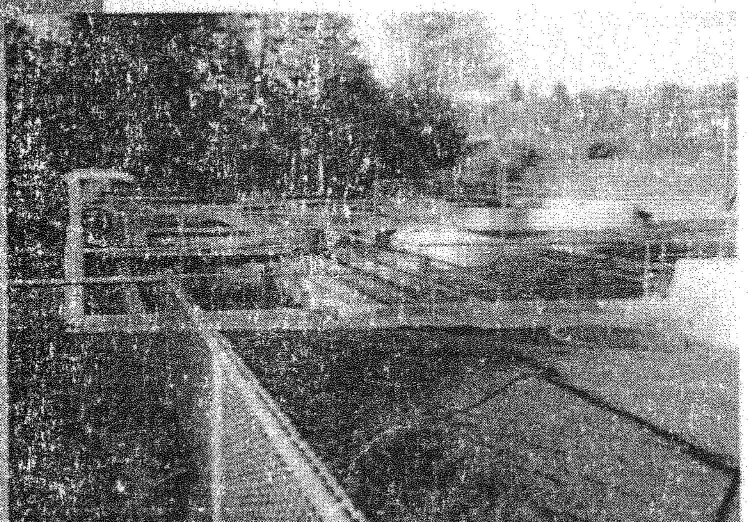
Engineering Report

CITY OF ESTACADA

Clackamas County, Oregon

WASTEWATER FACILITIES PLAN UPDATE



May, 2000

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CITY OF ESTACADA

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Funded in part by the Oregon Economic and Community Development Department

Clackamas County, Oregon

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May, 2000

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ACKNOWLEDGMENTS

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CITY OF ESTACADA
WASTEWATER FACILITIES PLAN UPDATE

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GLOSSARY OF TERMS

FLOWS

Average Dry-Weather Flow (ADWF) - The average of daily flows over the 6-month dry-weather period, May though October.

Maximum Monthly Dry Weather Flow (MMDWF-10) - The monthly average flow corresponding to the monthly rainfall accumulation during May with a 10% probability of being exceeded in any given year. West of the Oregon Cascades May is usually the rainiest summer month of high groundwater.

Maximum Monthly Wet Weather Flow (MMWWF-5) - The average monthly flow in the rainiest winter month (November-April) with high groundwater. West of the Oregon Cascades, this month usually corresponds to January. The 5-year MMWWF corresponds to the monthly rainfall accumulation during January with a 20% probability being exceeded. That is the amount of rainfall that exceeds 4 out of 5 totals that have been recorded in January.

Peak Daily Average Flow (PDAF-5) - The total daily flow that will result from a 5-year storm during a period of high ground water.

Peak Instantaneous Flow (PIF-5) - The peak hourly flow associated with a 5-Year PDAF. This value determines the hydraulic capacity of major process units, sewers, channels and pumps.

INFLOW AND INFILTRATION (I/I)

Infiltration - Water which enters the sewage system from the surrounding soil. Common points of entry include broken pipe and defective joints in pipe and manhole walls. Although generally limited to sewers laid below the normal groundwater level, infiltration also occurs as a result of rain or irrigation water soaking into the ground and entering mains, manholes, and even shallow house sewer laterals with defective joints or other faults.

Inflow - Stormwater runoff which enters the sewerage system only during or immediately after rainfall. Points of entry may include connections with roof and area drains, storm drain connections, and hole sin manhole covers in flooded streets.

INFLUENT AND EFFLUENT CHARACTERISTICS

Biochemical Oxygen Demand (BOD) The amount of oxygen required to stabilize the organic material in sewage by aerobic processes.

Total Suspended Solids (TSS)- All of the solids in sewage that can be removed by settling or filtration.

SOLIDS

Biosolids - Solid and semi-solid residuals resulting from wastewater treatment operations. Sludge must periodically be removed from treatment systems.

Dewatering - Removing the water from biosolids to reduce the volume which must be handled stored and hauled

RAS - Return Activated Sludge. The biomass settle din the secondary clarifiers which is pumped back to the solids contact chamber.

WAS - Waste Activated Sludge. The portion of biomass which is removed from the treatment system.

OTHER TERMS AND ACRONYMS

NPDES - National Pollutant Discharge Elimination System - Waste discharge permit issued by the Department of Environmental Quality. Includes conditions and limitations for operation of a wastewater collection, treatment, and disposal system and required effluent quality for disposal to public waters.

DEQ - Department of Environmental Quality

POTW -Public Owned Treatment Works

SDC - Systems Development Charge

MGD - Million Gallons per Day

EXECUTIVE SUMMARY

TREATMENT PLANT

The Estacada Wastewater Treatment Plant employs a trickling filter / solids contact (TFSC) process. The 1987 expansion included the construction of the solids contact chamber and the secondary floculating clarifiers to effect this treatment process. Additional improvements which were implemented at this time included the addition of a grit removal system, additional trickling filter influent pumping capacity, expanded chlorine contact volume, an effluent sand filtration system and expanded sludge treatment and storage components.

Plant Performance

The ability of the treatment plant to meet permitted mass loadings for discharge to the Clackamas River was investigated by analyzing current plant performance in detail. The Clackamas River is subject to the Three Basin Rule which stipulates that no new or increased waste discharges will be permitted. Since the permitted mass loadings will not change , it is possible to extrapolate what the future allowable organic and suspended solids concentrations will be.

Overall plant performance has been excellent. Discharge of BOD and SS average less than half the permitted mass loadings in the winter, and 70% of the summer permit. Infrequently, the TMDLs have been exceeded. Instances of non-compliance have been correlated with elevated flows.

High effluent BOD concentrations in the months of March through June are correlated with recycle from the biosolid storage ponds and a high solids inventory in the plant. Improvements to the solids handling capability of the treatment plant and associated operational changes should result in lowered effluent BOD concentrations in the spring.

With some improvements the existing secondary treatment process is capable of producing the high quality effluent which will be required to meet the permitted mass loadings within the twenty year planning period. Critical periods include May when dry weather mass loadings apply, but the flows may be indicative of a wet weather condition and extended high flows

High flows imply a high hydraulic loading rate which stresses the plant and reduces removal efficiencies. It is important that peak flows are reduced by an aggressive program of Inflow and Infiltration (I/I) control.

Process control strategies are outlined in detail. Maintaining high BOD removal efficiencies under a variety of conditions requires a consistent operational strategy including monitoring and analysis of process control parameters.

Plant Capacity

The treatment processes were analyzed separately to more accurately define the plant capacity. Currently the plant is within design capacities except during excess flow events when peak flows stress the individual processes. Reduced efficiencies may make it difficult to meet maximum day mass loading limits, emphasizing the importance of reducing peak flows by targeting I/I control.

When EDUs exceed 1970, secondary processes may approach design capacity. Performance of the treatment plant will need to be monitored and annually reviewed to precisely predict the expansion timing. The treatment system currently serves 1250 EDUs.

Recommended Improvements to the Treatment Plant

Improvements to the liquids processes of the existing treatment plant are necessary to provide pretreatment , pump projected peak flows, and comply with discharge regulations regarding chlorine. It is also recommended that the existing blowers be relocated to improve operating conditions in the control building while providing an opportunity to increase blower capacity.

Improvements to the sludge storage ponds and purchase of biosolids application equipment are necessary to provide for projected biosolid production. These improvements are a priority because without adequate storage or disposal too many solids are carried in the secondaries or recycled with the decant from the sludge storage ponds.

Improvements to the solids handling system are also an operational priority. Currently, lack of mixing in the sludge storage pond causes the accumulation of a very concentrated layer of solids which is difficult to pump with the existing equipment.

COLLECTION SYSTEM

Design of Interceptors to Serve the UGB

New interceptors were extended to the Urban Growth Boundary (UGB), and sized based on full buildout as defined by the City's Comprehensive Plan The capacity of existing trunk lines to handle the additional flow was also analyzed. Lines were laid out consistent with the topography to make use of natural drainage basins wherever possible.

Design flows for the existing main lines and proposed new interceptors were determined using, factors for the average number of equivalent dwelling units per acre (EDUs / acre). These were applied to each zoning designation and multiplied by the base sewage flow per residential unit, 290 gpd / EDU.

Inflow and Infiltration Monitoring

The purpose of the flow monitoring was to quantify the proportion of rainfall- induced I/I contributed by each basin, and to determine where the City should focus its control program. The monitoring results were compared to similar measurements in 1986, and 1988.

Flow monitoring was done after the ground was saturated so that measurements would include both inflow and infiltration. On the first day of sampling there had been no rainfall for the preceding 42 hours and the flows measured rates of infiltration. On the following morning sampling was done following a 5 hour period of heavy rainfall..

Flows were measured at downstream manholes on the four main lines and the relative contribution of each basin to I/I was calculated as a percentage of the total flow. Trunk line #1 contributed the highest percentage of flow and also showed the greatest increase in flow immediately following the rainfall, indicating significant inflow. Another area of potential inflow was identified on main line #4 by taking flow measurements above and below the lumber yards.

I/I contributions to peak day flows are estimated as 3.5 MGD. When the I/I is compared to the base flow of 0.36 MGD is apparent that I/I has a significant impact, increasing treatment plant flows nearly ten times.

RECOMMENDATIONS

The total cost of improvements to the wastewater treatment plant, which are required to meet the permitted mass loadings within the twenty year planning period, is estimated at $650,000. It is recommended that a pay as you go approach be taken and funding be provided from rates. Priorities and budgets can be prepared to accomplish needed biosolids projects first and dechlorination and blower building projects to follow. Improvement costs result in a rate increase of $7.33 / EDU/ month.

It is recommended that the annual amount budgeted for I/I control be increased to $50,000 per year, a rate increase of $2.66 / EDU / month. A strong investment now will reduce the stresses on the treatment processes and extend the life of the plant.

SYSTEM DEVELOPMENT CHARGES

(SDCs) are developed based on the updated capital improvement plan for the collection system, the portion of the treatment plant improvements which are capacity building, and reimbursement fees for the available capacity in the treatment and collection systems.

Reimbursement fees are base on the current value of the unused capacity of the treatment plant prorated to future users. The methodology previously adopted by the City uses a depreciated value which underestimates the remaining capacity of the treatment plant. To assure that future users contribute an equitable share of the capital cost of existing facilities we recommend a change in methodology to one that uses the current value of the treatment plant.

One method of estimating the current value is to estimate current cost from the original reimbursable cost of the treatment plant. This methodology results in a significantly larger reimbursement fee than the current methodology, and one that is more indicative of the remaining capacity. The total SDC using this alternative methodology was $2,475 per connection, compared to $1,600 per connection using the current methodology.

A third option for evaluating the current value of the treatment plant is to estimate today cost of replacement. This is probably the most accurate method of evaluating current value, if the City decides to change its methodology.

CHAPTER 1: PLANNING

1.1 STUDY AREA CHARACTERISTICS

The City of Estacada is located in the northeast portion of Clackamas County, Oregon at the foot of the Cascade Mountains. Adjacent to the city's southern and western limits is the Clackamas River flowing northwesterly to a confluence with the Willamette River at Oregon City. To the north and northwest lies level to moderately rolling terrain. The areas to the east, south, and southeast are characterized by forested ridges and valleys giving way to high plateaus.

Climatic conditions are characterized by moderate temperatures year round; wet winters, and relatively dry summers. Temperature extremes occur when higher atmospheric pressures force relatively warmer or cold air from east of the mountains through the Columbia Gorge and mountain passes to the west side of the mountains.

Annual average rainfall for the period form 1910 to 1998 was 40 inches. Wet weather rainfall in the period from September through April averages 4.0 inches per month and the dry weather period from May through October has an average of 1.56 inches per month.[1]

Estacada is a mixed community with forest related, educational, and service forms of employment. The desire to live in a rural and scenic environment has led to Estacada becoming somewhat of a bedroom community with residents commuting to other areas for employment. The areas largest employers are: the forest products industry, the U.S. Forest Service, Estacada School District, and service providers (restaurants, utilities, banks, gas stations etc.).

1.2 SURFACE WATER RESOURCES AND QUALITY

The Clackamas River has its origins in the high Cascade Mountains. Although it has generally excellent water quality, hydroelectric development consisting of a series of dams, and reservoirs on the middle and upper reaches, has created conditions which contribute to water temperature stratification and the growth of algae.

Hydroelectric operations also cause wide daily fluctuation in stream flow in response to wide variations in electrical demand. Runoff from timber harvest areas is a major contributor to the rivers silt and debris loading. The City of Estacada is the only sizeable urban concentration on the middle and upper reaches.

From River Mill dam down to Carver, the Clackamas River is designated as an Oregon Scenic Waterway. The portion of the Clackamas River where the plant discharges is just above River Mill Dam, and is known as the River Mill Reservoir, a popular fishing spot next to McIver Park. The dam has a fish ladder for the passage of adult anadromous fish and a downstream migrant pipeline

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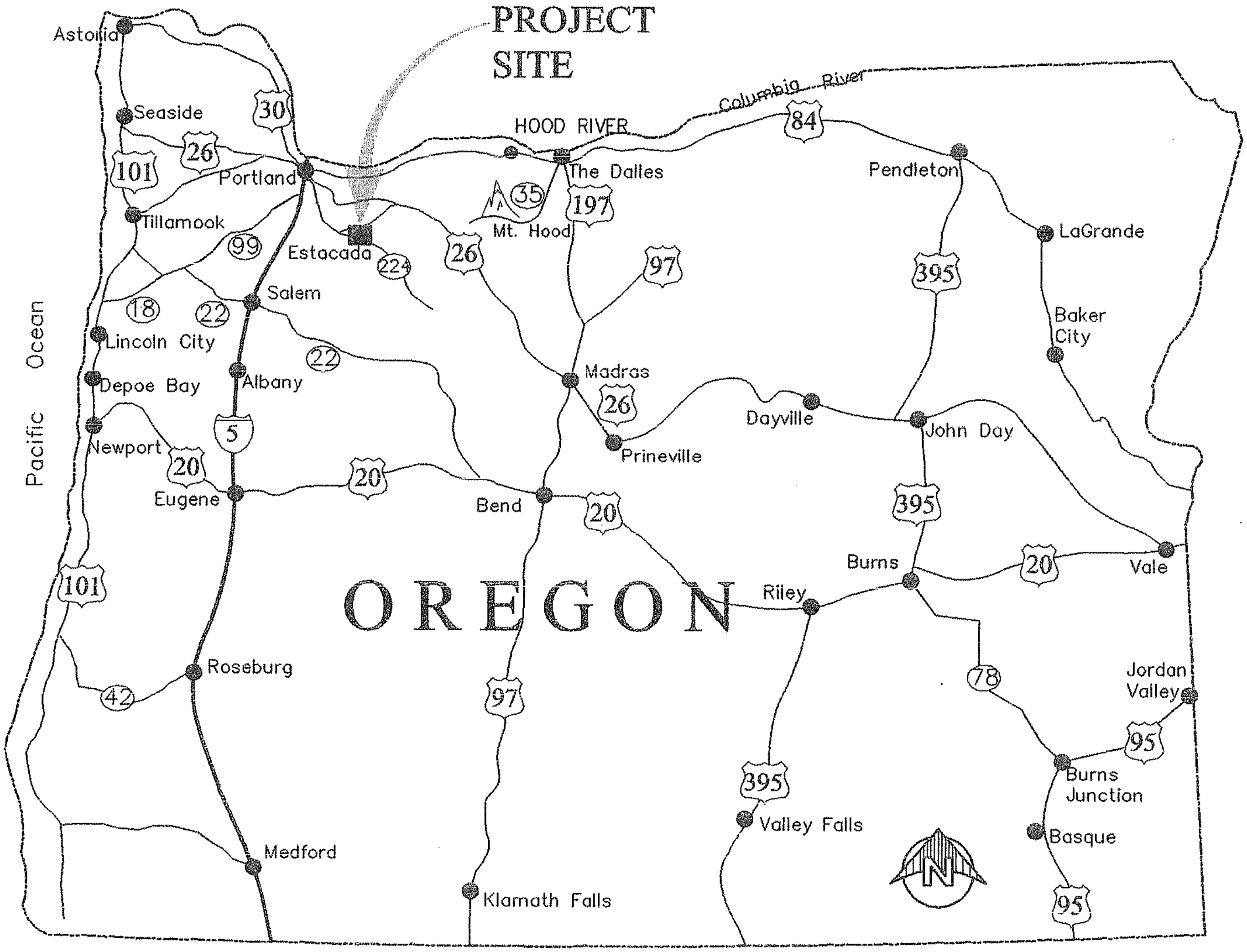
[1] <<http://www.ocs.orst.edu/pub_ftp/climate_data>> Gaging station 352693 Estacada 2 SE.



