STUDY

MEMORANDUM 14-4141.41

March 25, 1997

TO: JOHN HOLROYD, BROWN AND CALDWELL, EUGENE

FROM: MIKE FLANIGAN, BROWN AND CALDWELL, SEATTLE

SUBJECT: FLORENCE, OREGON, WASTEWATER TREATMENT PLANT OUTFALL EVALUATION

SUMMARY

Presented in this memorandum are the results of an outfall evaluation performed for the Florence, Oregon, Wastewater Treatment Plant (WWTP). This outfall evaluation is part of ongoing comprehensive facilities planning effort. Brown and Caldwell has already prepared a series of technical memoranda focused on WWTP equipment and operation, Siuslaw River water quality, and likely regulatory requirements. Data used in this outfall evaluation is drawn primarily from these previous technical memoranda.

We had three objectives for this outfall evaluation. First we sought to gather as much information as possible on the estuarine environment of the lower Siuslaw River. We found that there are only limited data for the upper freshwater reaches of the Siuslaw River, and almost no site-specific data for the lower estuarine portion of the river. Second, we sought to configure an outfall and a terminal diffuser section that would provide good initial mixing without inducing so much hydraulic head loss that effluent pumping would be required. Third, we sought to characterize water quality impacts to a level of detail sufficient for further consideration by the city, the Oregon Department of Environmental Quality (DEQ), and the Brown and Caldwell planning team.

We examined dilution occurring on a rough estuary-wide basis as well as on a more focused basis occurring within a defined regulatory mixing zone. Because of vigorous mixing with relatively large volumes of ocean water, we feel that long-term, estuary-wide accumulation of pollutants within the estuary will not be a problem. We also examined dilution in terms of water quality impacts that might occur within the mixing zone. We think that regulatory limits for ammonia can be met within a mixing zone extending about 210 feet from the diffuser. We estimate that a mixing zone of this size will provide a chronic dilution factor of about 120:1 and an acute dilution factor of about 30:1. In addition, we think that potential effluent limits for chlorine can also be met using conventional chlorination and dechlorination equipment and control schemes.

BACKGROUND

The Florence WWTP is located on the Siuslaw River about 4 miles inland from the mouth of the river on coastal Oregon. The WWTP provides conventional secondary biological treatment. The plant is currently rated for an average dry weather flow (ADWF) capacity of 0.75 million gallons per day (mgd), and has experienced an estimated peak hourly flow rate of about 3 mgd.

Future Discharge Requirements

Future discharge permits for the Florence WWTP will conform to the requirements of Oregon Administrative Rules (OAR) Division 340-41. Specifically, the Florence WWTP must comply with the water quality standards and treatment requirements for discharge to estuarine waters. In addition, special limitations may be applied to the WWTP if the Siuslaw River is found to be water quality limited for certain parameters.

Siuslaw River Water Quality Limitations. As required by Section 303 (d) of the Clean Water Act, the DEQ recently published a list of all streams that do not comply with applicable water quality standards. These waterways are referred to as water quality limited. The Siuslaw River is listed as water quality limited for temperature during the summer months. Discussions with DEQ indicate that this temperature listing will not place limits on future discharges from the WWTP which are more restrictive than those listed in the OAR. However, Florence may be required to participate in the development of a temperature management plan for the Siuslaw River basin.

Discussions with DEQ indicate that the Siuslaw River could be listed as water quality limited for other parameters in the future. The water quality parameters of concern include:

- Dissolved oxygen during the summer. Some past excursions of water quality standards have been noted.
- Habitat modification. More data are needed to determine if stream channelization or alterations to riparian areas is a problem.
- Nutrient and sediment impacts. More data are needed to fully evaluate nutrient and sediment impacts.

It is unclear at this time if the Siuslaw River violates the water quality standards for the above parameters.

Water Quality Parameters. Water quality parameters pertinent to this outfall evaluation are summarized in Table 1. All comments are based on discussions found in OAR 340-41.

Table 1. Summary of Pertinent Water Quality Parameters

Water quality parameter	Comments
Temperature	The Siuslaw River is listed as water quality limited for temperature during the summer. For marine and estuarine waters, no significant increase in temperature above natural background levels is allowed above 0.25 degrees F at the edge of the mixing zone.
Dissolved oxygen	DO concentration in estuaries must be maintained above 6.5 mg/L. DEQ may set more restrictive DO limits in the future if the Siuslaw River is listed as a water quality limited stream.
pH	pH for all fresh and estuarine waters must remain between 6.5 and 8.5.
Bacteria	Bacteria standards are relatively stringent because the WWTP discharges into an estuary containing shellfish-growing areas. The median fecal coliform concentration cannot exceed 14 organisms per 100 mL. In addition, no more than 10 percent of the samples can have more than 43 organisms per 100 mL.
Toxic substances	Toxicity limits for chlorine in marine water are 0.075 mg/L for chronic toxicity and 0.013 mg/L for acute toxicity. Ammonia toxicity is dependent on water temperature, pH, and salinity. Ammonia toxicity can be addressed by converting the ammonia to nitrate in the secondary process through nitrification, by providing adequate mixing of plant effluent and the receiving water, or through a combination of both.

Design WWTP flows expected through year 2020 are presented in Table 2.

Table 2. Expected Future WWTP Flows

Flow condition	Flow, mgd
Average dry weather flow	1.9
Maximum month flow	3.6
Peak day flow	4.3
Peak hour wet weather flow	6.9

Oceanographic Data

Enough stream flow and water quality data are available for the Siuslaw River above the WWTP discharge point to at least roughly assess water quality issues in the upper freshwater reaches of the river (see *Technical Memorandum 4.1, Water Quality Assessment*, prepared by Brown and Caldwell and dated December 18, 1996, for a summary of available stream flow and water quality data). However, the WWTP discharges at the downstream end of the Siuslaw in a region that is strongly estuarine in nature, and for which data are presently scarce.

For instance, tide elevations for Florence can be estimated from the National Oceanic and Atmospheric Administration (NOAA) reference station in Crescent City, California, with enough accuracy for this evaluation. However, corresponding NOAA predictions for tidal current speed are not available. The best estimates of current strength are reported by local fishermen and recreational boaters. Local fishermen claim that maximum current speeds in the vicinity of Florence can approach 4 knots (2 meters per second [m/s]).

Additionally, water quality data which are important to mixing zone studies, such as salinity and temperature, are not well documented. City personnel recently gathered limited salinity and temperature data at and upstream from the WWTP discharge point (see Attachment A). These winter data will not be fully representative of critical summer conditions because a pronounced freshwater layer is present during the winter months. As river flows fall to summer flow rates, the density structure will become less stratified because the estuarine portion of the Siuslaw River will increasingly be filled with fresh ocean water that has been vigorously mixed by strong tidal currents.

Values of ambient temperature and pH will also begin to resemble values for coastal ocean water as summer flow condition begin to establish. We obtained salinity and temperature data for Charleston, Oregon from the University of Oregon Marine Science Laboratory, and have assumed for this analysis that the Charleston data represent the characteristics of ocean water entering the Siuslaw River estuary.

