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.

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

	Means[a]			Extremes[a]		Mean number of days[b] Maximum		Mean number of days[b] 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)

	Discharge, cubic feet per second			
Month	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

	Discharge, cubic feet per second			
Month	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	90th percentile	min	max	mean	90th percentile	min	max	mean	10th percentile	min	max	mean	90th percentile
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.

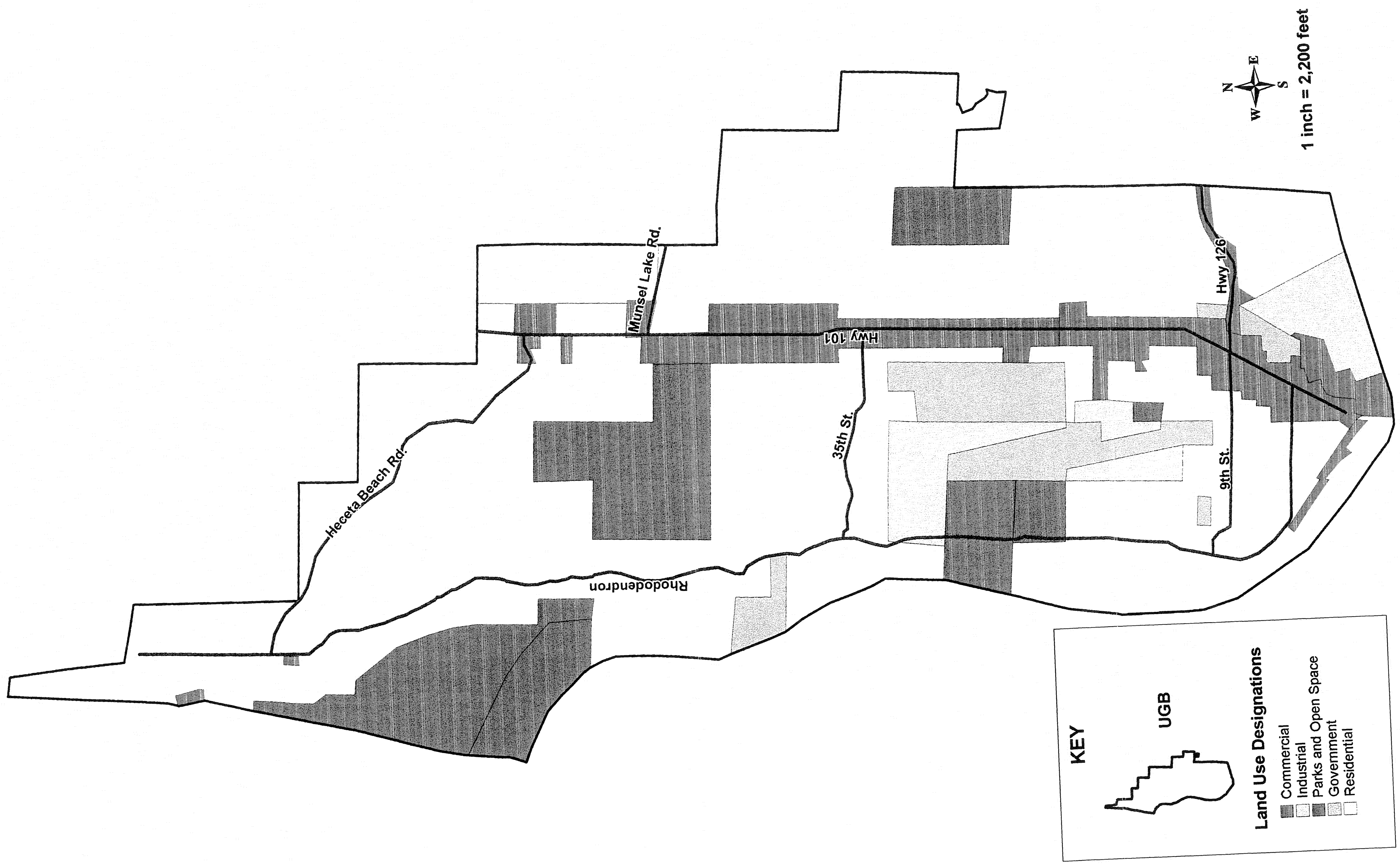

Figure 2-2. Land Use Designations in Florence Area

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.