VICINITY MAP

NTS

CITY OF ESTACADA
WASTE WATER FACILITIES PLAN
Figure 1-1
CLACKAMAS COUNTY, OREGON
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which bypasses the Rivermill Reservoir and dam to transport smolt downstream. Downstream from the outfall, the U.S. Fish and Wildlife Service operates a fish hatchery for salmonids on Eagle Creek.

In addition to the fishery, recreation, and scenic values, another important beneficial use is as the drinking water supply for the Clackamas Water District, Lake Oswego, Gladstone, Oregon City, Tigard, Tualatin and West Linn as well as the City of Estacada. Protection of the high quality of the Clackamas is the goal of the Oregon River Basin Standards for the Clackamas River subbasin of the Willamette basin, Oregon Administrative Rules (OAR 340-41-442 to 470).

Specifically OAR 340-41-470, the Three Basin Rule, prohibits any further waste discharges to the Clackamas River. Any increase in population and wasteload to a sewage treatment facility discharging to the Clackamas River must therefore be compensated for by increased treatment.

1.3 LAND USE

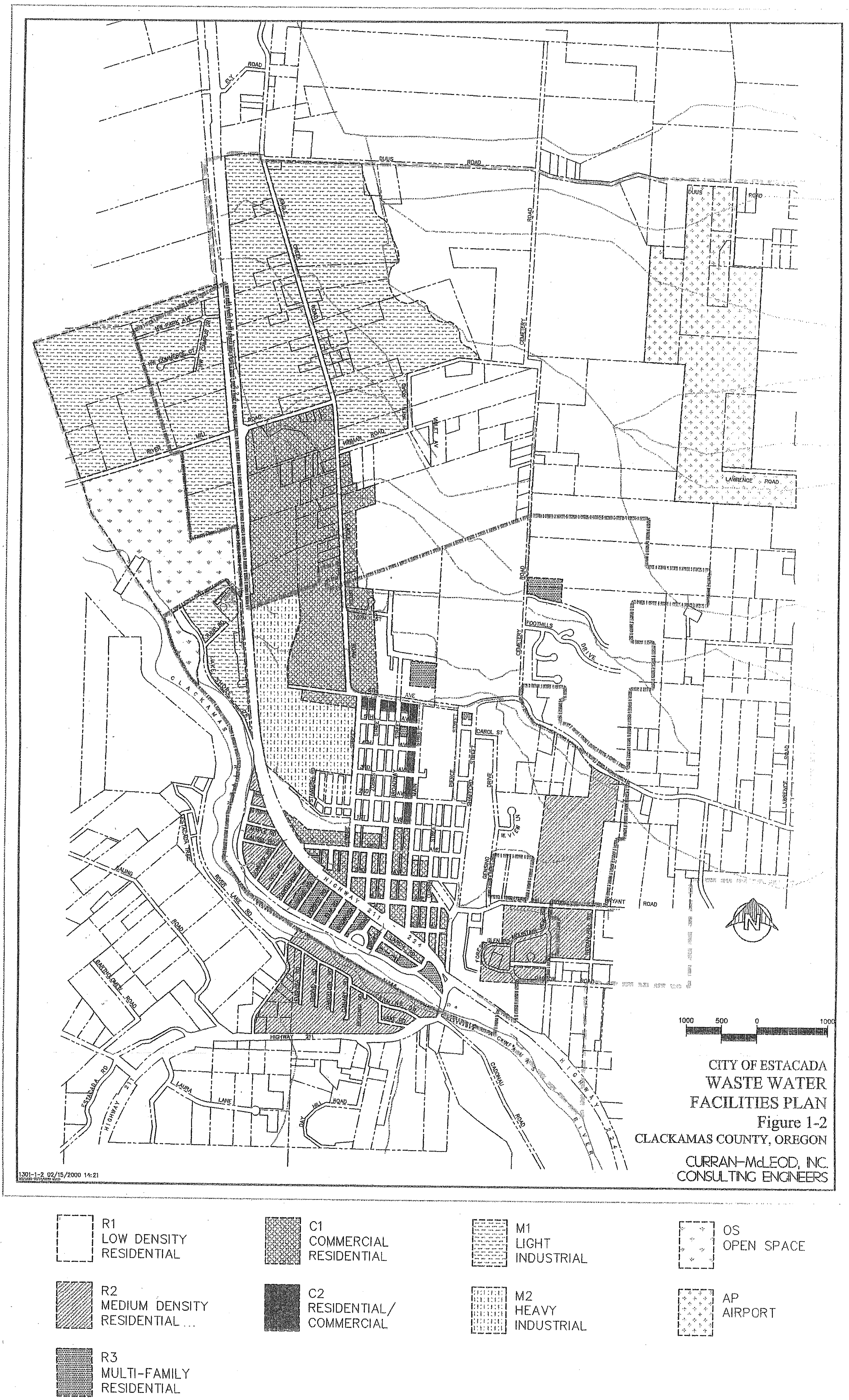
The study area for the Facilities Plan includes the City of Estacada and all areas within the Urban Growth Boundary for the City of Estacada (UGB). Land use within the City is governed by the Comprehensive Plan for the City of Estacada. Planning and development within the UGB is a coordinated effort by the City and County with the County's Comprehensive Plan and Zoning compatible with the City's recommendations considering land-use patterns.

The City has responsibility for public facilities planning within the UGB and plans to provide public services and facilities to all areas within the UGB. This Wastewater Facilities Plan Update includes plans for extending the existing sewer mains to the UGB as discussed in detail in the capital improvement plan for the collection system in Chapter 7.

Present land use patterns and zoning designations are shown in Figure 1-2. The 1978 Comprehensive Land Use Plan established the UGB to accommodate projected urban development needs in 1978. The designated area encompasses 2391 acres including the acreage within the City Limits. Table 1-1 gives a breakdown of the land use designations as contained in the Comprehensive Plan, 1979.

The plan assumes that new development within the UGB of single family residences will proceed at a rate of approximately 4 units per acre, which is slightly higher than the existing density, and less than the allowable density of 5.81.

Recently, 98 acres of residential land were annexed to the City including land to the north of the Foothills development along Cemetery Road, and a section of land within the City along Wade Creek and Copeland Road The Timber Park / River Mill Industrial Campus was annexed in 1998.


CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
Figure 1-2
CLACKAMAS COUNTY, OREGON
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1000 500 0 1000
R1 LOW DENSITY RESIDENTIAL
C1 COMMERCIAL RESIDENTIAL
M1 LIGHT INDUSTRIAL
OS OPEN SPACE
R2 MEDIUM DENSITY RESIDENTIAL ...
C2 RESIDENTIAL/ COMMERCIAL
M2 HEAVY INDUSTRIAL
AP AIRPORT
R3 MULTI-FAMILY RESIDENTIAL

Table 1-1: Land Use Zoning						
Land Use	Inside City Limits		Inside UGB		Total Designated	
	Developed	Total	Developed	Total	Developed	Total
Single family residential	216	265	134	474	350	739
Multi-family residential	13	15	0	90	27	105
Commercial	52	56	0	95	52	151
Industrial	83	83	0	352	83	435
Public	244	260	41	391	285	651
Semi-public	23	23	0		23	23
Airport			110	110	110	110
Hazard				197		197
Totals	631	702	285	1709	930	2411

Of the 82.6 acres of Industrial zoning within the City limits 58.65 are designated Industrial and are owned and operated by the Estacada Lumber Company. The Estacada Industrial Park, near the treatment plant, was developed with federal funding assistance in 1976 for industries seeking small plant sites.

The City Land Use Plan designated 352 acres of industrial land in the UGB in large tracts along the main transportation routes and in industrial parcels located on the northwest edge of the city and Urban Growth Boundary. A variety of sizes and locations were made available to encourage new industrial development.

Table 1-2: Industrial Zoning	
Description	Acres
West of Highway 224	132
East of Highway 224	90
South of Duus Road, with frontage on Eagle Creek Road	130
Total	352

The Timber Park and Campus Industrial Parks west of Highway 224 were annexed in 1998.

Developed land within this industrial park includes a 26 acre industrial development with multiple tenants.

1.4 POPULATION PROJECTIONS

At the time of the last Facilities Plan Update in 1986 the city planning staff projected an annual growth rate of 3.9%. This was partially based upon an expectation that the industrial base would be expanded. This growth rate was never realized, and in fact the population decreased from 1985-1990. The rate of growth of the population in Estacada has increased in recent years.

Population estimates and projections for Estacada through 2020 have been developed using estimates Portland State Center for Population Research and Census, census data for Estacada, Clackamas County data, and discussions with the City planning staff. City planning staff now project an annual increase of 2.3 % for projections over the twenty year planning period, based on recent growth.

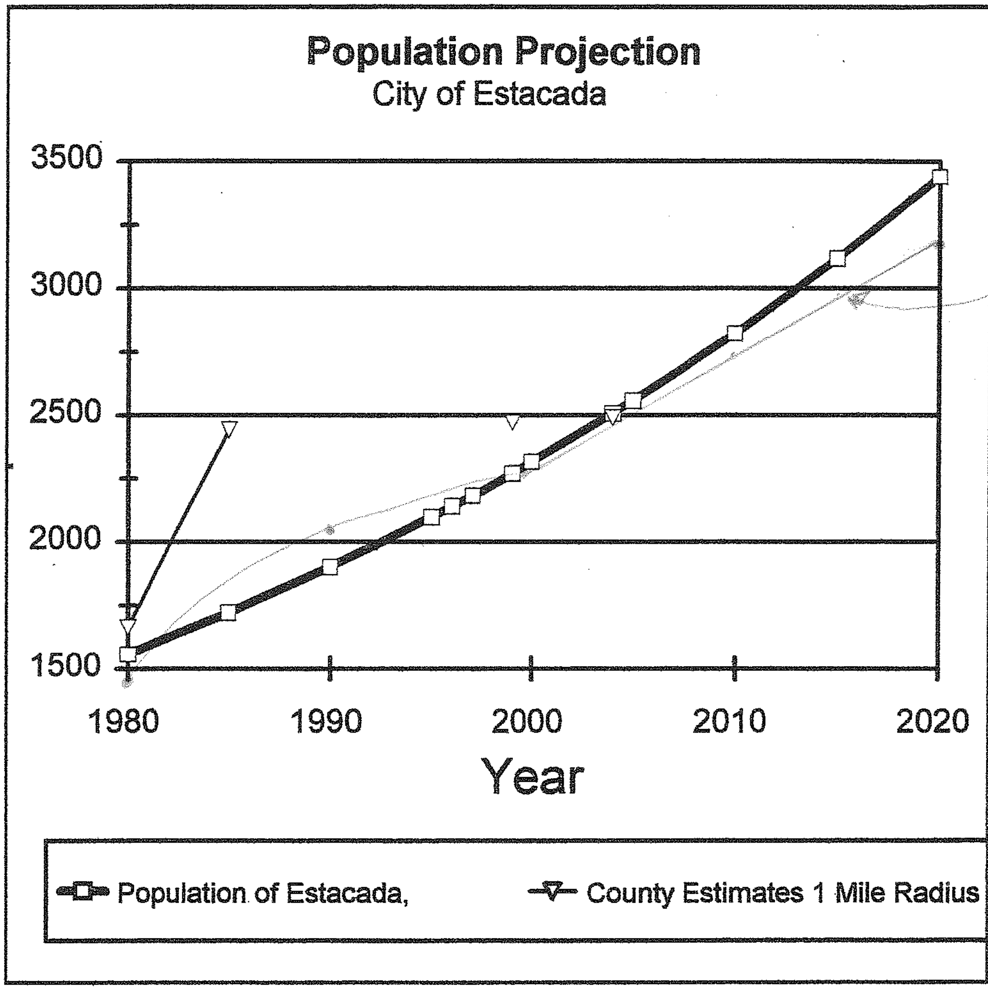
The data in Table 1-3 includes census information through 1990, official Portland State estimates through 1999, and projected growth at 2.3 % through 2020. These data are shown graphically on the following page. Clackamas County estimates for the area within a one mile radius of Main Street and 1st Street Estacada are included for comparison. County estimates have been taken into account in developing the City's projected growth as new areas are annexed.

Table 1-3: City of Estacada Population Projections

Year	City of Estacada		County Estimates	
			1 Mile Radius	5 Mile Radius
	Population	Annual Growth	Population	Population
1960	957			
1970	1164	1.90%		
1980	1419	2.00%	1666	9930
1990	2016	3.50%	2444	10427
1996	2065	0.60%		
1997	2100	1.70%		
1998	2190	4.20%		
1999	2200	0.45%	2474	11226
2000	2250	2.30%		
2004			2493	11578
2010	2824	2.30%		
2020	3545	2.30%		

3187

2250
2000
2011 = 2725
11 yrs @ 1.76 %/yr.
not 2.3 %



1.76%

2.12%/yr
1980-2011
(1419) (2725)

2020
(3363) 2.12%
(3197) 1.76%

Annexation is required to assure City services. Development to urban densities is prevented until properties are annexed into the City. For example a new housing development of 300 units is presently being considered off Cemetery Road in the northwest section of the City.

2.1 CURRENT FLOWS AND WASTEWATER CHARACTERISTICS

The critical design variables for sewage treatment are dry weather loading, maximum winter flows and peak instantaneous flows (PIF). The summer loads are important in estimating the treatment requirements needed to protect the river and satisfy water quality requirements. Maximum winter flows produce lengthy periods of hydraulic and organic stress, and the peak instantaneous flows dictate the hydraulic design capacity.

Under OAR 340-41-120(13) and (14), summer and winter storm conditions are defined. Raw sewage may not be permitted to discharge:

> Between November 1, and May 21, except during a storm event greater than the one - in - five year, 24-hour duration storm.
>
> Between May 22 and October 31, except during a storm event greater than the one - in - ten year , 24 - hour duration storm.