Streamflow data for the Siuslaw River were presented in *Technical Memorandum 4.1*. There is some disagreement between USGS data and that agency's estimate of the 7Q10 summer flow. However, it can be conservatively assumed that the 7Q10 summer flow just upstream from the WWTP is about 75 cubic feet per second (cfs).

HYDRAULIC AND DILUTION ANALYSES

Outfalls must perform well both hydraulically and hydrodynamically. Hydraulic performance can be characterized by total head loss through the outfall and diffuser and by the flow distribution that occurs through the ports along the length of the diffuser. Hydrodynamic performance is defined by the amount of initial dilution that the diffuser can achieve given the limited water column available in the vicinity of the WWTP.

Hydraulic Analysis

One goal of this analysis was to configure an outfall and diffuser which would disrupt existing plant operation as little as possible. The existing WWTP outfall is a shoreline discharge that does not require effluent pumping. We therefore modeled a preliminary outfall and diffuser configuration which offers good initial effluent mixing without requiring effluent pumping.

We set the internal diameter of the outfall and the diffuser section to 24 inches in diameter. The total length of the modeled outfall was about 1,100 feet, of which about 700 feet was placed offshore. This preliminary outfall layout extended from the existing treatment plant to a point just inshore from a navigation beacon located near the edge of the dredged ship channel. We aligned the outfall thus to place the diffuser section as deep as possible without entering the dredged section. Placing the diffuser near to, but inshore from, the navigation beacon will provide additional protection for the diffuser since vessels drawing enough water to damage the diffuser will steer clear of the beacon.

Diffuser hydraulics were examined using DIFF$$.EXE, a proprietary Brown and Caldwell diffuser hydraulic model (see Attachment B). We used a diffuser configuration consisting of 50 identical 2-inch diameter ports placed 4 feet apart. This configuration will impart good initial dilution within the limited water column available for dilution. Further, total head loss through both the diffuser section and outfall pipeline will be about 5.3 feet at 6.9 mgd. Preliminary surveying data supplied by the city indicate that there might be about 10 feet available between mean higher high water (MHHW) and the ground surface elevation at the WWTP. So long as the peak hourly flow rate does not rise appreciably above 6.9 mgd in the future, there should be enough driving head available at the plant to provide gravity flow through the outfall, even at tidal or flood stages higher than MHHW.

Dilution Analysis

Our focus for the dilution analysis was twofold. Our first goal was to roughly characterize the amount of overall dilution that can be achieved within the Siuslaw River estuary over a tidal cycle. The second goal of the dilution analysis was to characterize dilution occurring within a mixing on time scales much shorter than the tidal cycle.

Overall Tidal Dilution. Estimating the volumes of water entering a estuary through freshwater stream flows and through tidal ocean exchange can used to determine if effluent accumulation will be a problem. As note above, little is known about the hydrodynamics of the Siuslaw River estuary. However, a rough calculation can be performed to estimate the volume of ocean water entering the estuary by noting the tidal fluctuations and the extent to which saltwater reaches upstream. As can be seen in Attachment A, salinity begins to drop rapidly about 13,000 feet upstream from the WWTP, or about 6.5 miles upstream from the river jetty. Note that the data in Attachment A were collected during winter conditions. However, the data can be taken as a conservative estimate of saltwater intrusion since during summer conditions saltwater will extend farther upstream.

Additionally, vertical density stratification will be much less in the summer, resulting in higher overall dilution. Winter stratification in the Siuslaw River estuary is caused by a layer of relatively fresh water floating on top of a layer of denser sea water. Stratification tends to reduce dilution because the rising effluent plume becomes trapped beneath the fresh water layer, thereby restricting the extent of vertical mixing that can be achieved. Summertime stratification in deep water can be significant, but this stratification is due more to temperature variations in the water column than salinity differences. The Siuslaw River estuary is shallow, fast-moving, and winding. Turbulence due to friction and bends will mix the estuary well, resulting in only minor stratification.

Additional assumptions must be made to estimate the volume of ocean water entering the estuary. We searched NOAA tide predictions for Florence to determine the smallest predicted tidal elevation change. Using a small tidal elevation change results in a conservative estimate of ocean water entering (and leaving) the estuary. The smallest tidal elevation change we found in our search was 0.9 feet. Applying this elevation change over the lower 6.5 miles of the river and an average estuary width of about 1,100 feet results in a minimum tidal prism of about 38 million cubic feet.

The tidal prism can also be expressed as an average flow rate about 1,700 cfs occurring over a 6-hour tidal cycle. Combining this tidal flow rate with the assumed 7Q10 river flow of 75 cfs results in a dilution of about 930:1 when compared to the ADWF of 1.9 mgd. We assumed for this exercise that none of water leaving the Siuslaw River jetty on ebb tide will return on the subsequent flood tide. We believe this is a reasonable assumption given the jet-like discharge conditions through the jetty and the presence of consistent littoral currents. Given this large tidal-influenced dilution, we have assumed that effluent accumulation in the estuary will not be a significant environmental problem.

Initial Dilution. Typically, the amount of initial dilution that can be reliably achieved is used to address how well an outfall and diffuser perform to protect water quality. Based on the results of a mixing zone analysis, DEQ may grant a mixing zone in which water quality criteria for chronic exposure may be exceeded. The water quality criteria must be met by the time an effluent plume reaches the edge of the mixing zone. DEQ may also grant a zone of initial dilution (ZID) lying within the mixing zone. Acute water quality criteria may be exceeded within the ZID. Mixing zone dimensions are not specifically set forth in the OAR. Instead, DEQ typically requires that the dimensions of a proposed mixing zone be set as small as possible to ensure that water quality criteria are met within the smallest ambient volume reasonable.

The results of computer simulations for chronic and acute conditions are summarized on Figures 1 and 2, respectively. We performed the computer simulations using two EPA-approved effluent dilution models. Acute dilution factors were estimated using PLUMES, while chronic dilution factors were estimated using CORMIX2. We chose PLUMES for estimating acute dilution because this model is far more stable in the highly turbulent environment just outside the diffuser discharge ports. CORMIX2 was used to estimate chronic dilution factors since it is the only EPA-approved dilution model that takes boundary effects such as streambank reflections into account. Because the Siuslaw River estuary is relatively narrow, there is potential for plume interaction with the shore as the plume moves away from the diffuser. Note, however, that CORMIX2 can be as unstable at some distance downstream as it can be close to the point of initial discharge.

Based on direction from DEQ, we conducted both our chronic and acute modeling using an ADWF of 1.9 mgd. We used an iterative approach to determine mixing zone dimensions by first determining initial acute and chronic dilution, and then calculating water quality impacts. We performed a number of computer simulations to determine the dimensions of a proposed mixing zone and a proposed ZID. Chronic dilution factors shown on Figure 1 are for a point about 210 feet from the diffuser, corresponding to a distance 200 feet from the plus the depth of water over the diffuser as measured at mean lower low water (MLLW). Acute dilution factors shown on Figure 2 are predicted values occurring about 21 feet from the diffuser, representing the edge of a ZID extending upstream and downstream 1/10th the length of the overall mixing zone.