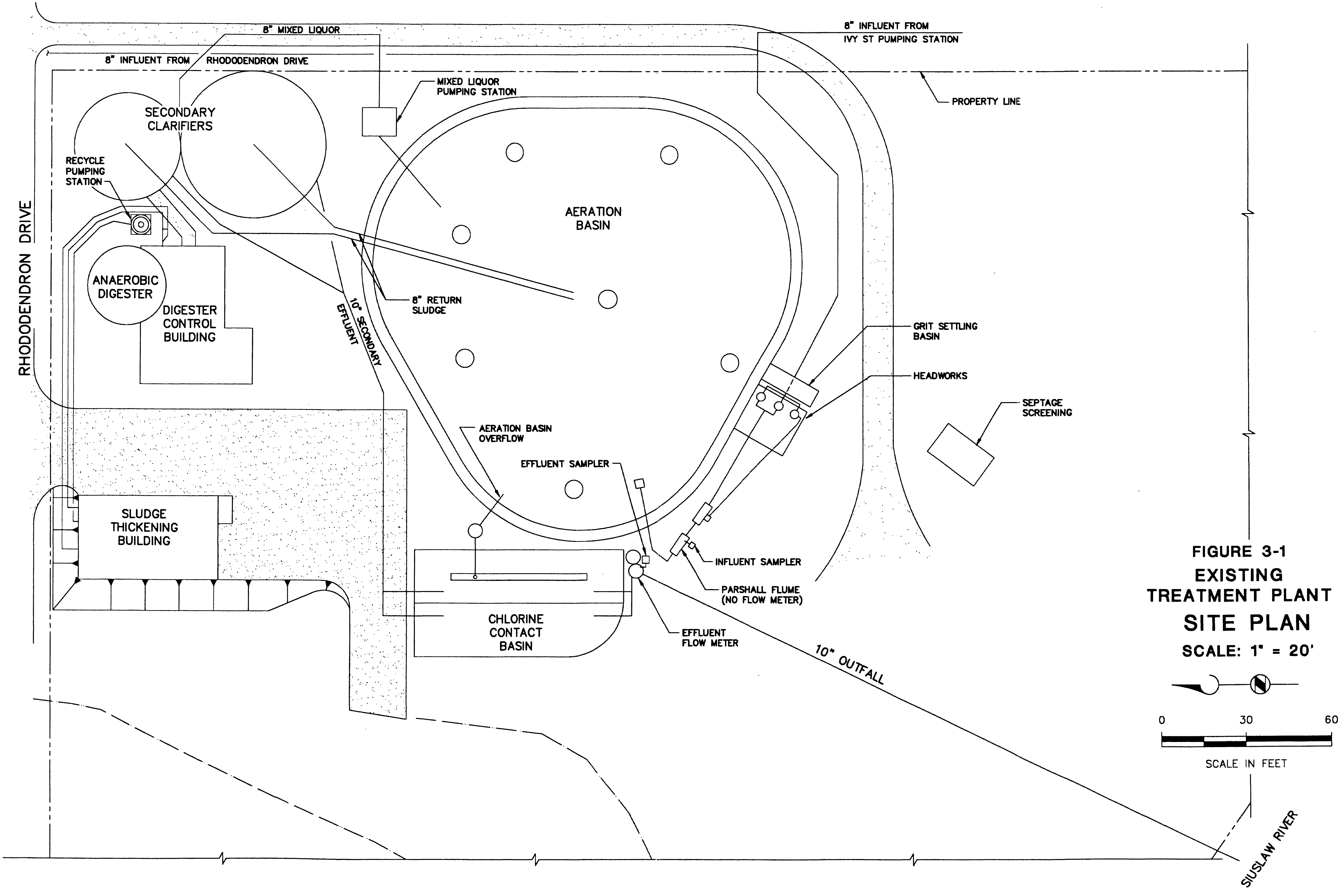

FIGURE 3-1
EXISTING
TREATMENT PLANT
SITE PLAN
SCALE: 1" = 20'

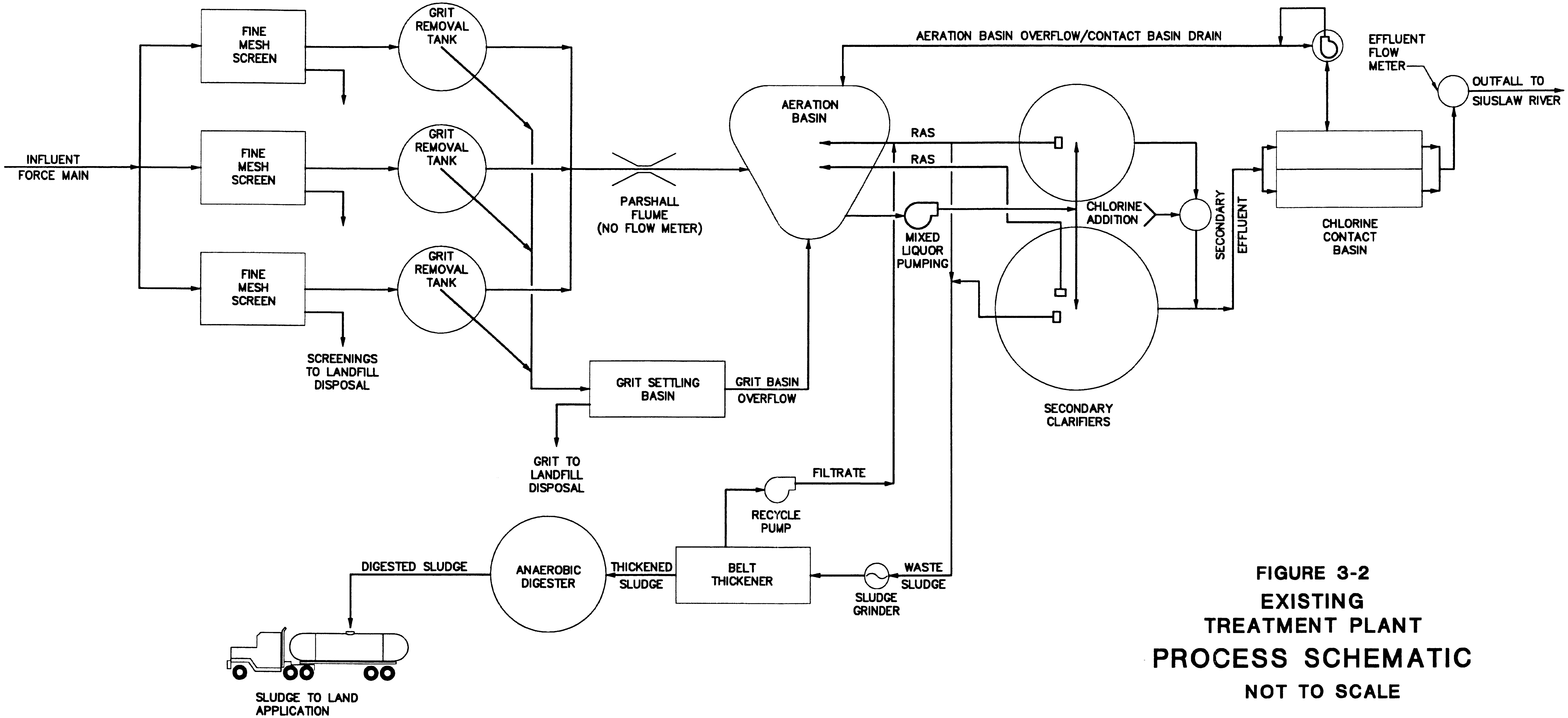

FIGURE 3-2

EXISTING
TREATMENT PLANT
PROCESS SCHEMATIC
NOT TO SCALE

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.