The City of Estacada wastewater pumping and treatment facilities are configured to prevent the discharge of raw sewage, ever. The only occasion when this prevention mechanism was abrogated was in February , 1996 when two one -in - 25 year storm events over two days overwhelmed the Lakeshore Drive pumping station and resulted in raw sewage and rainwater surging from street manholes, inundating the cul de sac, and overflowing the curbs into the Clackamas River.

In western Oregon, peak sewage flows are generally linked to rainfall. The Department of Environmental Quality has developed guidelines for estimating sewage flow rates in areas significantly impacted by rainfall based upon the relationship between sewage flow data and precipitation data and has defined several design flows.

The Maximum Month Dry Weather Flow ($MMDWF_{10}$) is the monthly dry weather flow with a 10 -year recurrence interval. The probability of a summer failure or overflow is reduced to 10 %.

The Maximum Month Wet Weather Flow$_5$ ($MMWWF_5$) is based upon the monthly wet weather average flow with a recurrence interval of 5 years, i.e. the flow that has an 80 % chance of not being exceeded. The design criteria for the dry period are more stringent due to greater risk of damage to human health and the environment during the lower natural stream flows associated with this period.

Estimates of the monthly rainfalls with these recurrence intervals is based on a probability analysis of monthly precipitation data from the Climatological Summary for Estacada.[2] In Estacada the total

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[2] Climatology of the United States No. 20, Climatic Summaries for Selected Sites,1951-1981: Asheville, N.C., National Climate Data Center, NOAA, US Department of Commerce.

monthly rainfall which has a 10 % chance of being exceeded in May is estimated as 5.65 inches, and the total monthly rainfall which has a 20 % chance of being exceeded in January is 13.24 inches.

The relationship between peak storms and peak sewage flows is determined by correlating monthly average flow rate to total monthly rainfall. DEQ guidelines use the period of January through May as coincident with high groundwater levels. This relationship is used to determine $MMWWF_5$ and $MMDWF_{10}$ as shown in the following graph.

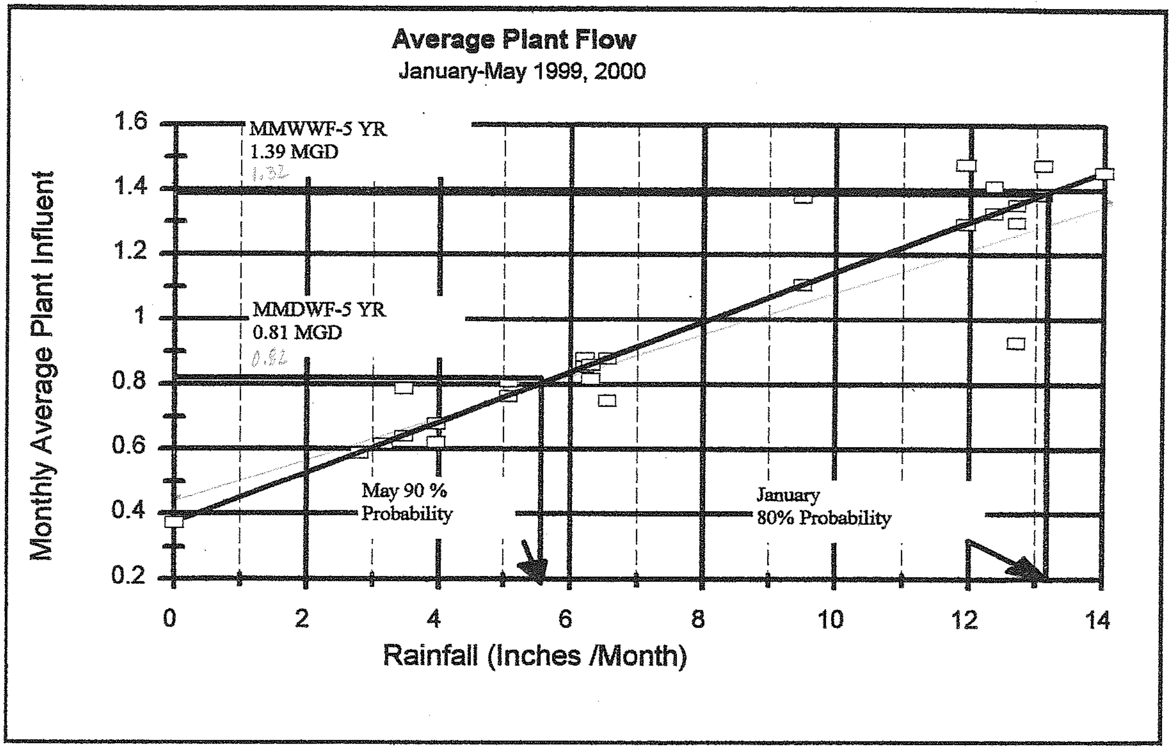


Table 2-1: Average Plant Flow / Winter Cumulative Rainfall		
	Average Flow (MGD)	Rainfall (Inches / Month)
January 1999	1.38	9.5
February 1999	1.30	12.7
March 1999	0.93	12.7
April 1999	0.59	2.8
May 1999	0.579	4.3
January 2000	1.161	9.6
February 2000	1.129	8.05
March 2000	0.942	5.55
April 2000	0.549	3.15

n = 9
r = 0.79
Q = 0.0658 R + 0.4517

The Peak Day Average Flow Associated with a 5-Year Storm ($PDAF_5$) is the flow that will result from a 5-year storm during a period of high ground water. From rainfall maps of the 5-year 24-hour precipitation, in Estacada the 5-year storm is estimated as 3.5 inches. This rainfall map is taken from NOAA Atlas 2, Volume X, Figure 26 (Oregon) and is included in Appendix V.

The $PDAF_5$ is estimated from a graph which shows the relationship between daily plant flow and daily rain for storms going back several years. Rainfall events greater than 1-inch in twenty-four hours were included, but only where the antecedent conditions were wet and the groundwater levels were high.

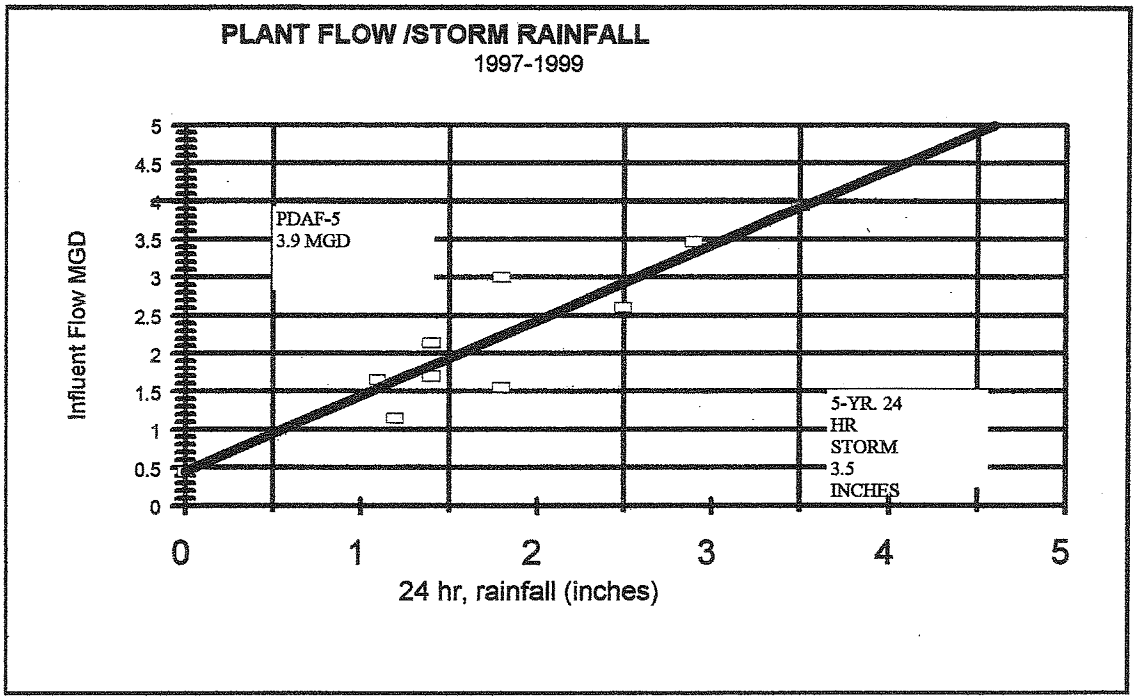
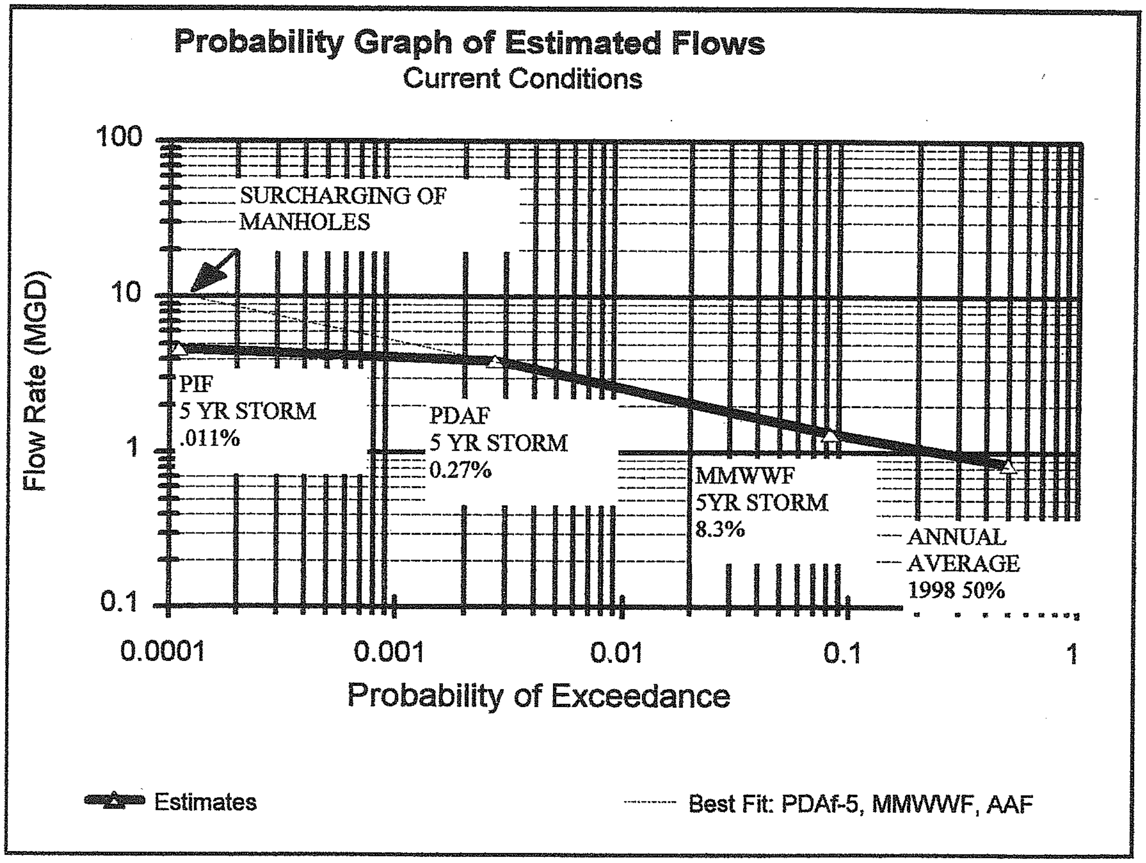


Table 2-2: Plant Flows / Daily Rainfall for Storms					
Date	Rainfall (inches / day)	Flow (MGD)	Date	Rainfall (inches / day)	Flow (MGD)
01-Jan-97	2.9	3.5	19-Feb-99	1.4	2.1
17-Jan-97	1.4	2.1	23-Feb-99	1.4	1.7
30-Jan-97	2.5	2.6	28-Feb-99	1.8	3.0
16-Jan-98	1.1	1.7	07-March-99	1.2	1.2
27-Dec-98	3.5	3.9	28-March-99	1.8	1.6

$n = 10$
$r = 0.89$
$Q = 0.9776R + 0.4829$

The Peak Instantaneous Flow (PIF_5) is defined as the sustained one-hour peak or instantaneous flow rate. Daily records for the month of December were examined and the average peaking factor for Peak Daily Flow / Peak Hourly Flow was 1.2. This peaking factor was applied to the $PDAF_5$ of 3.92 MGD to obtain the current PIF_5 of 4.7 MGD.

As outlined in the DEQ guidelines a probability graph of annual average flow in 1998, $MMWWF_5$, $PDAF_5$, and PIF_5 was drawn based on the expected frequencies of occurrence. The relationship between the estimated flows and the expected probabilities is statistically significant indicating that the estimates are reasonable.



Another method of estimating PIF_5 outlined in the guidelines, is to extrapolate the value from $PDAF_5$, $MMWWF_5$, and average annual flow in a wet year. This extrapolation results in a value for PIF_5 of 10.54 MGD which is more than twice the flow that the 18 inch interceptor upstream of the plant can deliver without surcharging.

It is apparent that the PIF_5 estimated from the probability plot is dampened by surcharging in the sewers. When upstream interceptors are replaced the plant may see higher peak instantaneous flows.

The PIF_5 is used as a design flow for the hydraulic capacities of the individual treatment plant processes. The estimated PIF_5 of 4.7 is greater than the 4.68 MGD (3250 gpm) pumping capacity of the trickling filter pumps, causing periodic overflows during high flow events which are usually of short duration. In the high flow event of January 1996, which was considered to be equivalent to a 50 year event, primary effluent overflowed to the outfall for several hours.

2.2 FLOW AND LOAD PROJECTIONS

To project future flows the projected annual growth rate is applied to the base flow. Infiltration and Inflow (I/I) are not expected to increase at the same rate, because sewers in new developments can be expected to have much lower infiltration rates than existing sewers.

Estacada is expected to continue to pursue an aggressive inflow and infiltration program which will be discussed in detail. Future improvements should offset any continued deterioration of the downtown section, therefore future flows are estimated using the assumption that there is no net increase in I/I.

The base flow is the domestic contribution treatment plant flows excluding I/I. The projected growth rate of 2.3% is applied to this base flow to estimate the projected increase in flow due to growth. Projections of future flow for $MMDWF_{10}$, $MMWWF_5$, PIF_5 and $PDAF_5$ are made by adding the projected increase in base flow.

The base flow for the dry weather period excluding May and October is 0.362 MGD. At a growth rate of 2.3% for 20 years the expected increase in base flow equals 0.208 MGD.

Example calculation:

Base flow 0.362 at 2.3% over 20 years = 0.57 MGD
Increase in base flow = (0.57 - 0.362) = 0.208 MGD

Projected $MMDWF_{10}$ = (Current $MMDWF_{10}$ + Increase) = (0.81 MGD + 0.208) = 1.0 MGD

The current organic and solids loading rate is based upon average dry weather values for BOD and TSS for a period of 3 years. The dry weather values accurately represent the current loadings because they are not subject to the dilution effects of wet weather.

Pounds of BOD = Average Dry Weather Concentration Times Base Flow
= (0.362 MGD) (230 mg/l) (8.34 lb/gallon)
= 694 lbs.
Pounds of TSS = Average Dry Weather Concentration Times Base Flow
= (0.362 MGD) (208 mg/l) (8.34 lb/gallon)
= 628 lbs.