For the purposes of this evaluation, we have selected a chronic dilution factor of 120:1 and an acute dilution factor of 30:1. The selected chronic dilution factor corresponds to the estimated dilution achieved when the ambient current is traveling at 1 m/s, or the average of the extreme current speeds of 0 and 2 m/s. Note that we have chosen to pick the chronic dilution factor from a best-fit polynomial line because of the "scatter-gun" instability shown in the CORMIX2 results.

The acute dilution factor was estimated from PLUMES simulations alone (though we have included the dilution factors from of corresponding CORMIX2 simulations to illustrate the relatively high variability of the CORMIX2 predictions close to the diffuser). The selected acute dilution factor corresponds to the estimated dilution achieved when the ambient current is traveling at 0.1 m/s, the ambient current speed we have chosen to represent the 10th percentile current speed for this analysis.

Chronic and acute water quality impacts were assessed as permit limits that DEQ might set for chlorine and ammonia based on available dilution at distances of 21 and 210 feet. Potential permit limits were calculated using a spreadsheet prepared by the Washington State Department of Ecology (Ecology). Ecology developed their spreadsheet on statistical approaches found in the US EPA *Technical Support Document for Water Quality-based Toxics Control* (TSD, 1985). While DEQ may not be familiar with the Ecology spreadsheet, the statistical approaches found in the TSD are the basis of water quality-based toxics control throughout the nation.

Potential permit limits for ammonia are shown on Attachment C. The ammonia permit limits were based on the assumed dilution factors plus conservative estimates of ambient salinity, temperature, and pH. Conservative estimates of background ammonia were also used. The potential permit limits as calculated by the Ecology spreadsheet are about 55 mg/L (as NH_3-N) for a daily maximum limit and about 21 mg/L (as NH_3-N) for a monthly average limit. Both of these effluent limits can be achieved using conventional secondary treatment.

Potential permit limits for chlorine are shown on Attachment D. The chlorine permit limits were based on the assumed dilution factors along with an acute limit for chlorine concentration at the edge of the ZID of 0.013 mg/L and a corresponding chronic limit at the edge of the mixing zone of 0.075 mg/L. To ensure that these criteria are consistently met, DEQ might set a daily maximum limit for chlorine of about 0.39 mg/L. DEQ might set the corresponding monthly average limit at about 0.15 mg/L. These effluent concentrations can be maintained with standard chlorination and dechlorination equipment and control schemes, especially if the city overdoses somewhat with dechlorination agent.

SUMMARY AND RECOMMENDATIONS

Based on our evaluation, we make the following recommendations.

- Construct a new outfall extending about 700 feet offshore from the existing effluent discharge location. Construct a diffuser section consisting of 50 ports, each 2 inches in diameter and spaced 4 feet apart. The diffuser section should be at an average depth of about 10 feet, as measured at MLLW.

- Request that the dimension of the mixing zone be set a no less than 210 feet upstream and downstream form the diffuser. Request that a ZID be granted that extends no less than 21 feet upstream and downstream from the diffuser.

- Collect field data in the estuary during the environmentally-critical months of late summer and early fall. These field data should include current as tide measurements and measurements of water quality parameters such as salinity, temperature, pH. Background concentrations dissolved oxygen concentration and the concentration of toxic substances such as ammonia and metals should also be measured. Close examination of these field data might justify the use of higher acute and chronic dilution factors, which in turn can be used by DEQ to grant higher, more readily achievable permit limits for the plant.

ATTACHMENT A. DENSITY PROFILES FOR FEBRUARY 4, 1997

Distance upstream from WWTP, feet	Time	Depth, feet	Temperature, deg C	Salinity, ppt	Density, σ-t units
@ WWTP	0920	20	10	28	21.53
		1	10	5	3.67
2,700	0930	25	10	26	19.97
		8	10	24	18.42
		1	10	5	3.67
4,800	0940	14	9	23	17.79
8,700	0945	20	9	20	15.45
12,900	0955	20	9	11	8.45
14,400	1000	29	9	2	1.42
@ WWTP	1013	20	9	28	21.69
		8	9	20	15.45
		1	9	6	4.54
@ WWTP	1200	14	10	20	15.32
		1	10	5	3.67

ATTACHMENT B

DIFFUSER HYDRAULIC PERFORMANCE

```
1     florence diff w/ 50 2-in & Qt at 6.9 mgd

          HYDRAULICS FOR A MULTIPORT DIFFUSER
  DENSITY RATIO =       .02212


  DIFFUSER CHARACTERISTICS

   THIS DIFFUSER HAS  1 DISSIMILAR SECTIONS

  PORTS PER     PORT       SECTION       PORT          SECTION
   SECTION    SPACING     DIAMETER    DIAMETER         SLOPE
    50.00000     4.00000    24.00000     2.00000         .01611
```

PORT NUMBER	STATION, FEET	FLOW IN PIPE, MGD	PIPE VEL, FPS	PORT DISCH,MGD	PORT VEL, FPS	DIFFERENTIAL HEAD, FT
1	.0	.14	.07	.14	9.66	1.53
6	20.0	.82	.40	.14	9.68	1.53
11	40.0	1.50	.74	.14	9.70	1.54
16	60.0	2.19	1.08	.14	9.71	1.55
21	80.0	2.87	1.41	.14	9.73	1.57
26	100.0	3.56	1.75	.14	9.75	1.59
31	120.0	4.25	2.09	.14	9.78	1.61
36	140.0	4.94	2.43	.14	9.83	1.65
41	160.0	5.63	2.77	.14	9.89	1.69
46	180.0	6.33	3.12	.14	9.98	1.74
50	196.0	6.90	3.40	.14	10.06	1.79

```
0MINIMUM FROUDE NUMBER =    28.05 AT PORT    1
0TOTAL PORT AREA/OUTFALL AREA =  .347



   SUMMARY OF SYSTEM HYDRAULICS
   WITH TOTAL FLOW OF    10.68 CFS


   COMPONENT            HEAD LOSS,FEET

   DIFFUSER                  1.80
   DENSITY                    .14
   PIPE   FRICTION           3.17
   MINOR LOSSES               .18


   TOTAL                     5.29
```

ATTACHMENT B

DIFFUSER HYDRAULIC PERFORMANCE

```
1     florence diff w/ 50 2-in & Qt at 1.9 mgd

         HYDRAULICS FOR A MULTIPORT DIFFUSER
  DENSITY RATIO =       .02212


  DIFFUSER CHARACTERISTICS

   THIS DIFFUSER HAS  1 DISSIMILAR SECTIONS

  PORTS PER      PORT       SECTION       PORT        SECTION
   SECTION     SPACING     DIAMETER    DIAMETER       SLOPE
   50.00000     4.00000    24.00000     2.00000        .01611
```