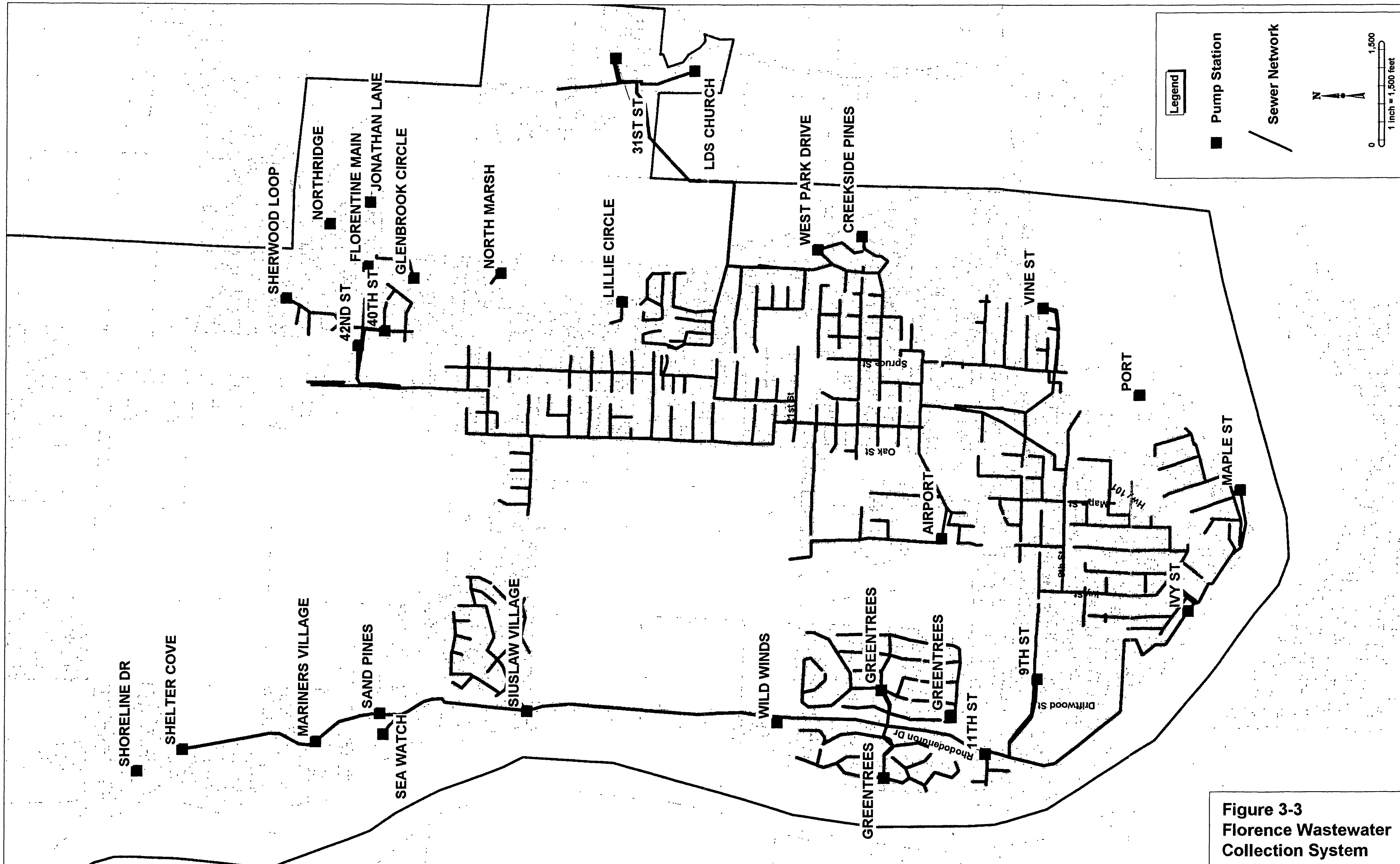

Figure 3-3
Florence Wastewater Collection System

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.