Table 2-3: Projected Design Flows and Loadings									
		BOD		SS		$MMDWF_{10}$	$MMWWF_5$	$PDAF_5$	PIF_5
Year	population	lbs/day	ppcd	lbs/day	ppcd	MGD	MGD	MGD	MGD
2000	2250	694	0.31	628	0.28	0.81	1.39	3.92	4.70
2005	2521	778	0.31	704	0.28	0.85	1.43	3.96	4.74
2010	2824	871	0.31	788	0.28	0.90	1.48	4.01	4.79
2015	3165	976	0.31	883	0.28	0.96	1.54	4.07	4.85
2020	3546	1094	0.31	990	0.28	1.02	1.60	4.13	4.91

ppcd: Pounds per capita per day

Regression Equations for current flows:

$MMWWF_5$ = 0.37 MGD + 0.077 (Rainfall in January with a 20 % chance of being exceeded)
$MMWWF_5$ = 0.45 MGD + 0.066 (13.24 inches / month)
= 1.39 MGD

$MMDWF_{10}$ = 0.37 MGD + 0.077 (Rainfall in January with a 20 % chance of being exceeded)
$MMDWF_{10}$ = 0.37MGD + 0.077 (5.65 inches / month)
= 0.81 MGD

$PDAF_5$ = 0.45 MGD + 0.99 (Rainfall with a five-year reoccurrence interval)
= 0.45 MGD + 0.99 (3.5 inches)
= 3.92 MGD

PIF_5 = $PDAF_5$ (Peaking Factor)
= (3.92 MGD) (1.2)
= 4.70 MGD

CHAPTER 3: REGULATORY REQUIREMENTS

3.1 CURRENT PERMIT

The current NPDES permit # 01542 was issued to the City of Estacada, February 3, 1998, and expires on January 31, 2003. The effluent limitations are summarized in the following table.

The permit also stipulates that no wastes shall be discharged and no activities shall be conducted which will violate water quality standards as adopted in OAR 340-41-445 except in the defined mixing zone, and that the mixing zone will not extend more than 75 ft. from the point of discharge.

Table 3-1: NPDES Permit

		Average Effluent Concentrations		Mass Loading [3]		
		Monthly	Weekly	Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
May 1-October 31	Biochemical Oxygen Demand (BOD)	10	15	45	68	90
	Total Suspended Solids (SS)	10	15	45	68	90
November 1-April 30	Biochemical Oxygen Demand (BOD)	20	30	90	135	180
	Total Suspended Solids (SS)	20	30	90	135	180
pH	Shall be within the range of 6.0 - 9.0					
Escherichia coli (E. Coli)	30 day log mean of 126 organisms per 100 ml and no single sample shall exceed 406 organisms per 100 ml. Additional sampling is mandated.					
BOD and TSS	Shall not be less than 85% removal					

The Department of Environmental Quality has promulgated River Basin Water Quality standards for streams in Oregon. The Clackamas River is part of the Willamette River basin and for the mouth to River Mill Dam is subject to additional restrictions as a salmonid producing water. Basin standards, as defined in OAR 340-41-445, are summarized in Table 3-2.

[3] Mass loading based on an average dry weather design flow equaling 0.54 MGD

Table 3-2: Water Quality Standards for the Upper Willamette Basin	
Water Quality Parameter	Standard
Dissolved Oxygen	Cold Water Aquatic Life 8.0 mg/l minimum
Temperature	No measurable increases when stream temperature greater than or equal to: 64 ^{0} F salmonid rearing waters, 55 ^{0} F salmonid spawning waters
pH	6.5 - 8.5
E. Coli	30 day log mean 126, No single sample to exceed 406
Bacterial Pollution or other conditions deleterious to beneficial use	Prohibited
Liberation of dissolved gasses injurious to fish of other beneficial uses.	Prohibited
Development of fungi harmful to beneficial use.	Prohibited
Taste and odors affecting palatability	Prohibited
Bottom sludge deposits deleterious to beneficial use	Prohibited
Discoloration, scum or floating solids	Prohibited
Offensive conditions	Prohibited
Radioisotopes in water	Prohibited
Dissolved gasses	Less than 110 % of saturation, except for hatchery receiving water less than 105%.
Total dissolved solids	< 100 mg/l
Toxic substances	EPA Water Quality Criteria: Table 20 OAR 340

The current temperature standard for salmonid rearing waters is 64 degrees F (17.8 degrees C). The Clackamas River from the mouth to river Mill Dam has exceeded those standards and been placed on the Department of Environmental Quality (DEQ) 303(d) or water quality limited list in 1996, and again in 1998. Temperature limitations may be included in the discharge permit when it is renewed, and meanwhile should be monitored.

The sewage treatment plant effluent is generally not of sufficient volume to have a measurable impact on temperature. There are several months in late summer when the maximum temperature of the treatment plant effluent reaches 20 ^{0} C(71.6 ^{0} F). The worst case scenario is analyzed by using the critical dilution rate of 18.3:1 from the mixing zone analysis included in Appendix B. For the temperature of the river to be raised 0.25 Degrees F the flow in the river would need to be less would be an extremely rare occurrence. A flow of 757 cfs is exceeded 95% of the time so that flows less than this have a recurrence interval of greater than one in twenty years.

3.2 THREE BASIN RULE

What is known as the "Three Basin Rule", OAR 340-41-470 (1) (a), applies to The Clackamas, Mckenzie, and North Santiam river basins. "In order to preserve or improve the existing high quality water for municipal water supplies, recreation, and preservation of aquatic life, new or increased waste discharges shall be prohibited."

Since any new or increased waste discharges are prohibited, any increase in population and wasteload to any treatment plant discharging to the Clackamas must be compensated for by increased treatment efficiency.

Domestic wasteloads that are irrigated on land will not be considered an increase in permitted wasteload if there is no waste discharge to surface water and all groundwater protection requirements of OAR 340-040-0030 are met.

Under the Three Basin Rule, new storm water discharge permits are required to maintain a monitoring and water quality evaluation program which is effective in evaluation of the in-stream water quality impacts of the discharge. When sufficient data are available to do so the Department must assess the water quality impacts of the discharge.

Estacada is not currently required to have a NPDES permit for storm water discharge because of its size, but will be included under Phase II of the expanded NPDES system. In anticipation of these requirements, the City has identified several water quality modeling sites as part of its Storm Drainage Master Plan.

3.3 REGULATORY CRITERIA FOR SOLIDS PROCESSING

In 1993, the U.S. Environmental Protection Agency established final regulations governing the uses and applications of municipal sewage sludge. The criteria for land application fall into two broad categories:

- Processes to Significantly Reduce Pathogens (PSRP)
- Vector Attraction Reduction.

The application of PSRP involves the deactivation of pathogens with the classification of sludges into Class A or Class B sludges.

Class A - High quality sludges, low in pathogens (less than 1,000 FC/100 ml) and vector attractiveness: suitable for a variety of uses.

Class B - Lesser quality sludges for non-contact use only; pasture and woodland applications (less than 2,000,000 FC/100 ml).

Vector Attraction Reduction involves stabilization of sludges to avoid the accumulation of undigested organics which might attract insects, rodents, and other small animals. Vector Attraction Reduction is identified by measurement of the sludge stability through volatile solids reduction, oxygen uptake rate depression, application of thermal technology, alkalinity elevation or heat drying.

The City of Estacada may evaluate sludge using the following alternatives for full compliance with 40 CFR 503 regulations:

<u>PSRP - Class B Sludge</u>

1. Aerobic Digestion; 40 days detention time at 20°C, 60 days between 15° and 20°C; or,
2. Lime Stabilization: pH 12 following two hours of contact; or,
3. Fecal Coliform: less than 2 million / 100 ml.

<u>Vector Attraction Reduction</u>

1. Volatile Solids: 38% reduction; or,
2. Lime Stabilization: pH 11.5 following 24 hours of contact; or,
3. Bench Testing: 17% VSS reduction after 40 days @ 30°C-37°C or SOUR less than 1.5 mg/L/hr/g.

Surface application of sludge on available pasture is clearly the alternative of choice and production of Class B Biosolids permits this alternative. The most favorable operating modes for the City of Estacada include:

1. Aerobic digestion for 40 days at 20°C or higher
2. Aerobic digestion for 60 days with temperatures between 15° and 20°C;
3. Aerobic digestion and lime stabilization for 24 hours at pH 11.5 minimum

Lime stabilization offers the broadest compliance coverage because, in one process, PSRP and Vector Attraction are assured by definition. No further testing is called for and sampling, testing and reporting are minimized.

CHAPTER 4: EXISTING WASTEWATER FACILITIES

4.1 LIQUIDS PROCESSES

4.1.1 Description

Headworks, pretreatment and flow measurement

Grit removal is accomplished with a centrifugal grit collector with a hydraulic capacity of 2.5 MGD. The grit is pumped from the bottom of a conical bottomed collection chamber with an air lift pump to an inclined screw conveyor. The degritting process precedes the channel which formerly had a comminutor for grinding solids.

The degritting chamber and screw conveyor presently do a less than optimum job of removing water from the grit. The operation is inhibited by rags which catch on the conveyor and paddles.

Screening the wastewater stream removes coarse solids from entering the wastewater treatment facility and protects pumps, valves and other mechanical parts Presently the wastewater passes through a screen in the headworks channel following the Parshal flume. The 1 inch screen is raked by hand.

Primary Clarification

The Estacada Treatment Plant has two rectangular primary clarifiers with a total area of 1150 square ft. The overflow rate at 4.6 MGD is 4,000 gpd / sq. ft. This translates to an upward velocity of 0.37 ft. / min. which is sufficiently low to trap gross solids even at extremes of high flow.

The primary clarifiers can be dewatered by a 6" line connected to trickling filter pump #1. Sludge collection is achieved by new plastic reinforced flights and chains which transport sludge to the south end of the structure for removal and transfer to the aerobic digester.

Trickling Filters

The trickling filter provides an environment for the growth and proliferation of fixed - film bacteria. The fixed biomass on the stone media provides stability, and resilience under varying conditions. The trickling filter fixed-film bacteria adsorb the most volatile fraction of nutrients from the primary effluent and condition the trickling filter effluent for rapid metabolism in the solids contact channel.

The primary clarifier effluent is pumped to the primary clarifier. Trickling filter effluent is also recirculated over the filters with these pumps. The trickling filter pumps respond automatically to increases in filter wet well level and have a combined capacity of 3200 gpm.(4.6 MGD)

DESIGN FLOWS

AVERAGE DRY WEATHER FLOW (ADWF)	0.54 MGD
AVERAGE WET WEATHER FLOW (AWWF)	1.5 MGD
PEAK WET WEATHER FLOW (PWWF)	4.5 MGD

DESIGN LOADINGS

BIOCHEMICAL OXYGEN DEMAND (BOD^5)	1,080 LB/DAY
SUSPENDED SOLIDS (SS)	945 LB/DAY

GRIT REMOVAL SYSTEM

DESIGN FLOW CAPACITY	2.50 MGD

RAW SEWAGE COMMINUTION

DESIGN FLOW CAPACITY	3.50 MGD

PRIMARY CLARIFICATION (2)

TOTAL AREA, 2- 12'x48' RECTANGULAR	1,150 SF
OVERFLOW RATE @ ADWF	570 G/SF/D
OVERFLOW RATE @ AWWF	1,300 G/SF/D
OVERFLOW RATE @ PWWF	3,900 G/SF/D
SLUDGE REMOVAL CAPACITY	160 GPM

TRICKLING FILTER PUMPS

WET WELL CAPACITY	2,300 GAL/FT
NUMBER OF PUMPS	4
PUMPING CAPACITY	300- 3,200 GPM

TRICKLING FILTER

VOLUME OF MEDIA	43,000 CF
HYDRAULIC LOADING @ ADWF	80 GPD/SF
HYDRAULIC LOADING @ AWWF	220 GPD/SF
HYDRAULIC LOADING @ PWWF	650 GPD/SF
ORGANIC LOADING	163 LB BOD/1000 CF/D

SOLIDS CONTACT CHAMBER

VOLUME	25,600 GAL
RAS REAERATION TIME @ MAX RAS	0-40 MIN
SOLIDS CONTACT AERATION TIME @ ADWF	0-70 MIN

SECONDARY CLARIFICATION (2)

TOTAL AREA 2-52' DIA. CIRCULAR	4,250 SF
TOTAL FLOCCULATION WELL AREA 2-18' DIA. CIRCULAR	500 SF
OVERFLOW RATE @ ADWF	290 G/SF/D
OVERFLOW RATE @ AWWF	400 G/SF/D
OVERFLOW RATE @ PWWF	1,200 G/SF/D
MAX RAS CAPACITY	300 GPM/CLAFIFIER
WASTE ACTIVATED SLUDGE (WAS)	150 GPM

CHLORINE CONTACT CHAMBER

VOLUME	64,000 GAL
DETENTION TIME @ ADWF	2.75 HR
DETENTION TIME @ AWWF	1.0 HR
DETENTION TIME @ PWWF	0.33 HR

SAND FILTRATION

TOTAL AREA	640 SF
HYDRAULIC LOADING @ ADWF	0.60 GPM/SF
HYDRAULIC LOADING @ AWWF	1.6 GPM/SF
HYDRAULIC LOADING @ PWWF	4.9 GPM/SF

AEROBIC DIGESTER

VOLUME	70,000 GAL
TOTAL SLUDGE LOADING @ 17% SOLIDS	5,000 GPD
SOLIDS LOADING	15.6 LB/ 1000 CUFT

SLUDGE STORAGE PONDS

VOLUME POND 1	50,000 GAL
VOLUME POND 2	90,000 GAL

EFFLUENT QUALITY

BIOCHEMICAL OXYGEN DEMAND SUMMER/WINTER	45/90 LB/DAY
SUSPENDED SOLIDS, SUMMER/WINTER	45/90 LB/DAY

CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
1989 PLANT DESIGN - Figure 4-1
CLACKAMAS COUNTY, OREGON

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EPA RELIABILITY

CLASS I

DESIGN POPULATION

4500

DESIGN CAPACITY

AVERAGE DRY WEATHER FLOW (ADWF)	0.54 MGD
AVERAGE WET WEATHER FLOW (AWWF)	1.50 MGD
PEAK WET WEATHER FLOW (PWWF)	4.5 MGD

DESIGN LOADINGS

BIOCHEMICAL OXYGEN DEMAND (BOD)	1080 LB/DAY
TOTAL SUSPENDED SOLIDS	945 LBS/DAY

REQUIRED EFFLUENT QUALITY

DRY WEATHER BOD/SS	10/10 MG/L
WET WEATHER BOD/SS	20/20 MG/L
DRY WEATHER MASS LOAD LIMITS MO. AVE.	45 LBS
WET WEATHER MASS LOAD LIMITS MO. AVE.	90 LBS

INFLUENT CHARACTERISTICS

BIOCHEMICAL OXYGEN DEMAND (BOD)	
AVE WET WEATHER CONCENTRATION	110 MG/L
AVE DRY WEATHER CONCENTRATION	220 MG/L
LOADING	700 LBS/DAY
TOTAL SUSPENDED SOLIDS	
AVE WET WEATHER CONCENTRATION	95 MG/L
AVE DRY WEATHER CONCENTRATION	200 MG/L
LOADING	625 LBS/DAY