PORT NUMBER	STATION, FEET	FLOW IN PIPE, MGD	PIPE VEL, FPS	PORT DISCH,MGD	PORT VEL, FPS	DIFFERENTIAL HEAD, FT
1	.0	.03	.02	.03	2.28	.09
6	20.0	.20	.10	.03	2.37	.09
11	40.0	.37	.18	.03	2.46	.10
16	60.0	.55	.27	.04	2.55	.11
21	80.0	.73	.36	.04	2.63	.11
26	100.0	.92	.45	.04	2.71	.12
31	120.0	1.11	.55	.04	2.79	.13
36	140.0	1.31	.65	.04	2.87	.14
41	160.0	1.52	.75	.04	2.96	.15
46	180.0	1.73	.85	.04	3.04	.16
50	196.0	1.90	.94	.04	3.11	.17

```
0MINIMUM FROUDE NUMBER =    6.62 AT PORT    1
0TOTAL PORT AREA/OUTFALL AREA =  .347


   SUMMARY OF SYSTEM HYDRAULICS
   WITH TOTAL FLOW OF     2.94 CFS


   COMPONENT           HEAD LOSS,FEET

   DIFFUSER                .17
   DENSITY                 .14
   PIPE    FRICTION        .24
   MINOR LOSSES            .01


   TOTAL                   .57
```

Figure 1. Predicted Chronic Dilution

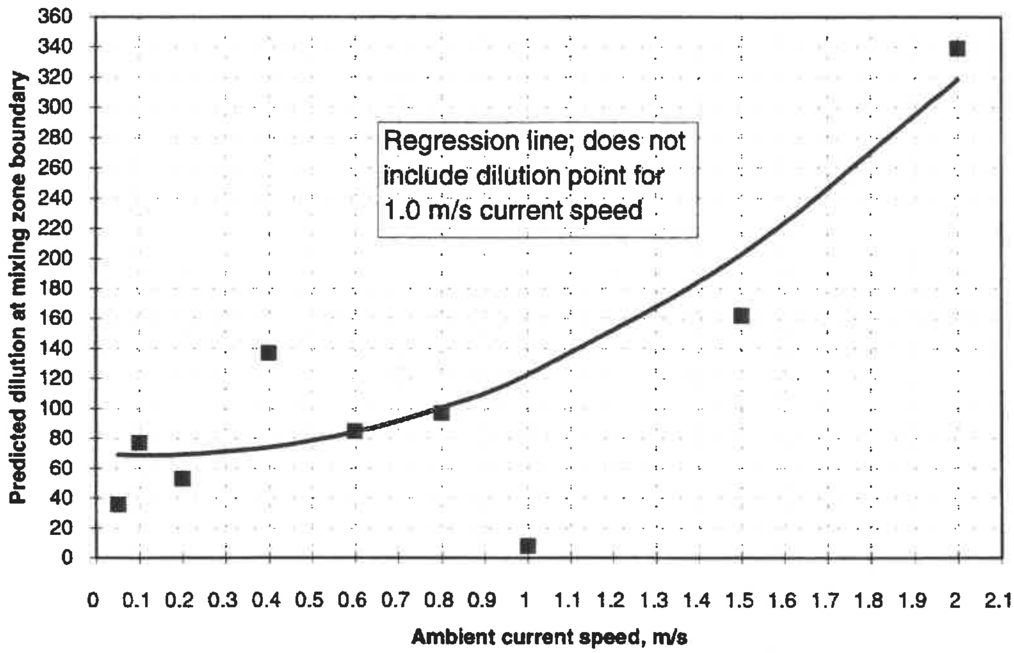
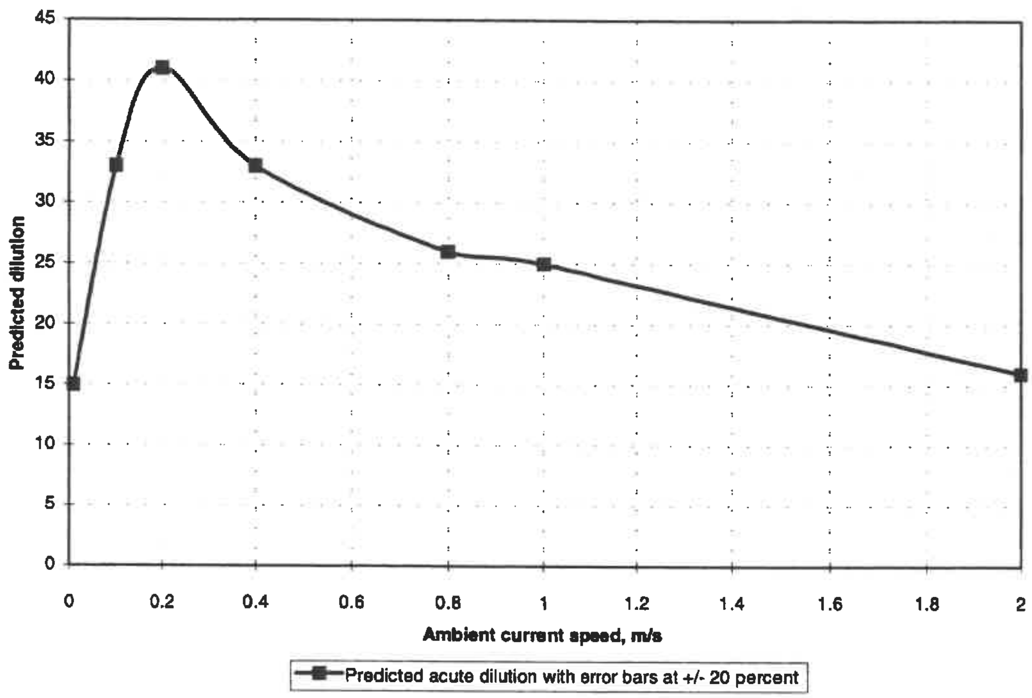


Figure 2. Predicted Acute Dilution



ATTACHMENT C
REPRESENTATIVE PERMIT LIMITS FOR AMMONIA

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

INPUT	
1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	2.515
Chronic (n-day) Criteria:	0.378
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.235
Upstream Concentration for Chronic Condition (7Q10):	0.100
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	20.000
Chronic Receiving Water Dilution Factor at 7Q10:	100.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	30

OUTPUT	
1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	45.836
Chronic (n1-day) WLA:	27.880
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	14.717
LTA for Chronic (n1-day) WLA:	14.705
Most Limiting LTA (minimum of acute and chronic):	14.705
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.0119
Daily Maximum Permit Limit:	45.798
Monthly Average Permit Limit:	17.494

Source: Permit writer's spreadsheet used by the Washington Department of Ecology.

ATTACHMENT D
REPRESENTATIVE PERMIT LIMITS FOR CHLORINE

Water Quality-Based Permit Limits for acute and chronic criteria.
(based on EPA/505/2-90-001 Box 5-2).