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

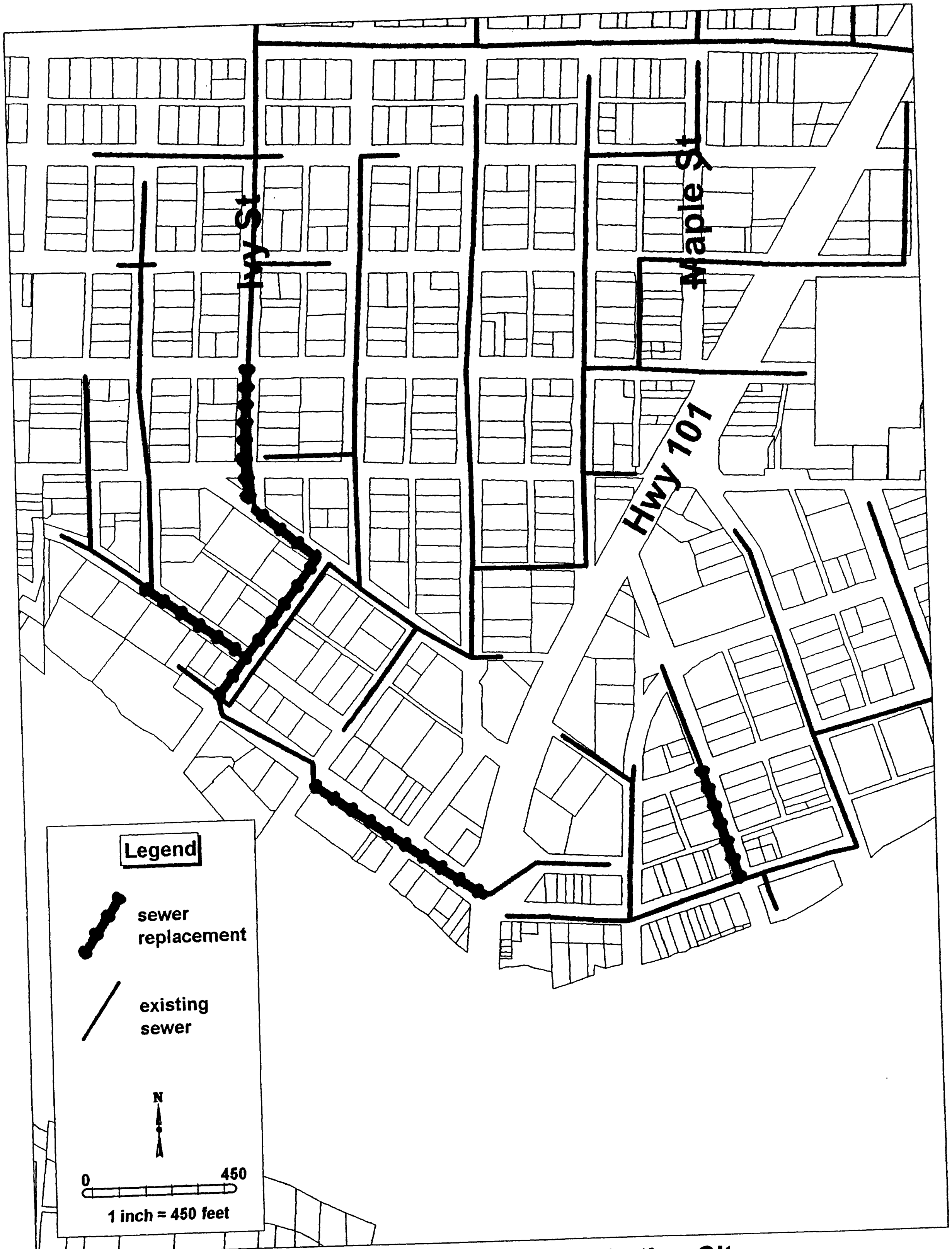

Figure 3-4. Sewer Rehabilitation Sites

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.

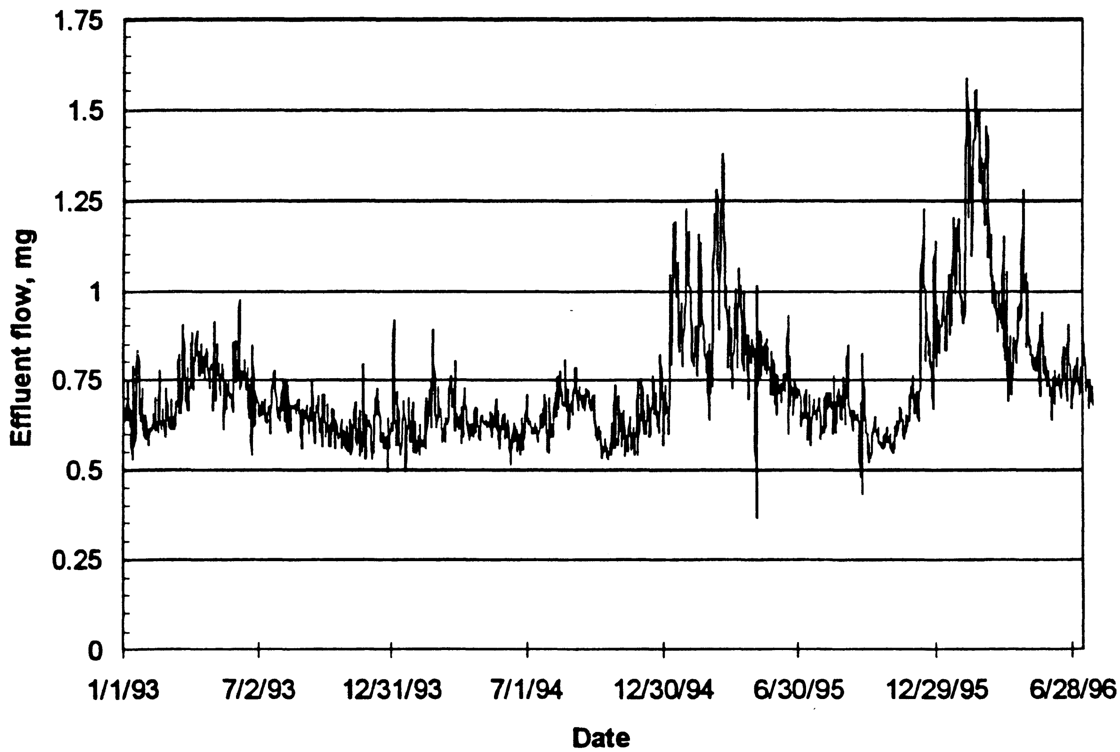

Figure 4-1. Daily Plant Flows

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.