FLOWS 2020

MAXIMUM MONTH DRY WEATHER FLOW MMDWF-5	1 MGD
MAXIMUM MONTH WET WEATHER FLOW MMWWF-5	1.6 MGD
PEAK DAY FLOW PDAF-5	4.1 MGD
PEAK INSTANTANEOUS FLOW PIF-5	4.9 MGD

LOADING 2020

BIOCHEMICAL OXYGEN DEMAND (BOD)	1103 LBS/ DAY
TOTAL SUSPENDED SOLIDS (TSS)	985 LBS/ DAY

GRIT REMOVAL SYSTEM

VORTEX GRIT REMOVAL CHAMBER	
DESIGN CAPACITY	2.5 MGD
AIR LIFT PUMP TO CONVEYOR	

PRIMARY CLARIFIERS

2-12x48

TOTAL AREA	1150 SF
2-12'X8'RECTANGULAR	
OVERFLOW RATE	
@ MMDWF-5 YEAR 2020	869 G/SF/D
@ MMWWF-5 YEAR 2020	1391 G/SF/D
@ PDAF-5 YEAR 2020	3565 G/SF/D
@ PIF-5 YEAR 2020	4260 G/SF/D

TRICKLING FILTER PUMPS

WETWELL CAPACITY	2300 GAL/FT,11FT
PUMPS (4)	
PUMP 1	350GPM @ 27'
PUMP 2	650GPM @ 27'
PUMP 3 ,4	1200GPM @ 32'

TRICKLING FILTER

CAPACITY	3200 GPM (4.6 MGD)
VOLUME OF MEDIA	43000 CF
ORGANIC LOADING - YEAR 2020	20 LBS BOD/1000 CF

SOLIDS CONTACT CHAMBER

VOLUME	25600 GALONS
DETENTION TIME	
@ MMDWF-5 YEAR 2020	36 MINUTES
@ MMWWF-5 YEAR 2020	23 MINUTES
FINE AIR DIFFUSERS	120 FLEXIBLE MEMBRANE DISC

SECONDARY CLARIFIERS

TOTAL AREA	4250 SF
2-52' DIAMETER CIRCULAR	
TOTAL FLOCCULATION WELL AREA	500 SF
2-18' DIAMETER CIRCULAR	
OVERFLOW RATE	
@ MMDWF-5 YEAR 2020	235 G/SF/D
@ MMWWF-5 YEAR 2020	376 G/SF/D
@ PDAF-5 YEAR 2020	965 G/SF/D
@ PIF-5 YEAR 2020	1152 G/SF/D

RETURN SLUDGE PUMPS

AIR LIFT (2) EACH	300 GPM
FLOW MEASUREMENT OVER V-NOTCH WEIR	

WASTING PUMP

SUBMERSIBLE TORQUE FLOW	150GPM @ 20'
BATCH WASTE FROM SUMP WITH VOLUME OF 18000 GALLONS	

BLOWERS

POSITIVE DISPLACEMENT BLOWERS	
BLOWER 1,CHANNEL AIR	150SCFM @ 4.5 PSI
BLOWER 2, AIR LIFT PUMPS AND GRIT REMOVAL PUMP	150 SCFM @ 5.5 PSI
BLOWER 3, DIGESTER	120SCFM @ 9.0 PSI

CHLORINE CONTACT CHAMBER

VOLUME	64000 GAL
CONTACT TIME	
@ MMDWF-5 YEAR 2020	92 MINUTES
@ MMWWF-5 YEAR 2020	58 MINUTES
@ PDAF-5 YEAR 2020	22 MINUTES
@ PIF-5 YEAR 2020	19 MINUTES
LENGTH/WIDTH RATIO	225/4 = 56
FLASH MIXER	3 AXIAL FLOW S.S. BLADES
MOTOR	1725 RPM

DISINFECTION

FLOW PROPORTIONAL V-NOTCH CHLORINATOR	
CAPACITY	150 LB/DAY W 2-4 GPM CARRIER AT 5PSI
FIXED RATE BY-PASS CHLORINATOR	ACTIVATED BY HIGH FILTER PUMP WET WELL LEVEL

SAND FILTRATION

TOTAL AREA	640 SF
DIMENSIONS 16' X 40'. MEDIA 11" DEEP.	
AUTOMATIC BACKWASHING SAND FILTER WITH TRAVELING BRIDGE	
DESIGN HYDRAULIC LOADING RATE AVE. - PEAK	2 GPM/SF - 5 GPM/SF
HYDRAULIC LOADING RATE	
@ MMDWF-5 YEAR 2020	1.1 GPM/SF
@ MMWWF-5 YEAR 2020	1.7 GPM/SF
@ PDAF-5 YEAR 2020	4.5 GPM/SF
@ PIF-5 YEAR 2020	5.3 GPM/SF

AEROBIC DIGESTION

VOLUME	70,000 GALLONS
PRIMARY SOLIDS LOADING	
YEAR 2000	7 LBS/ 1,000 CF
YEAR 2020	11.3 LBS/ 1,000 CF
DETENTION TIME	
YEAR 2000	52 DAYS
YEAR 2020	33 DAYS

LIME STABILIZATION

VOLUME	22,000 GALLONS
DETENTION TIME	24 HRS AT PH 11.5
LOADING IN YEAR 2000	
DIGESTED PRIMARY SLUDGE, 190 LBS @ 1.7 %	1340 GPD
WASTE ACTIVATED SLUDGE 175 LBS @ 0.7%	3000 GPD
LIME SLURRY 64 LBS @ 6%	120 GPD
LOADING IN YEAR 2220	
DIGESTED PRIMARY SLUDGE, 295 LBS @ 1.7 %	2080 GPD
WASTE ACTIVATED SLUDGE 275 LBS @ 0.7%	3276 GPD
LIME SLURRY 100 LBS @ 6%	200 GPD

PRIMARY SLUDGE PUMPS

PLUNGER TYPE SLUDGE PUMPS	2
CAPACITY	86 GPM @ 25'
FLOW MEASURED AS (TOTAL STROKES/STROKES PER MINUTE)xGPM	

SLUDGE STORAGE PONDS

VOLUME POND 1	50000 GALLONS
VOLUME POND 2	90000 GALLONS
STORAGE	140000 GALLONS
YEAR 2000 4%	108 DAYS
YEAR 2020 4%	70 DAYS

FLOW METERING

INFLUENT - 9" PARSHAL FLUME	3.5 MGD
EFFLUENT V-NOTCH WEIR	0-9 MGD
ULTRASONIC LEVEL TRANSMITTER (2)	0-24 INCHES,
CONTINUOUS RECORDER, INDICATOR, TOTALIZER AND SAMPLER	4.20 MA

SAMPLERS

DIVERTER TYPE SAMPLERS (2)	24 SAMPLES/DAY
SELF PRIMING PUMPS (20)	5.8 GPM
LOCATION	INFLUENT CHANNEL EFFLUENT MANHOLE

STAND-BY GENERATOR

CAPACITY	100 KW
TOTAL RUNNING LOAD	57 KW
AUTOMATIC TRANSFER SWITCH	
EMERGENCY OPERATIONS	TRICKLING FILTER PUMPS 1, 2, & 3 SLUDGE PUMP #1 BLOWERS 1 & 2 DIGESTER MIXER CRITICAL LIGHTING FLASH MIXER

OUTFALL

LENGTH	650 FT/80 FT
MATERIAL	CONCRETE/STEEL
SUBMERGENCE	6 FT
MANHOLES	3

WASTEWATER FACILITIES PLAN
PLANT CAPACITY SUMMARY

CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
Figure 4-1A
CLACKAMAS COUNTY, OREGON

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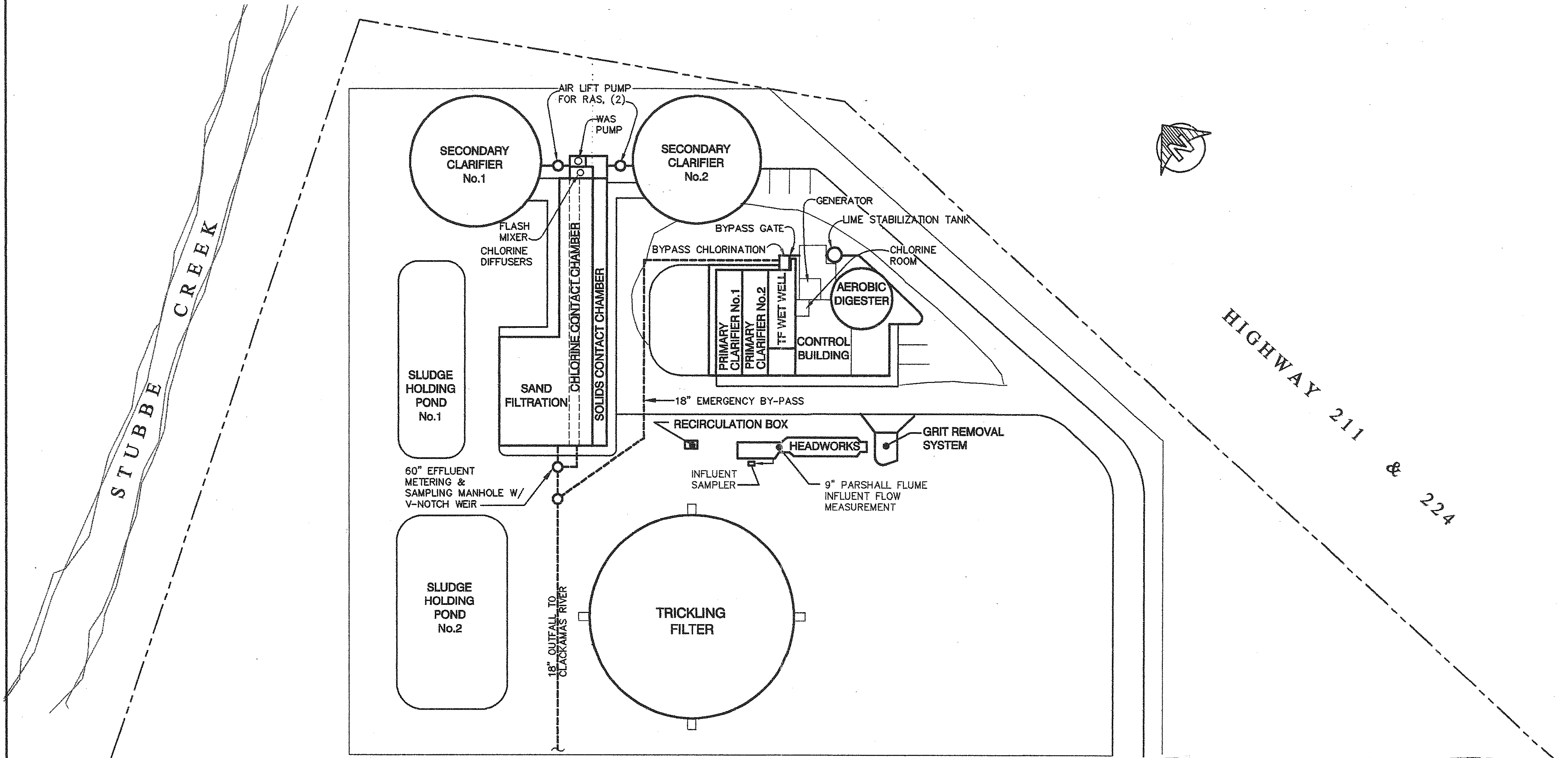

CITY OF ESTACADA
WASTEWATER FACILITIES PLAN
EXISTING WASTEWATER TREATMENT PLANT - Figure 4-2
CLACKAMAS COUNTY, OREGON
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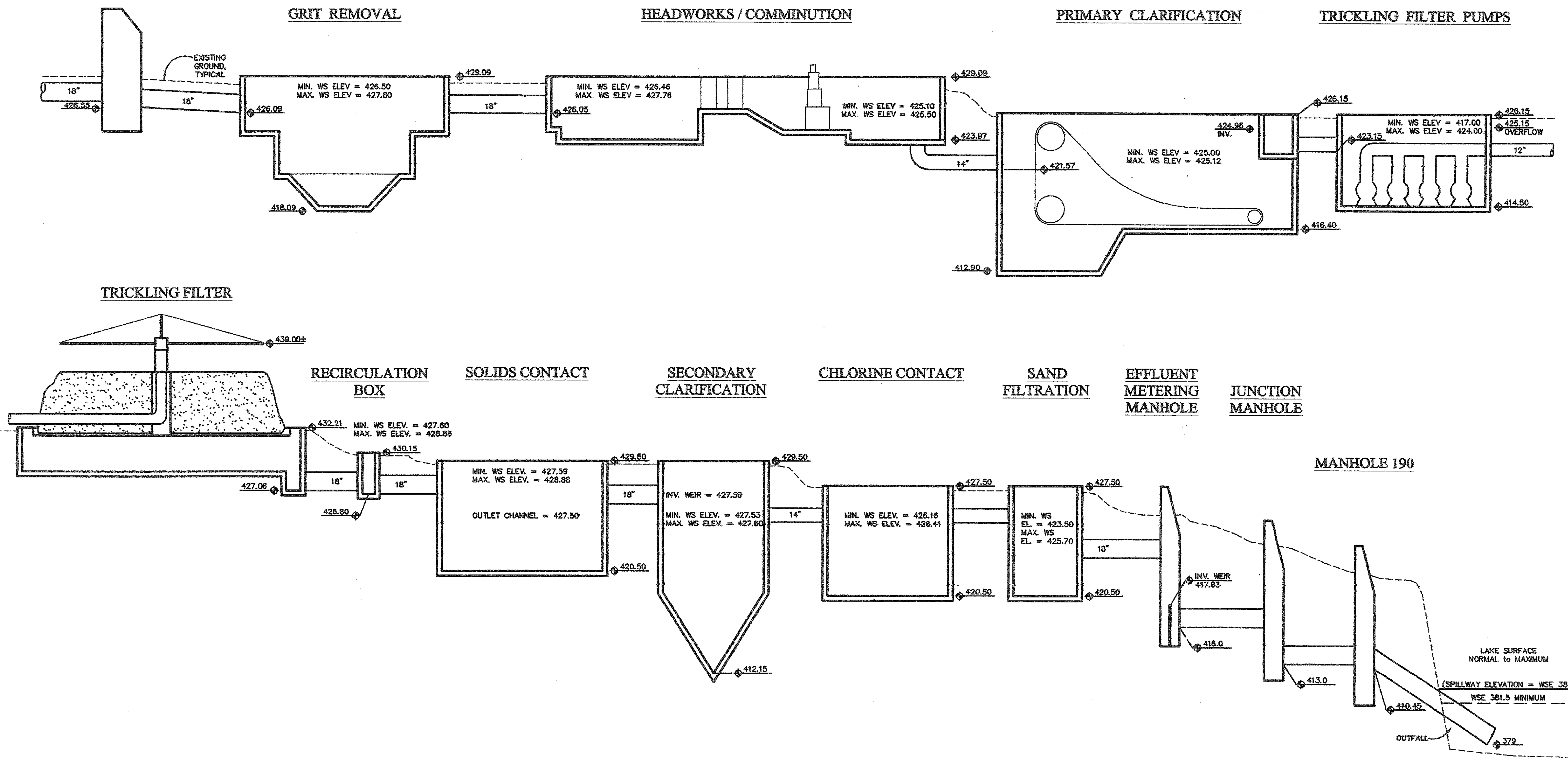