Based on Lotus File WQBP2.WK1 Revised 19-Oct-93

INPUT	
1. Water Quality Standards (Concentration)	
Acute (one-hour) Criteria:	0.013
Chronic (n-day) Criteria:	0.075
2. Upstream Receiving Water Concentration	
Upstream Concentration for Acute Condition (7Q10):	0.000
Upstream Concentration for Chronic Condition (7Q10):	0.000
3. Dilution Factors (1/{Effluent Volume Fraction})	
Acute Receiving Water Dilution Factor at 7Q10:	20.000
Chronic Receiving Water Dilution Factor at 7Q10:	100.000
4. Coefficient of Variation for Effluent Concentration (use 0.6 if data are not available):	0.600
5. Number of days (n1) for chronic average (usually four or seven; four is recommended):	4
6. Number of samples (n2) required per month for monitoring:	30

OUTPUT	
1. Z Statistics	
LTA Derivation (99%tile):	2.326
Daily Maximum Permit Limit (99%tile):	2.326
Monthly Average Permit Limit (95%tile):	1.645
2. Calculated Waste Load Allocations (WLA's)	
Acute (one-hour) WLA:	0.260
Chronic (n1-day) WLA:	7.500
3. Derivation of LTAs using April 1990 TSD (Box 5-2 Step 2 & 3)	
Sigma^2:	0.3075
Sigma^2-n1:	0.0862
LTA for Acute (1-hour) WLA:	0.083
LTA for Chronic (n1-day) WLA:	3.956
Most Limiting LTA (minimum of acute and chronic):	0.083
4. Derivation of Permit Limits From Limiting LTA (Box 5-2 Step 4)	
Sigma^2-n2:	0.0119
Daily Maximum Permit Limit:	0.260
Monthly Average Permit Limit:	0.099

Source: Permit writer's spreadsheet used by the Washington Department of Ecology.

APPENDIX D

SLUDGE MANAGEMENT PLAN

CITY OF FLORENCE
SLUDGE MANAGEMENT PLAN

GENERAL INFORMATION:

The City of Florence operates an extended Aeration Activated Sludge Sewage Plant at 794 Rhododendron Drive. The plant was constructed in 1962 as a primary treatment facility. In 1972 the 35' primary clarifier was converted to secondary and aeration was added at that time. An additional 50' diameter clarifier and headworks were built in 1985. Two inclined screens and teacups were installed then, and one more screen and teacup in 1992.

Sewage is pumped to the plant from a main pump station at Ivy Street or a pressure main along Rhododendron Drive. A total of 27 pump stations are used in the collection system. Sewage flows into the plant at an average .68 MGD dry weather flow or .78 MPG wet weather flow. The treatment plant design capacity is .75 MGD. The City is currently under a mutual agreement order allowing bypasses when plan flows exceed .75 MGD.

WASTEWATER PROCESSING:

Approximately 75% of the incoming flow is domestic sewage, the remaining 25% being Commercial. There is no industrial source at this time, nor a sewer ordinance pertaining to industrial waste. Septage is no longer accepted at the plant, although future additional digester's will allow for receiving septage.

Sewage enters the plant at a headworks, consisting of 3 inclined screens and 3 grit separators. Sewage screening and grit are taken to the Lane County Transfer Station. Screened sewage then flows into a 725,000 gallon aerations basin. Optimum MLSS concentration is 3500 mg/L with a minimum of 3000 mg/L and maximum of 4000 mg/L. Seven floating aerators are in use with timers on all circuits to meet different oxygen demands. All aerators are 15 Hp and currently 5 operate 24 hours per day with the remaining two cycling on and off every half-hour. At average summer flows, 25 hours of detention time is achieved and 22 hours at winter flows. Aerated activated sludge is pumped to two secondary clarifiers, a 35' diameter older unit, and a 50' unit installed in 1985. Clarifier volumes are 80,000 and 160,000 gallons. Effluent flows to a 75,000 gallon chlorine chamber. Return sludge flows by gravity back to the aeration basin. Waste sludge is pumped off the bottom of the 50' clarifier to a gravity belt thickener. From there 5-7% solids are pumped to the digester heat exchanger. The high-rate digester has a volume of 90,000 gallons. Feed and mix pumps are progressive cavity. A draft tube mixer further mixes the digester contents.

SLUDGE PROCESSING:

Settled sludge is removed from the secondary clarifier by a Cornell Solids handling pump at a rate of 100 gpm and discharged onto a 1 meter Eimco gravity belt thickener. From there, 5% - 7% solids are pumped to the digester heat exchanger for preheat prior to discharge into the digester at 4 ports spaced equally along the top. Daily thickened feed rates average 2200 gpd for a detention time of about 41 days. Operating temperature is 99 degrees fahrenheit. Feed solids entering the digester are about 82% volatile and digested solids are about 3% and 75% volatile. Average volatile solids reduction for 1996 was 59%. Annual solids production at the present time is .75 MG or 93 dry tons/year. Beneficial use sludge application is on-going at 6 DEQ approved sites. Florence meets EPA 503.32 (Pathogen reduction) for class B sludge with its PSRP anaerobic digester time and temperature. Also 503.33 vector attraction reduction with option #1 (> 38% volatile solids reduction).

TRANSPORTATION AND LAND APPLICATION:

Sludge is removed from the plant via the mix pump (250 gpm) through piping routed across the top of the building to valves located above the truck loading position in the parking area. At the application site, sludge is either pumped through a diesel high pressure pump to an oscillating spray nozzle, or truck spread with a discharge pipe located at the side of the truck. All trucking, pumping and piping equipment is owned by the City of Florence. Truck operators are Tom Cannon, Brad Wilson and Ron Rainwater.

SITE NAME	ACRES	AUTHORIZED VOLUME/YEAR
Airport	19.9	199,000
Beatty	2	20,000
Nordahl	8.2	82,000
Chastain	13.1	131,000
King	90	900,000
Elliott	17	170,000
	150.2	1,502,000

METAL ADDITION
LBS/METAL/ACRE AT AGRONOMIC LOADING RATE

SITE NAME	Z	PB	NI	CU	CA
Airport	1.50	.07	-0-	1.60	.006
Beatty	1.50	.07	-0-	1.60	.006
Nordahl	1.50	.07	-0-	1.60	.006
Chastain	1.50	.07	-0-	1.60	.006
King	1.50	.07	-0-	1.60	.006
Elliott	1.50	.07	-0-	1.60	.006

NITROGEN AND AGRONOMIC LOADING

Organic = 67300 x .1876 x .2 = 2525 lbs/yr

Nitrate = 100 x .1876 = 18.8 " "

Ammonia = 52100 x .1876 x .5 = 4887 " "

7431 lbs/yr

$\frac{100}{7431}$ = 74.3 acres needed

EMERGENCY OPTIONS:

In the event of a spill, either at the treatment plant or along a roadway between the plant and the site, the City has available vacuum tanker trucks that could remove the sludge and return it to the treatment plant. Lime is to be used on all small spills.