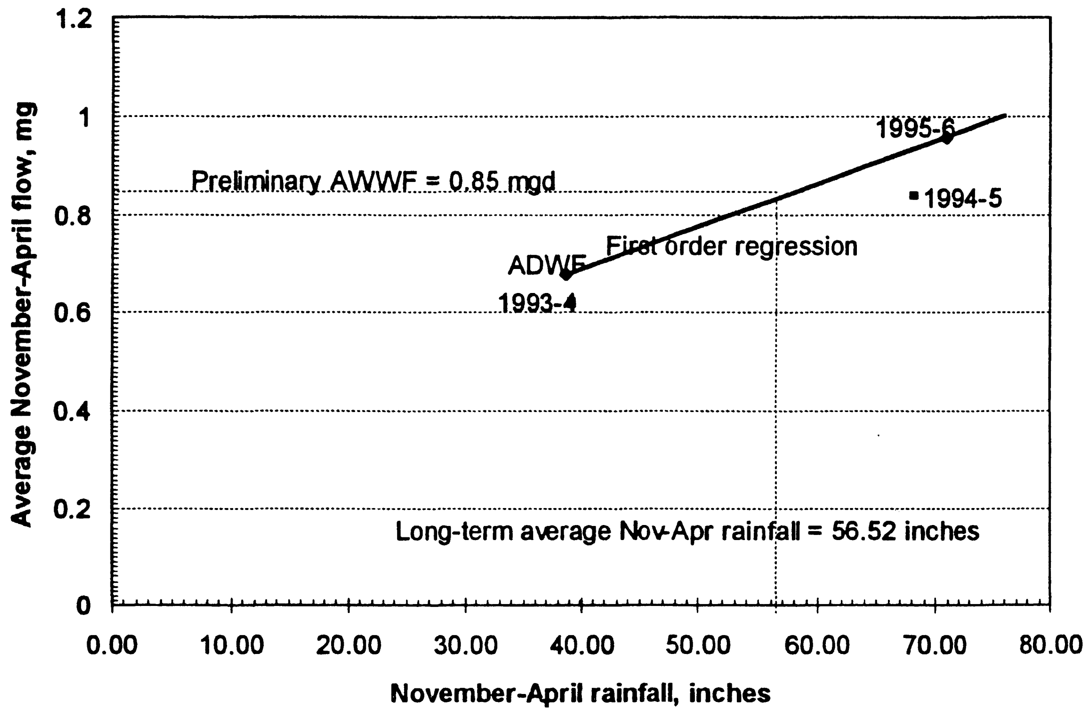

Figure 4-2. Average Wet Weather Flow

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).

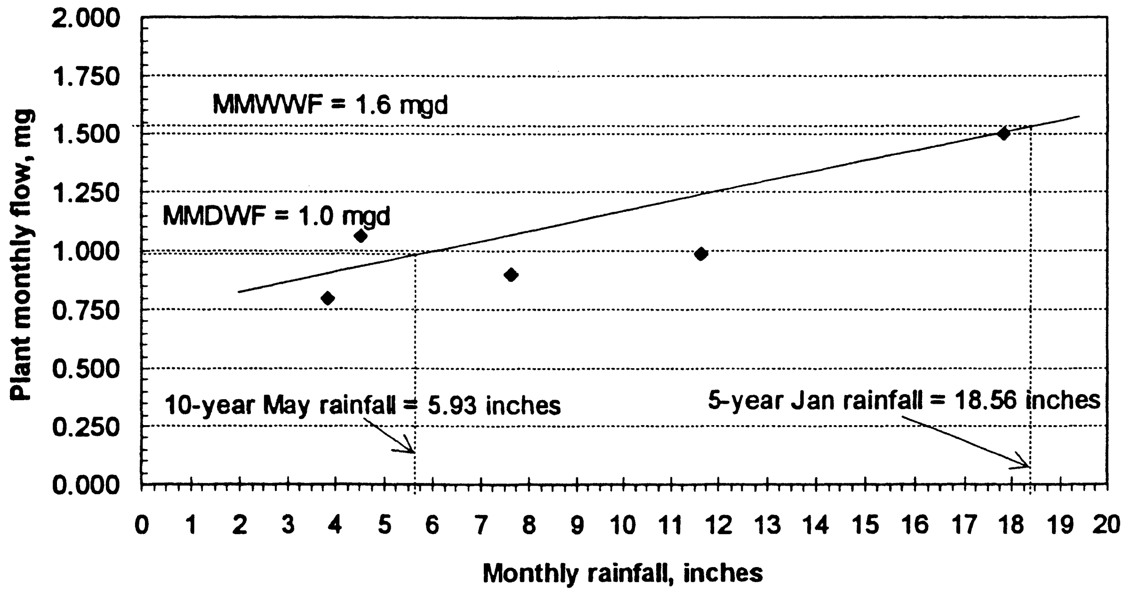

Figure 4-3. Maximum Month Flows

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.