HYDRAULIC PROFILE

NTS

CITY OF ESTACADA
WASTE WATER
HYDRAULIC PROFILE
Figure 4-2A
CLACKAMAS COUNTY, OREGON

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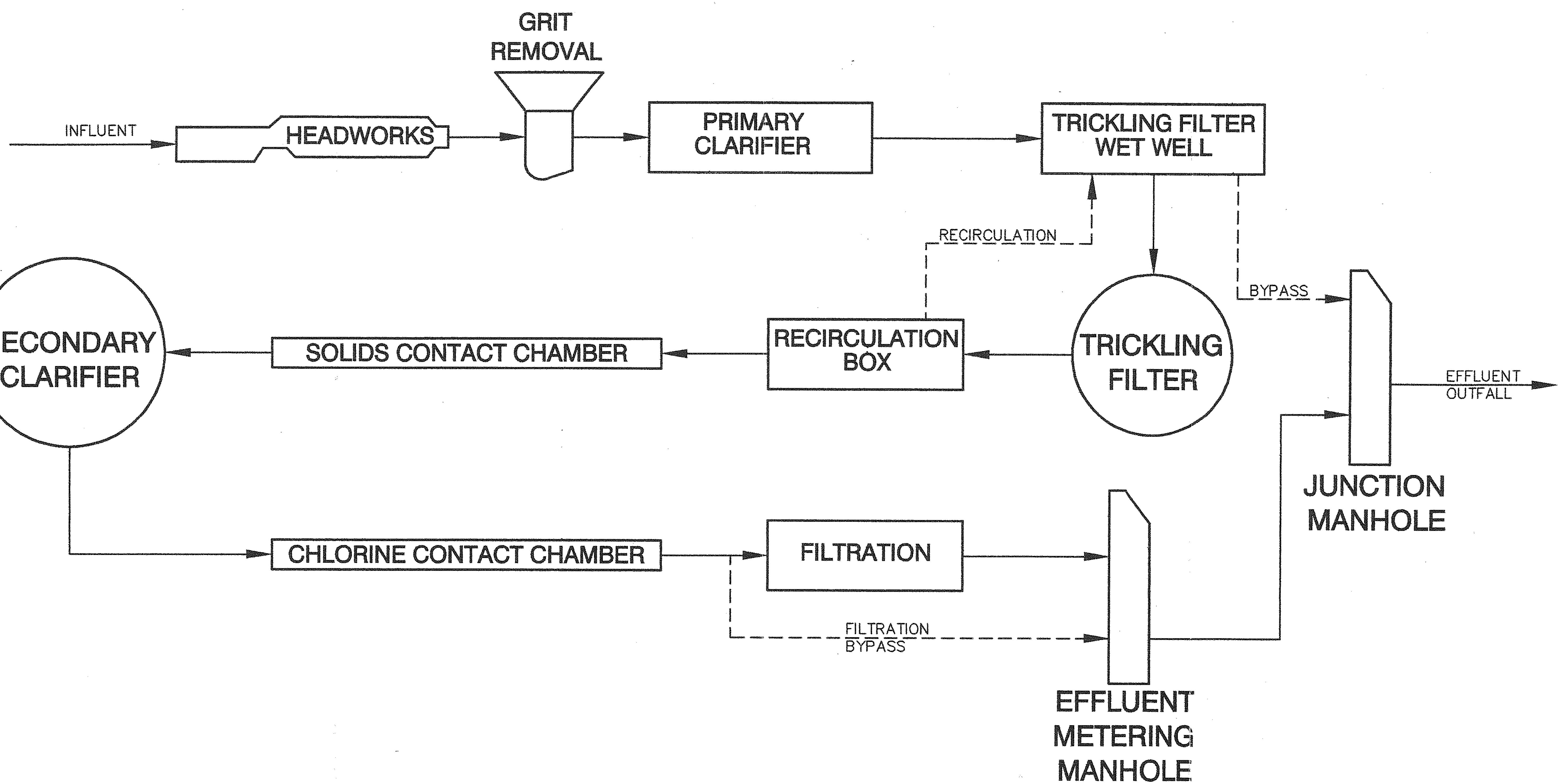

CITY OF ESTACADA

WASTEWATER
FACILITIES PLAN

LIQUID STREAM PROCESS - Figure 4-2B

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The trickling filter performance is dependent upon load and recycle rate. The recycle rate is adjusted by using a slide gate in the trickling filter effluent splitter box. The efficiency increases with increased recirculation. Up to a ratio of 2.5 to 5.

Solids Contact

Following the trickling filter and filter recycle, the trickling filter effluent is mixed with the settled sludge from the secondary clarifier in the Solids Contact Channel. Contact with return activated sludge from the secondary clarifiers serves to stabilize suspended and dissolved BOD.

In the contact tank, the sludge is aerated for less than one hour beginning the flocculation of the suspended solids and bacterial assimilation of soluble BOD.

When influent flow exceeds 1.4 MGD, contact time in the solids contact basin is less than 20 minutes and there is a loss of removal efficiency in the solids contact channel. The effect of contact time, and Mixed Liquor Suspended Solids concentrations (MLSS) on removal efficiencies is discussed in more detail in Chapter 5. In general shorter contact times can be compensated for increasing the MLSS during the wet weather months.

Secondary Clarifiers

There are two secondary clarifiers which together have a conservative overflow rate of 1,270 gallons per day per square foot (gpd/ft^2) at 4.6 MGD. The clarifiers have a flocculating center well of 18 ft. in diameter. The contact tank effluent has a high level of dispersed solids, and this type of clarifier has been found to be effective in maximizing solids capture.

Chlorine contact

The chlorine contact chamber is separated into two chambers which can be operated as one 120 ft. long x four ft. wide channel with a total capacity of 64,000 gallons. The easterly channel can be operated singly to provide 60 minutes of contact time up to 0.50 MGD.

The two channel westerly basin will provide 1 hour detention time for flows up to 1 MGD. The three-channel chlorine contact chamber provides an hour of contact at 1.5 MGD and 20 minutes at 4.5 MGD.

Sand Filter

The final process to the plant flow is effluent filtration through a 640 square foot, 11 inch deep, tertiary sand filter. The purpose of the sand filter is to assure that the plant effluent total suspended solids (TSS) limits will be met. The filter is designed to remove 60 % of the suspended solids at an average hydraulic loading of 2 gpm/ ft. Suspended BOD removal is also facilitated by the filters especially when there is a significant proportion of BOD in the suspended form.

4.1.2 Performance

Overall plant performance has been excellent and is summarized in the following tables. The plant effluent averages less than 10 mg/l Biochemical Oxygen Demand (BOD) and less than 5 mg/l Total Suspended Solids (SS). Average discharge of BOD and TSS is 40 % to 70% of Total Mass Discharge Limits (TMDLS).

Table 4-1: Summary of Wet Weather Plant Performance

Effluent January 1- April 31	BOD_5				SS			
	Monthly Ave.			Max Day	Monthly Ave			Max Day
	% removal	mg/l	lbs	lbs	% removal	mg/l	lbs	lbs
1997								
January	89	9	80	135	94	2	27	108
February	90	8	66	113	95	3	24	50
March	89	9	94	159	88	4	53	128
April	91	10	57	85	97	2	15	25
1997/98								
November	96	5	27	49	94	5	26	34
December	95	6	35	75	96	4	28	90
January	91	5	57	71	87	5	62	111
February	92	7	54	119	94	5	39	67
March	89	12	82	148	88	8	82	110
April	90	11	63	11	93	7	39	50
1998/1999								
November	95	6	52	176	93	5	43	176
December	92	5	74	225	93	3	55	157
January	91	8	76	188	94	4	38	81
February	86	9	99	157	92	4	43	86
March	87	11	87	161	89	7	53	88
April	92	11	55	81	95	6	28	44
Averages	91	8	66		93	4	41	
Max Day (TMDL = 180 lbs)				225				176

In the winter, plant performance has produced average BOD reductions exceeding 90% and BOD concentrations averaging 8 mg/l and seldom over 10 mg/l. Infrequently the TMDLs have been exceeded. The instances of non-compliance have occurred only when the plant flows were more than twice the permit base due to the stress placed on the plant by high hydraulic loadings.

The difficulty of meeting maximum day pounds when peak day flows approach 4 MGD emphasizes the necessity of implementing an aggressive inflow and infiltration(I/I) control program

Plant performance in the summer months has averaged 96% removal of BOD and 97% TSS.

TSS. Dry weather mass discharge limits have been exceeded in the Spring when flows are higher than average.

Table 4-2: Summary of Dry Weather Plant Performance Data								
	BOD				TSS			
	Monthly Average			Max Day	Monthly Average			Max Day
	%	mg/l	lbs	lbs	%	mg/l	lbs	lbs
1997								
May	96	10	32	48	98	4	13	19
June	94	10	35	83	98	4	13	21
July	97	7	16	33	98	3	7	9
August	95	10	28	41	97	5	12	16
Sept	96	8	23	44	97	4	14	31
Oct	95	7	36	67	97	4	19	42
1998								
May	91	10	70	127	92	6	40	76
June	94	10	44	65	97	5	20	30
July	96	9	24	37	99	3	8	11
August	99	4	12	17	98	4	10	15
Sept	97	7	8	32	97	4	5	19
Oct	97	5	18	27	98	3	10	15
Ave.	96	8	29		97	4	14	
Daily Max	(TMDL = 90 lbs)			127				76

Table 4-3: Summary of Permit Noncompliance					
	Reported		Limit	Ave. Flow	BOD
November 1 - April 30	Parameter	lbs	lbs	MGD	mg/l
January 14, 1999	Daily BOD	188	180	1.63	14
February 1999	Monthly BOD	99	90	1.44	9
March 4, 5, 1999	Weekly BOD	144	135	1.56	11
May 1 - October 31					
May 1999	Monthly BOD	49	45	0.579	9
May 18, 19, 1999	Weekly	75	68	0.82	11

In May of 1999 the weekly and monthly mass loadings were exceeded for pounds of BOD discharged. BOD loadings during this time period were higher than normal, because of decant from the solids lagoons. Operational problems associated with full solids lagoons were compounded by higher than normal flows.

4.1.3 Capacity of Unit Processes

By analyzing the treatment processes separately it is possible to more accurately define the plant capacity. The loadings on individual processes as a fraction nominal capacities (NC) are given in Table 4-2. The individual liquids processes are generally within design ranges although the contact time in the solids contact is less than optimum at peak flows

Table 4-4: The Present and Projected Capacity of Liquids Process Units

		Present Flows (2000)			Projected Flows (2020)			Design Criteria
Loading		$MMDWF_{10}$ 0.8 MGD	$MMWWF_5$ 1.39 MGD	PIF_5 4.70 MGD	$MMDWF_{10}$ 1.0 MGD	$MMWWF_5$ 1.60 MGD	PIF_5 4.91 MGD	
Primary	Overflow Rate	695	1208	4086	869	1391	4260	1500 -2000 gals/SF/day
	N.C.	46%	81%	200%	58%	93%	284%	
Trickling Filter	Organic loading	13	13	13	20.5	20.5	20.5	25-40 lbs / 1000 CF
	N.C	50%	50%	50%	86%	86%	86%	
	Hydraulic Loading	118	205	696	148	237	727	230 -1400 gal/SF/day
		51%	89%	50%	64%	103%	52%	
Solids C.	Contact T.	46	26	NA	36	23	NA	20-120 minutes
	N.C	74%	67%	NA	96%	80%	NA	
Secondary	Overflow Rate	188	327	1105	235	376	1152	400-1200 gal/SF/day
	N.C	47%	82%	92%	59%	94%	96%	
Contact Basin	Contact Time	115 min	67 min	27 min	92 min	58 min	19 min	1 hr. @ avg 20 min. @ PIF
	N.C.	20%	80%	100%	25%	100%	122%	
Filter	GPM/SF	0.9	1.5	5.1	1.1	1.7	5.3	2.0 - 5.0 gpm / SF
		45%	75%	102%	55%	85%	106%	

*NA :Not Applicable: Solids contact channel is insensitive to PIF due to the sludge inventory in the secondary clarifiers.
*NC: Nominal Capacity

In the preceding table the design criteria for contact time in the solids contact channel are given as 20 minutes to 2.0 hrs. a range based on data from similar processes. Nominal capacities are based on a minimum of 20 minutes for the wet weather flow period and an optimum of 45 minutes for dry weather. Longer solids contact time are necessary to meet the more stringent dry weather mass loadings.

Trickling filter design organic loading rates usually fall into four categories low, intermediate, high and roughing. Intermediate loading rates of 25-40 lbs BOD/ 1000 cubic feet are considered most appropriate for the combined TF/SC process at the Estacada treatment plant. The majority of the soluble BOD is removed by the trickling filter, with the solids contact channel acting as a polishing process to achieve quality effluent concentrations less than 10 mg/l.

In 2020 the secondary processes approach design capacity indicating that the treatment plant will need to be expanded. Using the design criteria cited above, the solids contact basin will be at 96% of capacity, and the trickling filter will be at 86% capacity.

At the projected MMWWF in 2020 the primary clarifiers will be at 86% of design and the secondary clarifiers at 88%. This performance is reasonable to expect since the projected MMWWF in 2020 will be equal to the average wet weather flow of 1.5 MGD for which the plant was designed.

Currently the plant is within design capacities except during excess flow events when peak flows stress the individual processes. Reduced efficiencies may make it difficult to meet maximum day mass loading limits, emphasizing the importance of reducing peak flows by targeting I/I control.

The projected effluent concentrations which are necessary for compliance with current discharge limitations are summarized in Table 4-3. The Three Basin Rule stipulates that there shall be no increase in mass discharge to the Clackamas River therefore these limits are not expected to change.

A critical period for meeting the permitted mass loadings is during the month of May when dry weather mass loadings apply , but the flows are more typical of a wet weather condition. As indicated in the previous section the stress on the plant may be compounded by carrying a high solids inventory.

The fact that solids recycle impacts effluent BOD concentrations is indicated by the fact that effluent BOD concentrations average 10.5 mg/l for the months of March through June. This problem can be controlled by providing increased solids storage or year-round disposal , and operating the secondary clarifiers with a consistent sludge blanket.

Table 4-5: Projected Organic Loads and Required Effluent									
		Influent Averages		45 lbs / day (dry monthly average)		90 lbs/day (wet monthly average)		180 lbs day (maximum day wet weather)	
YR	EDU	BOD lbs/day	SS lbs/day	MMDWF MGD	Required Effluent mg/l	MMWWF MGD	Required Effluent mg/l	PDAF MGD	Required Effluent mg/l
2000	1250	695	628	.81	6.66	1.39	7.76	3.92	5.51
2005	1401	778	704	.85	6.35	1.43	7.55	3.96	5.45
2010	1569	871	788	.90	6.00	1.48	7.29	4.01	5.38
2015	1758	976	883	.96	5.62	1.54	7.01	4.07	5.30
2020	1970	1094	990	1.02	5.29	1.60	6.74	4.13	5.22

The sand filter has greatly enhanced the ability of the plant to achieve consistently high suspended solids removal, and it has helped the plant comply with the required daily minimum removal rate of 85%. At the $MMWWF_5$ projected for 2020 the sand filter will be within design average hydraulic rates of 2 gpm /square foot.