In the event of mechanical breakdown of sludge or recirculation pumps, spare parts are available to immediately rebuild pumps or replace parts. If digester cleaning is needed, scheduling it for summer months will allow us to hold solids until it is back in service. A winter shutdown of short term (0-7 days) would necessitate ceasing waste sludge pumping. Medium term solution (7-30 days) would be a combination of drying bed use and lime stabilization. Long term solution would be lime stabilization with the City's tanker and possibly other vacuum tankers.

ısultants, Inc.

Lab Report No. 74484
Client P.O.
Date Received 1/8/97 1000

IS REPORT

Attention	Rick Mumpower	Collected Date	1/7/97 Time 1430
Client	City of Florence	Collected by	Tom Cannon
	PO Box 340	Source	Sludge / Digester
	Florence, OR 97439	Location	WWTP, Florence, OR

SEWAGE SLUDGE DEQ LIST

PARAMETER	METHOD	RESULTS	
Arsenic (Total)	EPA 206.2/7060	14.7	mg/kg dry weight
Cadmium (Total)	EPA 213.2/7131	3.1	mg/kg dry weight
Chromium (Total)	EPA 218.2/7191	17.9	mg/kg dry weight
Copper (Total)	EPA 220.1/7210	613	mg/kg dry weight
Lead (Total)	EPA 239.2/7421	33.2	mg/kg dry weight
Mercury (Total)	EPA 245.1/7470	2.2	mg/kg dry weight
Molybdenum (Total)	EPA 246.2/7481	7.4	mg/kg dry weight
Nickel (Total)	EPA 249.2/7521	24.1	mg/kg dry weight
Selenium (Total)	EPA 270.2/7740	ND@ 10.0	mg/kg dry weight
Zinc (Total)	EPA 289.1/7950	712	mg/kg dry weight
Total Nitrogen (TKN)	EPA 351.3	11.6	% dry weight
Nitrate Nitrogen	EPA 353.3	ND@ 0.01	% dry weight
Ammonia Nitrogen	EPA 350.2	6.35	% dry weight
Total Phosphorus	EPA 365.3	4.73	% dry weight
Potassium (Total)	EPA 258.1/7610	1.48	% dry weight
pH	EPA 150.1/9040	7.7	
Total Solids	EPA 160.3	2.72	%
Volatile Solids	EPA 160.4	72.3	%

ND means "not detected"

APPROVED Roy E. White DATE 2/10/97

4484.WPS

APPENDIX E

SUMMARY OF BIOASSAY FOR WASTEWATER TREATMENT PLANT EFFLUENT

NORTHWESTERN AQUATIC SCIENCES
A Division of NAS Associates inc
P.O. Box 1437, Newport, Oregon 97365 (503) 265-7225

August 27, 1997

Mr. Rick Mumpower
City of Florence
P.O. Box 340
Florence, OR 97437

Dear Mr. Mumpower:

Enclosed, please find copies of Repts. No. 573-1 and 573-2 giving the results of a dual-endpoint fathead minnow test and a rainbow trout acute test, respectively. No acute toxicity was observed in either test.

In the chronic toxicity test, due to a fungal problem encountered, there was significant mortality and also growth inhibition in the 50% effluent treatment. There was no signficant mortality or growth inhibition at the 100% effluent treatment. Therefore, it was concluded that the effects seen in the 50% effluent treatment were an anomoly due to the fungus and not to effluent toxicity. The no-observed-effect-concentration (NOEC) was concluded to be 100% effluent (no chronic toxicity). In addition, although the computation methods indicated an IC25 (point estimate for chronic toxicity) of 38.4% effluent, we conclude, for the reasons given above, that this does not indicate that effluent toxicity occured at this concentration.

If you have any questions, please feel free to call me at 541-265-7225.

Sincerely,

Gary A. Buhler
Project Manager

Encl.

TOXICITY TEST REPORT

TEST IDENTIFICATION

Test No.: 573-1

Title: Fathead Minnow dual endpoint (acute/chronic) toxicity test using static exposure to City of Florence effluent.

Protocol No.: NAS-XXX-PP2, September 15, 1990, Revision 2 (6-1-96). Based on U.S. EPA. 1994. Method 1000.0, Fathead minnow, *Pimephales promelas*, larval survival and growth test, pp. 58-113, In: Short-term methods for estimating the chronic toxicity of effluents and receiving waters to freshwater organisms. Third edition. EPA/600/4-91/002.

STUDY MANAGEMENT

Study Sponsor: City of Florence, P.O. Box 340, Florence, OR 97439.

Sponsor's Study Monitor: Mr. Rich Mumpower

Testing Laboratory: Northwestern Aquatic Sciences, P.O. Box 1437, Newport, OR 97365.

Test Location: Newport Laboratory.

Laboratory's Study Personnel: G.A. Buhler, B.S. Proj. Man./Study Dir.; L.K. Nemeth, B.A., QA Officer; M.S. Redmond, M.S., Aq. Toxicol.; G.J. Irissarri, B.S., Aq. Toxicol.; E. Coffey, B.S., Tech.

Study Schedule:

Test Beginning: 7-24-97, 12:00 p.m.

Test Ending: 7-31-97, 10:30 a.m.

Disposition of Study Records: All specimens, raw data, reports and other study records are stored according to Good Laboratory Practice regulations at: Northwestern Aquatic Sciences, 334 SW 7th Street, Suite B, Newport, OR 97365.

Good Laboratory Practices: The test was conducted following the principles of Good Laboratory Practices (GLP) as defined in the EPA/TSCA Good Laboratory Practice regulations revised August 17, 1989 (40 CFR Part 792).

Statement of Quality Assurance: The test data were reviewed by the Quality Assurance Unit to assure that the study was performed in accordance with the protocol and standard operating procedures. This report is an accurate reflection of the raw data.

TEST MATERIAL

Description: City of Florence unchlorinated secondary effluent. Details and water quality conditions at time of sample receipt are as follow:

NAS Sample No.	8694E	8703E	8701E
Collection Date	7-23-97	7-25-97	7-28-97
Receipt Date	7-24-97	7-26-97	7-29-97
Conductivity (umhos/cm)	370	360	380
pH	7.1	7.2	7.4
Hardness (mg/L)	60	40	40
Alkalinity (mg/L)	140	130	150
Total ammonia-N (mg/L)	4.5	7.5	8.0

Treatments: Samples briefly temperature equilibrated prior to use.

Storage: Stored at 4°C in the dark until used.

DILUTION WATER

Source: Moderately hard synthetic water prepared from Milli-Q water.
Dates of Preparation: 7-23-97, 7-28-97
Water Quality: Conductivity, 300/280 umhos/cm; pH, 7.9/7.8; hardness, 100/100 mg/L as $CaCO_3$; alkalinity, 80/70 mg/L as $CaCO_3$.
Pretreatment: Aerated $\geq$24 hr

TEST ORGANISMS

Species: *Pimephales promelas*, fathead minnow.
Age: 24 to 48-hr-old.
Source: Aquatox, Inc., Hot Springs, Arkansas
Acclimation: Holding conditions during the 24 hours prior to testing were: temperature, 22.4°C; dissolved oxygen, 8.3 mg/L; pH, 7.8; conductivity, 475 umhos/cm.