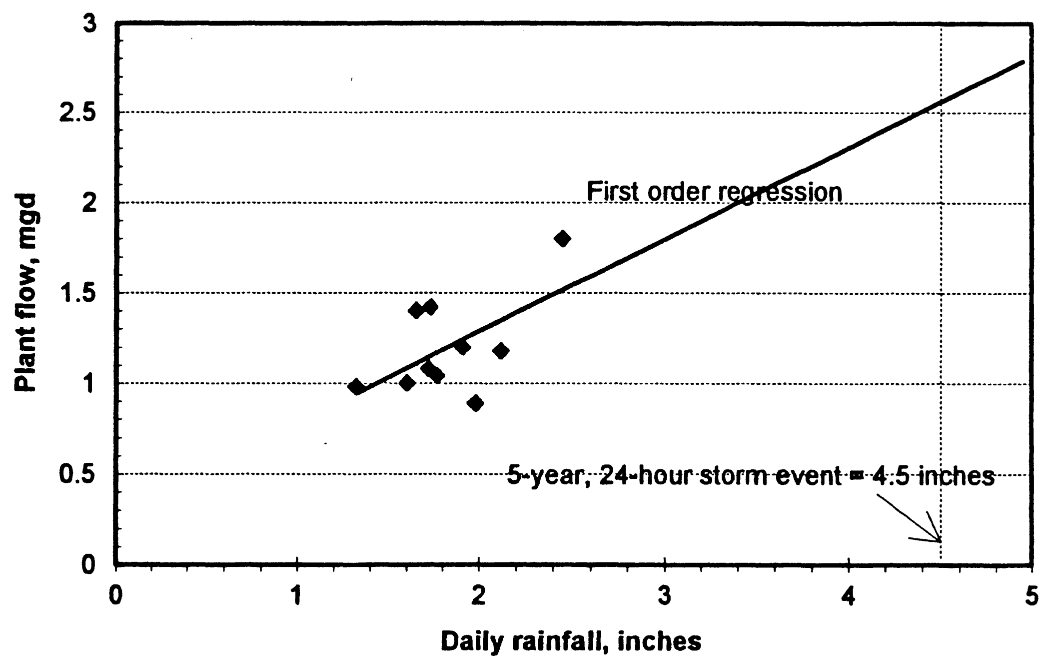

Figure 4-4. Peak Day Flow

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).

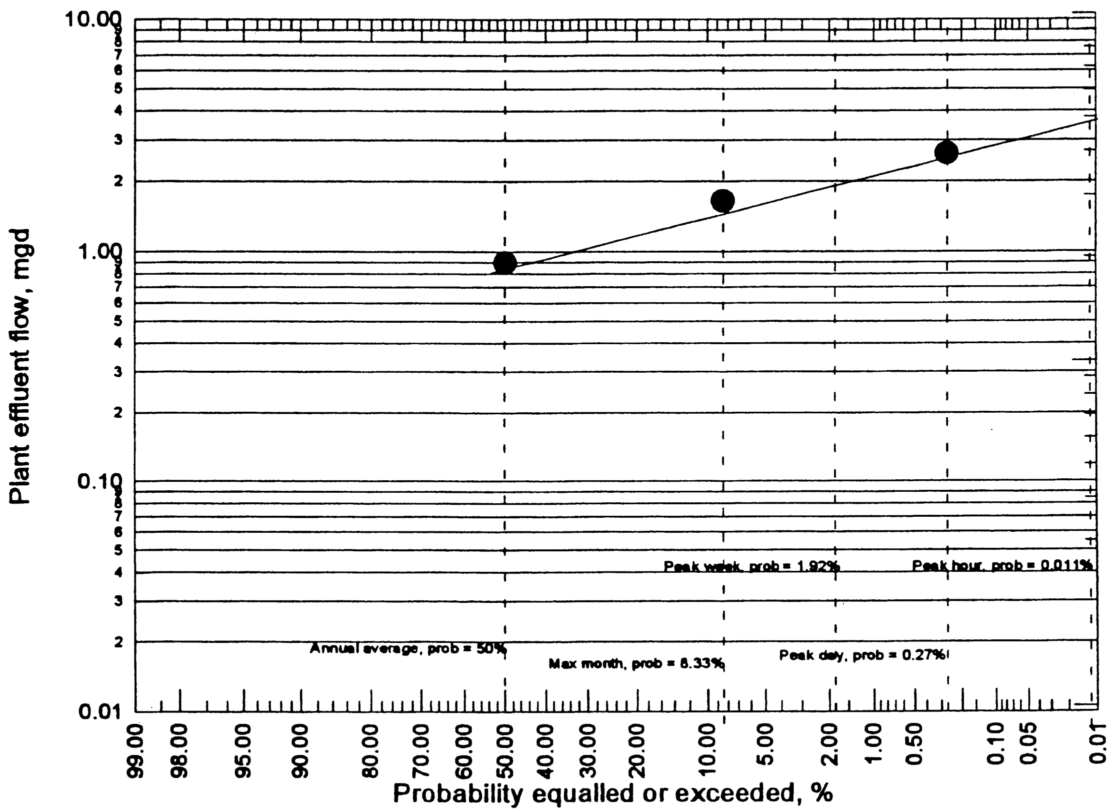

Figure 4-5. Flow Probability Analysis

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.