At peak flows the hydraulic limit of the filter is reached and operating practice has been to split the effluent flow that goes through filtration. Because of significant infiltration and high peak flows, meeting the maximum day mass load limit requires effluent concentrations below 5 mg/l for BOD and TSS and it is necessary to filter as much of the effluent as possible. A notched gate at the end of the filter allows for a water surface elevation of over a foot over the filters before some of the effluent is bypassed without filtering.

Existing secondary treatment at the Estacada Plant is capable of producing the required high quality effluent under average future loading conditions. To maintain these high removal efficiencies at high flows consistently will be a challenge operationally, and may require some increased sampling and monitoring.

Trickling filter recycle rates, concentrations in the solids contact basin, and lagoon and digester decant schedules need to be optimized to assure plant performance especially under high hydraulic loadings. Operational strategies for maintaining consistently high organic removal efficiencies in the trickling filter and solids contact basin are discussed in Chapter Five.

Hydraulic loadings on the secondary processes at peak instantaneous flows have exceeded the present capacity of the trickling filter pumps. In order to provide secondary treatment to peak flows it will be necessary to enlarge the trickling filter pump capacity. This recommendation is discussed in detail in Chapter 6.

it will be necessary to enlarge the trickling filter pump capacity. This recommendation is discussed in detail in Chapter 6.

4.2 SOLIDS PROCESSES

4.2.1 Description

Solids Handling

The primary clarifiers are equipped with automatic valves for sludge removal during the programmed shutdown of the aerobic digester. The automatic valves are sequenced to fraw from each clarifier hopper. Normal operation calls for the digester air blower and the mixer to deactivate during sludge pumping. This permits the sludge to settle, retaining the solids during sludge additions to the digester. The automatic valves are sequenced to draw from each clarifier hopper.

Sludge is pumped from the clarifier hoppers to the digester by sludge plunger pumps located in the control building basement. There are two pumps for each clarifier, and each can pump a maximum of 90 gpm. Sludge withdrawal cycles can remove sludge up to five times per day and are set to maintain a concentration of not less than 2%, and not more than 6%. It is not practical to pump higher densities because of a 40 ft. suction line with elbows, fittings and valves which impede the sludge flow.

The Return Activated Sludge (RAS) is controlled by air lift pumps to the splitter box between the clarifiers, and from there directed to the head of the solids contact channel. The clarifier riser pipes and sludge piping configuration require a minimum velocity to remain clear. The minimum flow is 250 to 300 gpm, and at low flow conditions exceeds required return rates.

The RAS is measured by a V notch weir in the sludge splitter box. When wasting is required the slide gate to the Waste Activated Sludge (WAS) pump sump is removed. The WAS pump is a recessed -impeller, solids handling pump with a capacity of 200 gpm at 16 ft. TDH. The waste activated sludge can be pumped to the digester, but normal operating procedure is to send it to the lime stabilization process.

Aerobic Digestion

Solids are batch-treated in the Aerobic Digester. In general, the hydraulic retention time is maintained at a minimum of 40 days under aeration by limiting the outlet discharge to 1,800 gallons per day to lagoon storage or land application. When digester temperatures to less than 20 Degrees C the required detention time is 60 days which equates to a 1,200 gallons per day removal rate.

Since July 1995, the practice has been to lime stabilize all sludges before land application The primary sludge is pumped to the digester on a daily basis. Once a week, 7500 - 11,000 gallons are pumped to the lime stabilization tank.

Lime stabilization

The lime stabilization process greatly reduces the time required to stabilize sludges for land application. During this process , sufficient lime must be added to meet the pH and contact criteria as established by the Part 503 sludge regulations for Class B sludges. To meet these criteria, the pH of the sludge must be raised to 12 and maintained for at least 2 hours, followed by a minimum detention of 22 hours at a pH of 11.5 or greater.

Prior to initiation of the sludge stabilization process, lime and water are mixed in a 200 gallon chemical tank located adjacent to the stabilization tank. The slurry is agitated by a mechanical mixer to maintain a suspension. Lime can be added to the sludge as it is being pumped to the stabilization tank or mixed by the recirculation pump.

Sludge is pumped to the 22,000 gallon lime stabilization tank from the digester with one of two plunger pumps in the lower level of the main control building. Waste activated sludge is pumped directly to the stabilization tank.

Two to three feet of sludge from the digester are added to several days of waste activated sludge for stabilization. Following stabilization, the sludge is generally pumped to the storage ponds using the recirculation pump.

Lagoon Stabilization and Storage

The two sludge lagoons have a volume of 140,000 gallons and are used for stabilization and storage. Sludge is stored during the wet weather period by selectively decanting from the lagoons. Pond outlets are primarily for collecting supernatant and delivering it to the plant headworks.

4.2.2 Biosolids capacity and limitations

The addition of the lime stabilization process has greatly increased the flexibility of the biosolids digestion and stabilization process A single process meets both pathogen reduction and vector attraction requirements and the process is not controlled by the detention time in the digester. Operating procedure has been to pump primary sludge to the digester, and waste activated sludge to the lime stabilization tank daily. Primary digested sludge is then batch fed from the digester to the lime stabilization tank on weekly and mixed with the waste activated sludge for stabilization.

Following lime stabilization, the sludge is pumped to the lagoons for storage. It is possible to pump directly from the lime stabilization tank in emergency situations or when the lagoons are full. This is not the most economical as far as hauling because the average percentage solids out of the lime tank is less than the lagoon solids concentrations.

The timely application of sludge from the lagoons to the fields in the spring is necessary, and can be a problem in years when the wet season extends into May. Sludge is generally stored for six months from November 1, through April 30. Sludge production estimated for the annual biosolids reports and presented in Table 4-3; was based upon samples taken in the spring of the year.

Average concentrations are probably higher than indicated because after the water and thinner sludge is pumped from the lagoon there are several feet of very thick sludge in the lagoons which must be mixed with water to be pumped.

A solids balance was based on the primary sludge pumped from the digester, the secondary sludge which is wasted to the stabilization tank and the lbs of lime added for stabilization. This indicates that total sludge production is approximately 426 lbs/ day (155,490 lbs/year).

At a solids concentration of 4%, the optimum for storing and pumping without mixing with water, the capacity of the lagoons is estimated to be 46,704 pounds,or approximately 3.6 months. There is an obvious need for additional biosolids storage or land application of biosolids during the winter months.

Land application of the sludge on pasture land is limited by available land, agronomic (available nitrogen) loading rate, and site life as determined by the metals concentrations in the sludge. The number of pounds of available nitrogen a crop will require is determined by site location, crop type, method and frequency of harvesting, reserve soil nitrogen and crop irrigation practice.

The pounds of nitrogen produced per year are estimated from the concentrations of nitrate nitrogen and ammonia nitrogen in grab samples of the sludge. By permit, the pounds of available nitrogen which can be applied to areas of pasture, irrigated pasture and hay, and irrigated hay fields are 100, 120 and 140 pounds per acre per year respectively.

Table 4-6: Biosolids Production from Annual Reports

	Total Solids	Total Solids		Available nitrogen	Acres Applied	Nitrogen loading
	gallons/yr	lbs/yr	%	lbs/yr	#	lbs/acre
1995	429900	60951	1.7	1027	38.0	27.0
1996	218000	37453	2.1	436	23.4	18.6
1997	279000	90748	3.9	2641	25.7	102.8
1998	292000	120105	3.7	1622	20.2	80.3

The 1993 Biosolids Report included 81.1 acres of available pasture land, which are authorized by the Department of Environmental Quality to receive sludge and these authorizations should remain valid unless there are changes in crop practices or sludge characteristics. The City is in the process of updating permits for sites which have been permitted and had sludge applied in the past, but may have changed ownership.

Only a percentage of the permitted sites are actually available for biosolids application The availability of land is dependent upon property owners needs and varies from year to year.

Table 4-7: Sites Available for Land Application, 1999			
Ownership	Field	Site #	Acres Available
Guttridge	1	27A 27B	12
	2		11
	3		13
Shibley	32		20
	33		
	34		
			56

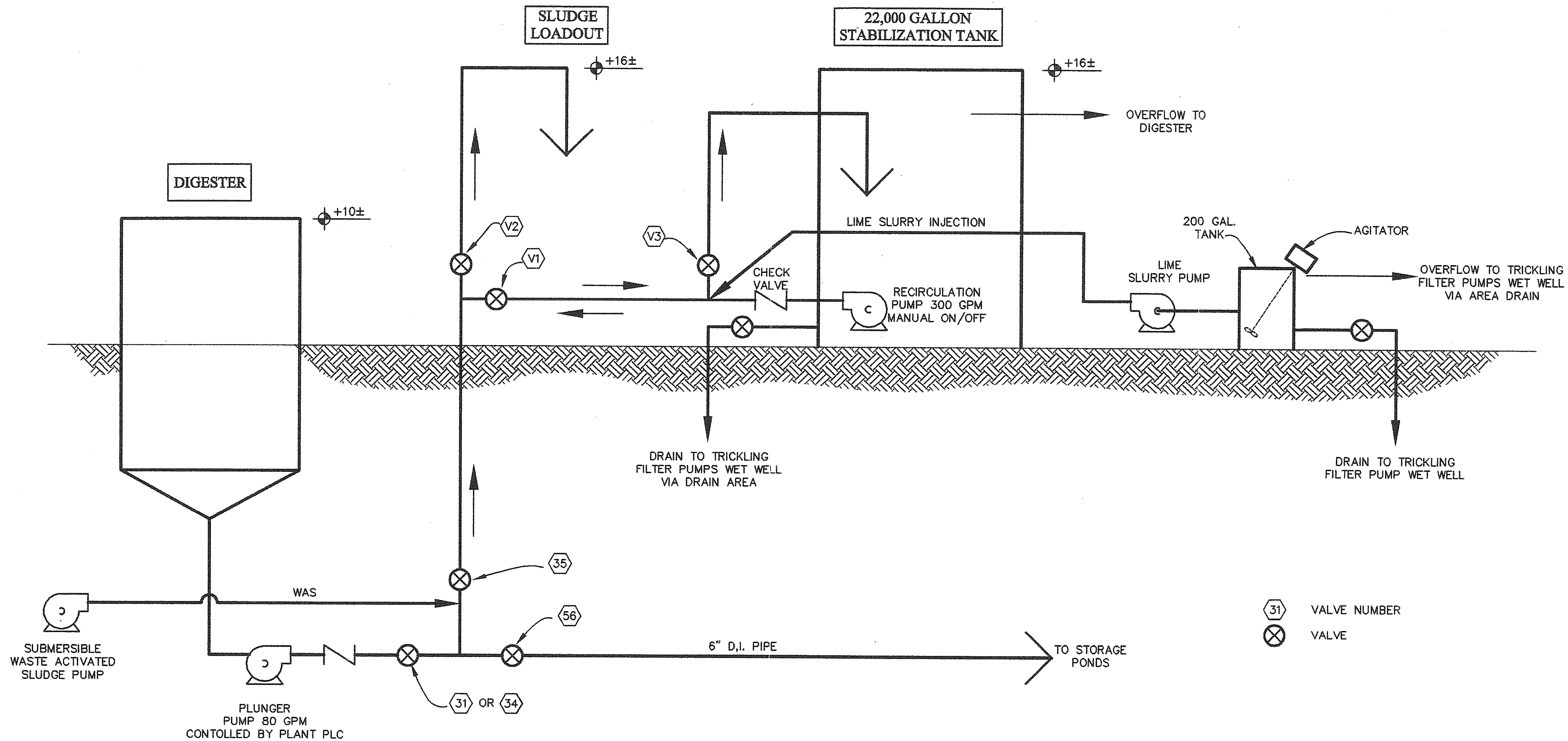
An estimate of the number of total number acres which are necessary for biosolids application is made by applying a factor for pounds of available nitrogen / lb solids, to the lbs of solids produced each year. From the biosolids production reports it is estimated that there is 1 lb of available nitrogen for every 53 lbs pound of solids. Currently there is an estimated 155,490 lbs of solids produced each year which translates into 29 acres at 100 lbs of nitrogen per acre. At a 2.3% growth rate this would translate into a need for 46 acres in 2020. There are presently 56 acres available.

Table 4-8 : Projected Land Application Needs			
EDUs	Solids Production lbs/year	Available Nitrogen lbs/ year	Acres
1250	155490	2934	29
1970	245052	4624	46
2500	310980	5868	59
3750	466470	8801	88
5000	621960	11735	117
6250	777450	14669	147

Currently the availability of land for the application of biosolids is dependent upon the on the land owners needs and the ability to deliver the biosolids when crop needs dictate its application. If the City owned its own land for biosolids application a more consistent operation would be possible.

Ideally City owned land could eliminate the need for large storage capacities for the biosolids at the treatment plant. To be able to apply solids year round at least half of these site would need to be in highland regions where the ground is not saturated during the winter months.

Estacada currently has permits for sites which allow for year round application. Application must comply with rules and guidelines indicated in OAR 340-50-005 to OAR 340-50-080. In general biosolids and domestic septage shall be applied at rates and methods which prevent the occurrence of runoff, erosion, leaching and nuisance conditions.