TEST PROCEDURES AND CONDITIONS

Test Chambers: 600 ml glass beakers containing 250 ml of test solutions.
Test Concentrations: 100, 50, 25, 12.5, 6.25, and 0 % (control).
Replicates/Treatment: 4
Organisms/Treatment: 40
Loading (based on final weight of control organisms): 0.02 g/L
Water Volume Changes per 24 hr: 1
Aeration: None
Feeding: Approximately 0.15g newly hatched *Artemia salina* nauplii per beaker twice daily, 6 hours apart, except on day 7.
Effects Criteria: The effect criteria used were: 1) mortality, and 2) growth inhibition. Mortality was defined as lack of visible movement during a 30 second observation period. Growth inhibition was measured as the difference in weight of fish between a treatment level and the control.
Water Quality and Other Test Conditions: Temperature, 24.6 ± 0.2°C; dissolved oxygen, 7.1 ± 1.3 mg/L; conductivity, 454 ± 41 umhos/cm (100% effluent), 297 ± 17 umhos/cm (control); pH, 7.5 ± 0.2; hardness, 40 ± 0 mg/L as $CaCO_3$ (100% effluent), 100 ± 0 mg/L as $CaCO_3$ (control); alkalinity, 124 ± 15 mg/L as $CaCO_3$ (100% effluent), 77 ± 5 mg/L as $CaCO_3$ (control); and photoperiod 16:8 hr, L:D.

DATA ANALYSIS METHODS

Percent survival and the average weight per larva were calculated for each treatment replicate from the raw data and the means were obtained for each treatment level. Average weights were calculated two ways: 1) based on the number of surviving fish (the historical method), and 2) based on the initial number of fish (as per EPA 600/4-91/002). The LC50 (survival) was calculated, where data permitted, either by the probit or the Trimmed Spearman-Karber method. The IC25 (growth) was calculated using the Linear Interpolation Method with bootstrapping. NOEC and LOEC values for survival and growth were computed using ANOVA and an appropriate *post hoc* test (Dunnett's test, T-Test with Bonferroni's adjustment, Steels Many-One Rank Test, or Wilcoxon Rank Sum Test with Bonferroni Adjustment). The appropriate test was selected after evaluating the data for normality and homogeneity of variance. Weight data were excluded from the ANOVA calculation if there was a survival effect. An arcsine transformation was performed on the survival data prior to statistical analysis. The statistical software employed for these calculations was ToxCalc, v. 5.0.15, Tidepool Scientific Software.

PROTOCOL DEVIATIONS

None

REFERENCE TOXICANT TEST

Test No.: 999-788

Reference Toxicant and Source: $CdCl_2$-2.5H_2O, Mallinckrodt, Lot No. TNZ.

Test Date: 7-24-97

Dilution Water Used: Moderately hard synthetic water.

Result: 7-day LC50, 21.4 ug/L Cd; 7-day IC25, 15.3 ug/L Cd; NOEC, 10 ug/L Cd. These test results are within the laboratory's control chart warning limits.

TEST RESULTS

Acute Endpoint: A detailed tabulation of the acute test results is given in Table 1. There was no mortality of fathead minnows exposed to 100% effluent after 48 hours. Therefore, the effluent passed the acute test according to Oregon DEQ guidelines (DEQ Whole Effluent Toxicity Testing Guidance Manual, January 1993).

Chronic Endpoint: A detailed tabulation of the chronic test results is given in Table 2. The biological effects, given as the NOEC and LOEC for survival and growth, and the LC50/IC25 for survival/growth, are shown below.

Comments: Control survival (97.5%) met the test acceptability criterion (80%). Control weights (0.58 mg) also met acceptability criterion (0.25 mg). The reference toxicant test results were within control chart limits. Therefore, this toxicity test is considered a valid test. It should be noted however, that there was significant fungal growth in most test chambers on days 5, 6, and 7 of the test with the heaviest growth in the 50% chambers and no fungal growth in the 100% chambers. All deaths except two occurred on day 7 of the test. Since this fungus also grew in control test chambers, it is likely that the fungus came with the batch of fish and not from the effluent. From examining the data, it appears that any toxic effect was likely due to the fungus.

	Survival	Growth based on survivors	Growth based on initial no.
NOEC (%)	100	100	100
LOEC (%)	>100	>100	>100
7-Day LC50/IC25 (%)	>100	>100	38.4
(95% conf. int.)	--	--	(23.2 - 51.5)
Method	Data Inspection	Linear Interpolation	Linear Interpolation

STUDY APPROVAL

Bryan A. Bahls 8/27/97 — Julie R. Fiore 8.26.97

Study Director — Date — Quality Assurance Unit — Date

J. O. A. Caldwell 8/27/97

Manager, Toxicology — Date

Table 1. Survival of fathead minnow larvae exposed for 48-hours to City of Florence effluent.

Effluent concentration (%)	Repl.	Number of larvae Exposed	Dead	Surviving	Percent survival	Mean* percent survival
100	1	10	0	10	100.0	
	2	10	0	10	100.0	
	3	10	0	10	100.0	
	4	10	0	10	100.0	100.0
Control	1	10	0	10	100.0	
	2	10	0	10	100.0	
	3	10	0	10	100.0	
	4	10	0	10	100.0	100.0

*An asterisk next to a treatment mean indicates that it is significantly ($P<0.05$) less than the control mean.

Table 2. Survival and growth of fathead minnow larvae exposed for seven days to City of Florence effluent.

Effluent conc. (%)	Repl.	Number of larvae Exposed	Dead	Surv.	% surv.	Mean % surv.	Ave.[1] wt/larva (mg)	Mean wt. (mg)	Ave.[2] wt/larva (mg)	Mean wt. (mg)
100	1	10	0	10	100.0		0.517		0.517	
	2	10	0	10	100.0		0.534		0.534	
	3	10	0	10	100.0		0.449		0.449	
	4	10	1	9	90.0	97.5	0.561	0.515	0.505	0.501
50	1	10	9	1	10.0		0.350		0.035	
	2	10	4	6	60.0		0.468		0.281	
	3	10	4	6	60.0		0.392		0.235	
	4	10	5	5	50.0	45.0*	0.520	0.433*	0.260	0.203*
25	1	9a	0	9	100.0		0.546		0.546	
	2	10	0	10	100.0		0.569		0.569	
	3	10	0	10	100.0		0.466		0.466	
	4	10	1	9	90.0	97.5	0.494	0.519	0.445	0.506
12.5	1	9a	0	9	100.0		0.526		0.526	
	2	10	0	10	100.0		0.556		0.556	
	3	10	0	10	100.0		0.585		0.585	
	4	10	0	10	100.0	100.0	0.500	0.542	0.500	0.542
6.25	1	10	0	10	100.0		0.530		0.530	
	2	10	2	8	80.0		0.526		0.421	
	3	10	0	10	100.0		0.543		0.543	
	4	10	0	10	100.0	95.0	0.567	0.542	0.567	0.515
Control	1	9a	0	9	100.0		0.632		0.632	
	2	10	0	10	100.0		0.608		0.608	
	3	10	1	9	90.0		0.481		0.433	
	4	10	0	10	100.0	97.5	0.585	0.577	0.585	0.565

* An asterisk next to a treatment mean indicates that it is significantly ($P<0.05$) less than the control mean.
[1] Average weight based on number of surviving fish larvae per replicate.
[2] Average weight based on initial number of fish larvae per replicate.
[a] Initial number of fish reduced by one to compensate for missing fish during the test.