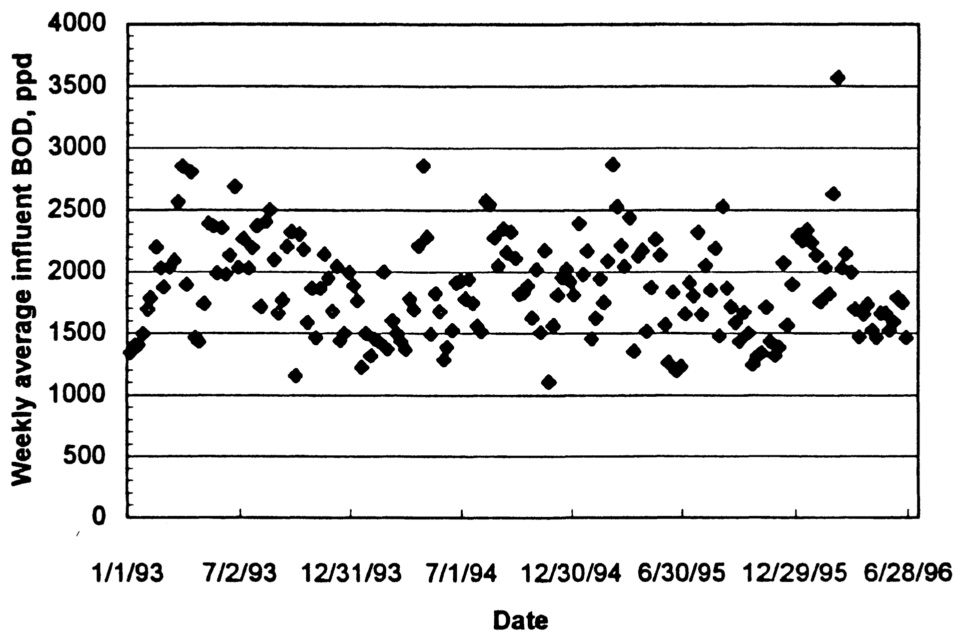

Figure 4-6. Weekly Average Influent BOD

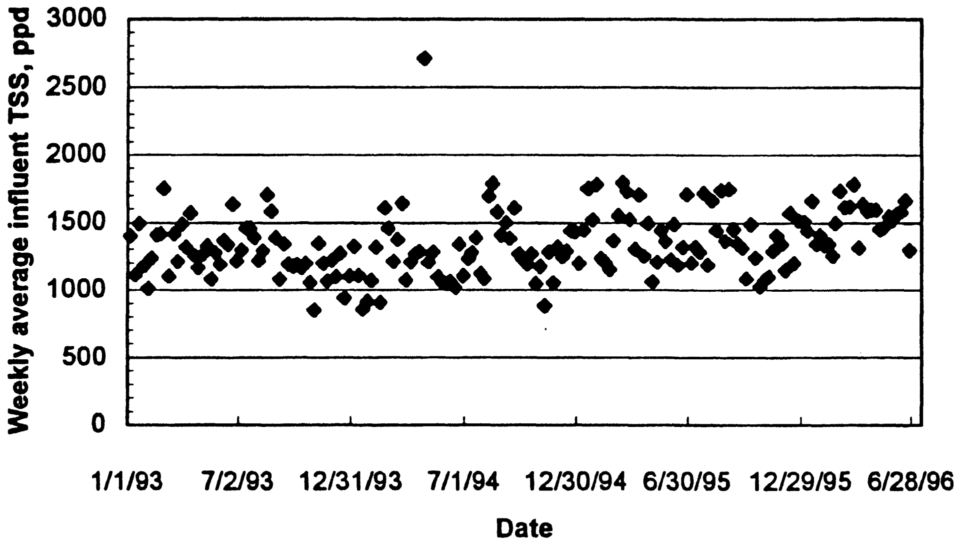

Figure 4-7. Weekly Average Influent TSS

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.