TOXICITY TEST REPORT

TEST IDENTIFICATION

Test No.: 573-2

Title: Rainbow trout, *Oncorhynchus mykiss*, 96-hr acute bioassay using static exposure to City of Florence effluent.

Protocol No.: NAS-XXX-OM1, June 23, 1990, Revision 2 (3-8-95). Based on Weber, C.I. 1991. Methods for Measuring the Acute Toxicity of Effluents and Receiving Waters to Freshwater and Marine Organisms (Fourth Edition), EPA 600/4-90/027.

STUDY MANAGEMENT

Study Sponsor: City of Florence, P.O. Box 340, Florence, OR 97437.

Sponsor's Study Monitor: Mr. Rick Mumpower

Testing Laboratory: Northwestern Aquatic Sciences, P.O. Box 1437, Newport, OR 97365.

Test Location: Newport laboratory.

Laboratory's Study Personnel: G.A. Buhler, B.S., Proj. Man./Study Dir.; L.K. Nemeth, B.A., QA Officer; M.S. Redmond, M.S., Aq. Toxicol.; Coffey, B.S., Tech.

Study Schedule:

Test Beginning: 7-24-97, 2:00 p.m.

Test Ending: 7-28-97, 2:20 p.m.

Disposition of Study Records: All specimens, raw data, reports and other study records are stored according to Good Laboratory Practice regulations at: Northwestern Aquatic Sciences, 334 S.W. 7th Street, Suite B, Newport, OR 97365.

Good Laboratory Practices: The test was conducted following the principles of Good Laboratory Practices (GLP) as defined in the EPA/TSCA Good Laboratory Practice regulations revised August 17, 1989 (40 CFR Part 792).

Statement of Quality Assurance: The test data were reviewed by the Quality Assurance Unit to assure that the study was performed in accordance with the protocol and standard operating procedures. This report is an accurate reflection of the raw data.

TEST MATERIAL

Description: Unchlorinated final effluent (24HC). Details and water quality conditions at time of sample receipt are as follow:

NAS Sample No.	8694E	8702E
Collection Date	7-23-97	7-25-97
Receipt Date	7-24-97	7-26-97
Receipt Temperature (°C)	8.0	6.1
Dissolved oxygen (mg/L)	9.2	8.6
Conductivity (umhos/cm)	370	360
pH	7.1	7.2
Hardness (mg/L)	60	40
Alkalinity (mg/L)	140	130
Total ammonia-N (mg/L)	4.5	7.5

Treatments: Samples thermal equilibrated prior to use in test.

Storage: Stored refrigerated (4°C) in sealed container until tested.

DILUTION WATER

Source: NAS spring water
Date of Collection: 7-24-97
Water Quality: Conductivity, 260 umhos/cm; pH, 7.7; hardness, 100 mg/L as $CaCO_3$; and alkalinity, 60 mg/L as $CaCO_3$.
Pretreatment: Aerated prior to use.

TEST ORGANISMS

Species: Rainbow trout, *Oncorhynchus mykiss*
Size/weight: 0.31 g/fish.
Source: Purchased 7-18-97 from Mt. Lassen Trout Farms, Red Bluff, CA. Hatch date, 6-15-97.
Acclimation: Trout were held in flow-through tanks supplied with fresh spring water and aeration. Trout were fed twice daily until 24 hours prior to testing. Water quality conditions for the week prior to testing averaged: Temperature, 11.5 ± 0.9°C ; dissolved oxygen, 12.0 ± 2.0 mg/L; pH, 7.2 ± 0.5; conductivity, 295 ± 13 umhos/cm; hardness, 95 mg/L as $CaCO_3$; and alkalinity, 95 mg/L as $CaCO_3$. Acclimation tank loading: 1.0 g/L.

TEST PROCEDURES AND CONDITIONS

Test Chambers: 2-gallon glass aquaria containing 4.0 L of test solution.
Test Concentrations: 100, 50, 25, 12.5, 6.25, and 0% (control).
Replicates/Treatment: 2
Organisms/Treatment: 20
Loading: 0.78 g/L
Duration: 96 hours
Aeration: None.
Feeding: None during the test or for 24 hours prior to testing.
Water Volume Changes: One at 48 hrs.
Effect Criterion: Mortality, defined as the lack of respiratory movement in response to tactile stimulation.
Water Quality and Other Test Conditions: Temperature, 12.0 ± 0.2°C; dissolved oxygen, 10.3 ± 0.5 mg/L; conductivity, 450 ± 21 umhos/cm (100% effluent), 296 ± 22 umhos/cm (control); pH, 7.5 ± 0.2; hardness, 60 mg/L as $CaCO_3$ (100% effluent), 100 mg/L as $CaCO_3$ (control); alkalinity, 150 mg/L as $CaCO_3$ (100% effluent), 60 mg/L as $CaCO_3$ (control); and photoperiod 16:8 hr, L:D.

DATA ANALYSIS METHODS

Percent survival was calculated for each treatment replicate from the raw data and the means were obtained for each treatment level. The LC50 was calculated, where data permitted, either by the Probit or the Trimmed Spearman-Karber method. The statistical software employed for these calculations was ToxCalc, v.5, Tidepool Scientific Software.

PROTOCOL DEVIATIONS

None.

REFERENCE TOXICANT TEST

Test No.: 999-787
Reference Toxicant and Source: SDS, sodium dodecyl sulfate (Sigma Chemical, Lot No. 17H-0459).
Test Date: 7-24-97

Dilution Water Used: NAS spring water.
Result: LC50, 12.0 mg/L. This result is within the laboratory's control chart warning limits.

TEST RESULTS

A detailed tabulation of the test results is given in Table 1. The biological effect, given as the LC50 is as shown below.

96-hr LC50 (%)	>100
(95% C.I.)	--
Method	By Data Inspection

STUDY APPROVAL

Gary A. Buhler	8/27/97	Julie R. Fiore	8-26-97
Study Director	Date	Quality Assurance Unit	Date
Rick D. Cardwell	8/27/97		
Manager, Toxicology	Date		

Table 1. Survival of Oncorhynchus mykiss exposed to City of Florence effluent.

Effluent Conc. (%)	Repl.	Number of trout surviving 0-hr	24-hr	48-hr	72-hr	96-hr	96-hr % Survival individual	mean
100	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
50	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
25	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
12.5	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
6.25	1	10	10	10	10	10	100.0	
	2	10	10	10	10	10	100.0	100.0
Control	1	10	10	10	10	10	100.0	
	2	10	10	10	9	9	90.0	95.0