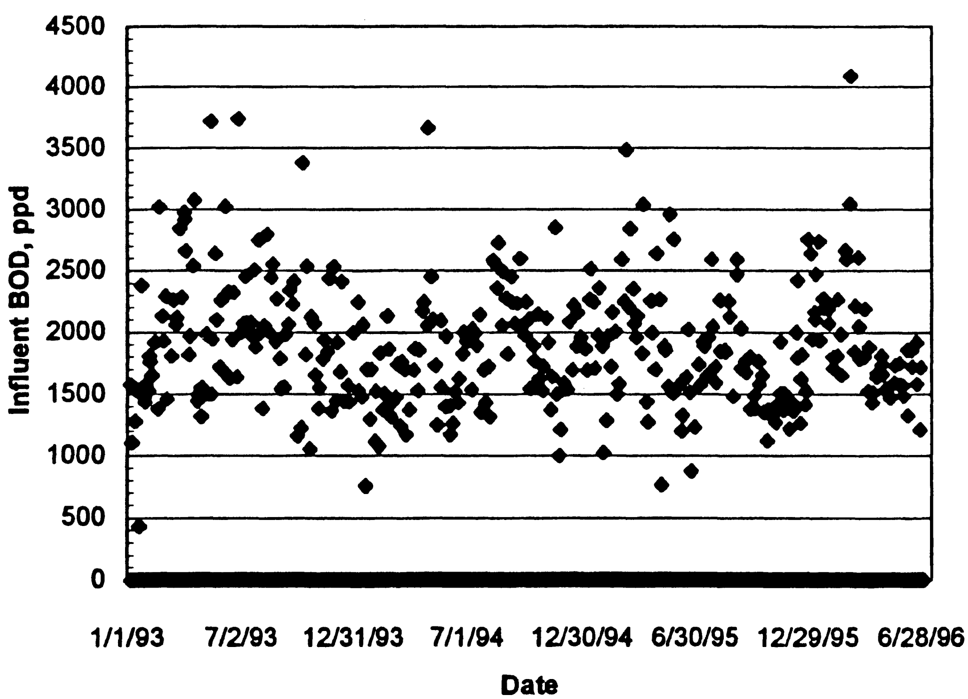

Figure 4-8. Daily Influent BOD

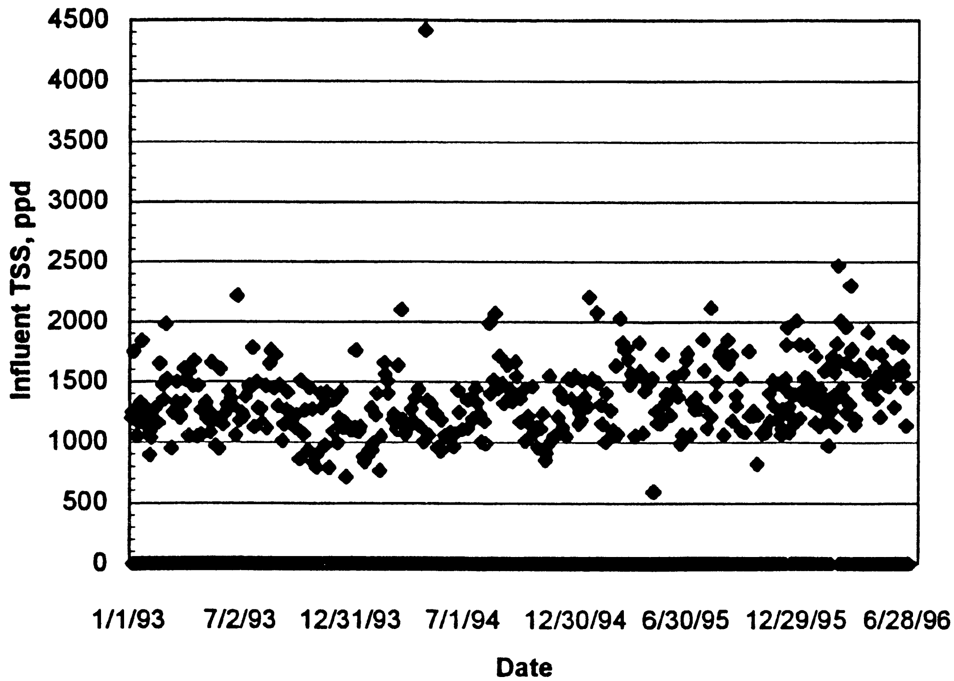

Figure 4-9. Daily Influent TSS

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.

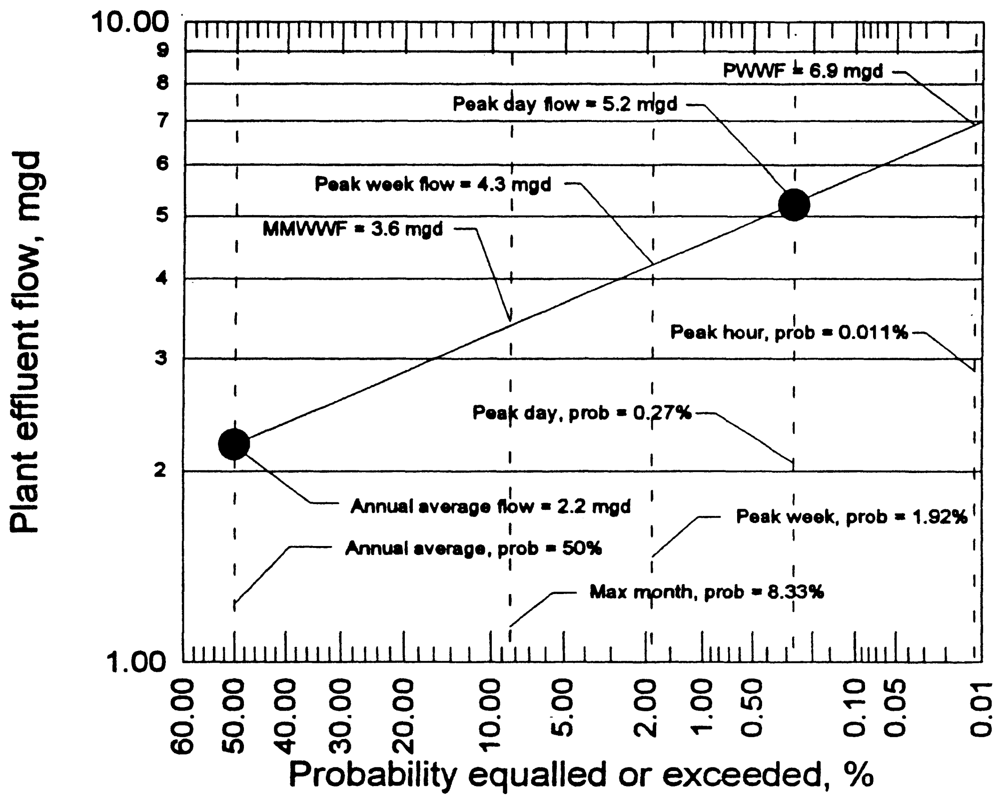

Figure 4-10. Peak Flow Projections

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: Monthly	Average effluent concentrations, mg/L: Weekly	Mass discharges, pounds per day: Monthly average	Mass discharges, pounds per day: Weekly average	Mass discharges, pounds per day: 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

<table>
<tr><th rowspan="2">Class</th><th colspan="5">Concentration and Period[a]
(All Units Are mg/L)</th><th rowspan="2">Use/Level of Protection</th></tr>
<tr><th>30-D</th><th>7-D</th><th>7-Mi</th><th colspan="2">Min</th></tr>
<tr><td rowspan="2">Salmonid Spawning</td><td rowspan="2"></td><td rowspan="2">11.0[b,c]</td><td rowspan="2"></td><td colspan="2">9.0[c]</td><td rowspan="2">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.</td></tr>
<tr><td>8.0[d]</td><td>6.0[e]</td></tr>
<tr><td>Cold Water</td><td>8.0[f]</td><td></td><td>6.5</td><td colspan="2">6.0</td><td>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.</td></tr>
<tr><td>Cool Water</td><td>6.5</td><td></td><td>5.0</td><td colspan="2">4.0</td><td>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.</td></tr>
<tr><td>Warm Water</td><td>5.5</td><td></td><td></td><td colspan="2">4.0</td><td>Waterbodies whose aquatic life beneficial uses are characterized by introduced, or native, warm-water species.</td></tr>
<tr><td>No Risk</td><td colspan="5">No Change from Background</td><td>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.</td></tr>
</table>

[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