LIME STABILIZATION PROCESS FLOW SCHEMATIC

NTS

CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
Figure 4-3
CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

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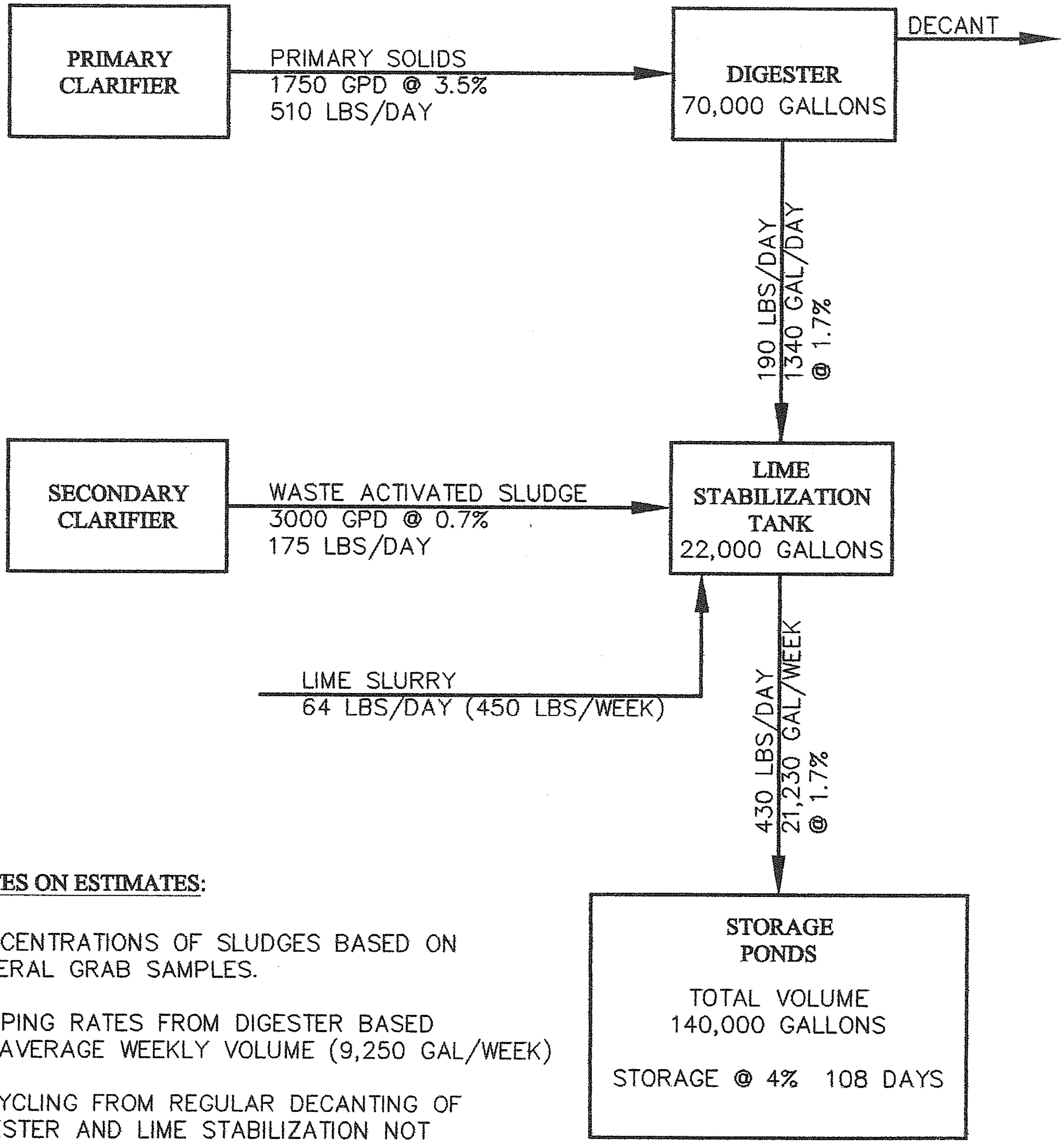
SOLIDS LOADING
YEAR 2000



NOTES ON ESTIMATES:

1. CONCENTRATIONS OF SLUDGES BASED ON SEVERAL GRAB SAMPLES.
2. PUMPING RATES FROM DIGESTER BASED ON AVERAGE WEEKLY VOLUME (9,250 GAL/WEEK)
3. RECYCLING FROM REGULAR DECANTING OF DIGESTER AND LIME STABILIZATION NOT METERED OR SAMPLED.

CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN UPDATE
Figure 4-4
CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

1301-4-5 05/30/2000 06:42

4.3 FLOW METERING

Influent flow is measured by Parshall flume equipped with an ultrasonic level sensor and signal transmitter. Flow measurement is periodically checked by measuring the flume levels. Influent flow measurement is accurate up to 3.9 MGD. Above 3.9 MGD the flume becomes partially submerged and loses accuracy.

Effluent is metered in the effluent flow manhole following tertiary treatment. Flow over a V-notch weir is measured by an ultrasonic level sensor and signal transmitter. Both transmitters are connected to continuous recorders, totalizers, and samplers by 4-20 MA signals. The accuracy of the effluent flow meter is checked by manually measuring the level over the weir.

When flow exceeds the capacity of the trickling filter pumps at 4.6 MGD, it flows over a bypass gate which is one ft lower than the walls of the trickling filter wet well. It flows into the emergency overflow chlorination wet well. A fixed rate chlorination is activated by the trickling filter wet well high level.

Following chlorination, the bypassed flow is transported through an 18 inch diameter line to a manhole downstream of the effluent metering manhole. Since the flow is not metered by the effluent flow meter, bypasses have been monitored by measuring the level over the bypass gate which acts as a rectangular weir.

The trickling filter wet well is equipped with a bubbler system which measures the wet well level. Currently the level is only recorded to the point at which overflow is by passed over the gate. The high level in the wet-well is the maximum reading on the strip charts. This set-point needs to be readjusted so that the level over the gate is measured. This will give a recorded measurement of any flow that is bypassed.

The bypassed flow is not sampled at the effluent metering manhole and grab samples must be obtained from the trickling filter wet-well overflow. These events are rare and, following pump revisions, will decrease in frequency.

CHAPTER 5: PLANT OPERATIONS

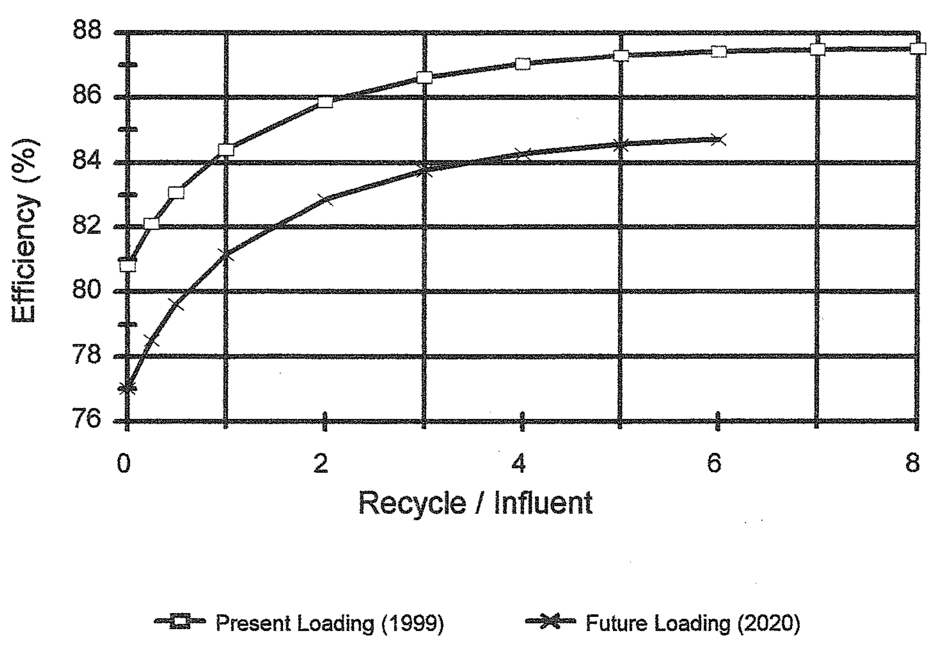
5.1 LIQUIDS PROCESSES

As plant loadings increase it will become increasingly important to optimize treatment plant performances and the efficiency of the secondary treatment processes. The following discussions of plant process are based on each process capability, and how the individual processes can be optimized.

5.1.1 Trickling Filter Efficiency

National Resource Council (NRC) equation for trickling filters was developed empirically in the 1940's from operating records of trickling filter plants. The NRC equation predicts an increase in removal efficiencies with increased recirculation rates as shown in Figure 5-1: Trickling Filter Efficiency / Recirculation Rate . The effect of increased recirculation on trickling filter efficiency is most pronounced up to recirculation ratio of 2 , Recycle / Influent.

Figure 5-1: Efficiency / Recirculation
NRC Equation



5.1.2 Solids Contact Basin

Maintaining consistent removal efficiencies in the solids contact basin is dependent upon optimizing control parameters such as Mean Cell Residence Time (MCRT), Food/Microorganism ratio (F/M) and mixed liquor suspended solids (MLSS) and maintaining them. Presently the operators rely a great deal on visual observation to estimate the sludge age.

Wasting rates are adjusted to maintain a constant sludge blanket in the clarifier. This may work as long as loading is consistent. When flows and loadings are subject to change, then it is important to have more quantitative measurements of operating parameters. This is especially important in the Spring and Fall when there is a great deal of variability in the influent loading rate.

In an activated sludge system the F:M ratio varies with detention time and process design. The trickling filter activated sludge process seems to work best from 0.3-0.5. At present maximum loading rates a MLSS concentration of 3532-5887 mg/l is implied. As loadings increase a correspondingly higher mixed liquor is needed. At maximum loading conditions in 2020 the range of mixed liquors is 5730-9550 mg/l.

The second operating factor, mean cell residence time (MCRT) expresses the average time a microorganism spends in the activated sludge process. By using the MCRT, the operator can control the F:M and to a certain degree the type of organisms which predominate in the system.. Manual of Practice 11 published by the Water Environment Federation gives the range MCRT for TF /SC combined processes as 0.5 to 2 days (using the volume of the aerated channel only).

The best MCRT for the plant must be determined experimentally and may vary from season to season.. For example MCRT is one of the parameters operators can use to control nitrification , because a high MCRT will favor nitrifiers, especially in the Spring and Summer months

The following curves of soluble BOD removal in the aerobic contact basin are based upon first order kinetics in a plug flow reactor. The reaction rate coefficient K_{20} in L / mg / min is based upon grab samples taken at the plant for trickling filter effluent and final effluent. [4]

Wet weather and projected flows will reduce the detention time in the solids contact basin. High average flows may be compensated for by adjusting the MLSS within the ranges predicted by F/M and MCRT. The negative effect on the efficiency may be partially compensated for by increased MLSS concentrations.

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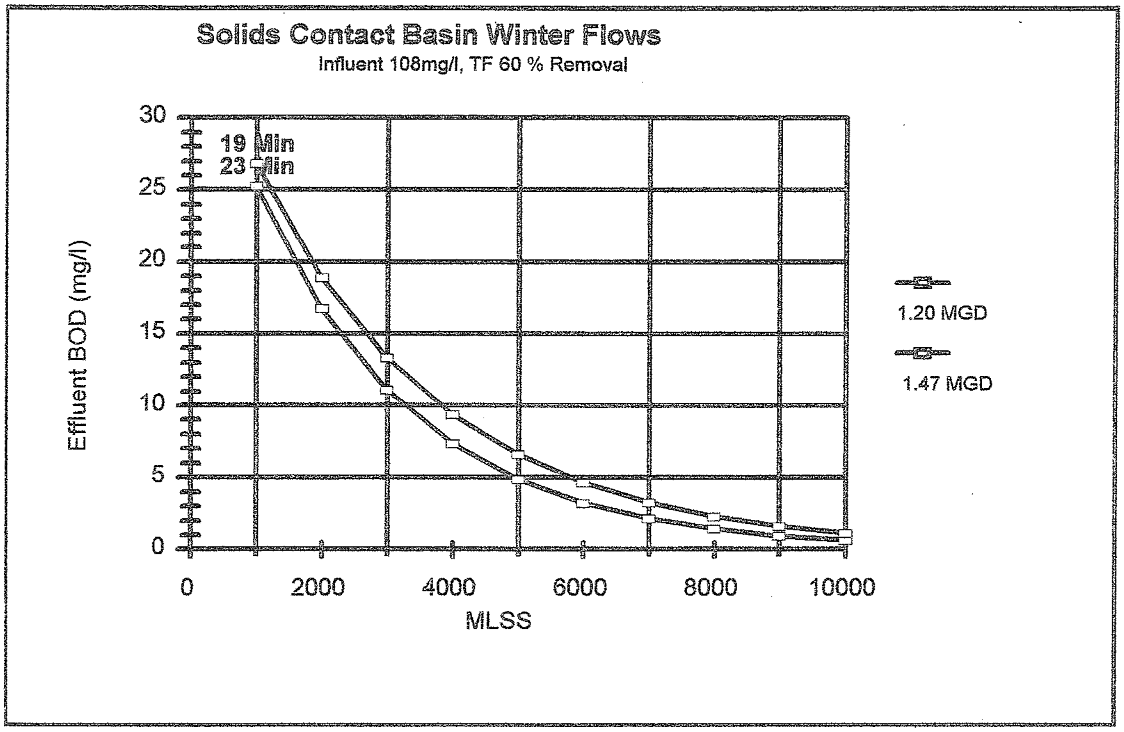
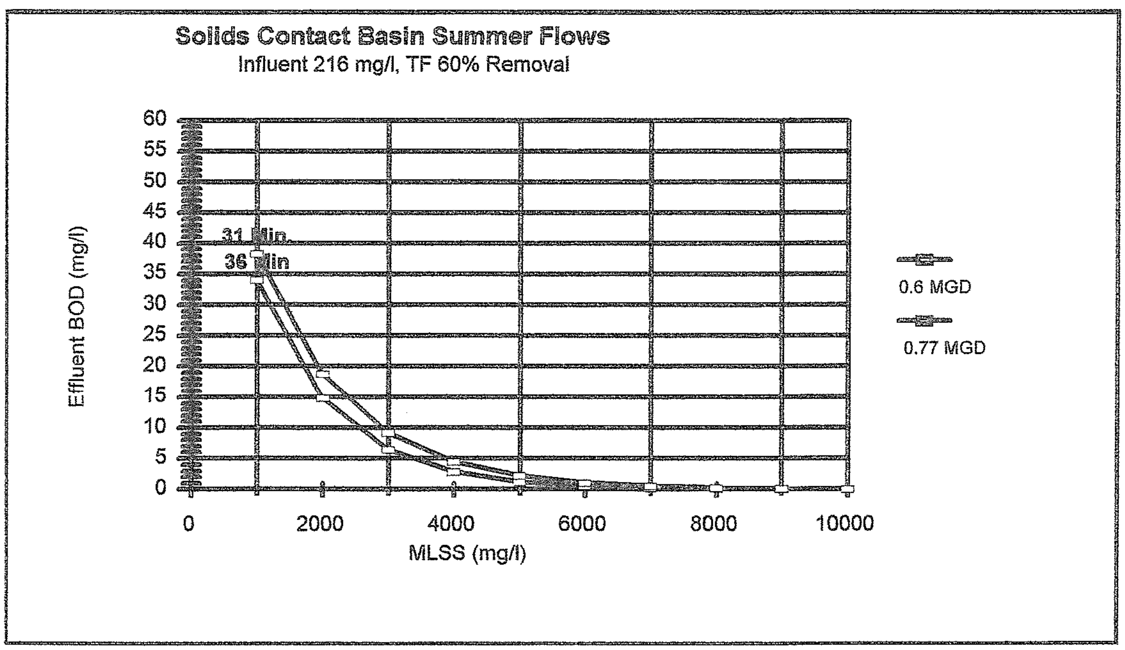
[4] Effluent / Solids Contact Influent (C/C_0) from grab samples, MLSS of 2610 mg/l, Detention time of 43.5 minutes, an average summer temperature of 18.5 Degrees C, winter average temperatures of 14 $^\circ$ C . $\ln(C/C_0) = (-K_{20}\,\theta^{(T-20)}\,\mathrm{MLSS})\,t$


Solids Contact Basin Winter Flows
Influent 108mg/l, TF 60 % Removal
19 Min
23 Min
Effluent BOD (mg/l)
30
25
20
15
10
5
0
0
2000
4000
6000
8000
10000
MLSS
1.20 MGD
1.47 MGD


Solids Contact Basin Summer Flows
Influent 216 mg/l, TF 60% Removal
31 Min.
36 Min
Effluent BOD (mg/l)
60
55
50
45
40
35
30
25
20
15
10
5
0
0
2000
4000
6000
8000
10000
MLSS (mg/l)
0.6 MGD
0.77 MGD

The trickling filters promote the growth of snails which accumulate in the solids contact channel. An accumulation of close to a foot was measured out of a total channel depth of 7.75 feet. The biodegradation of this biomass contributes to the dissolved BOD in the solids contact channel effluent. Regular cleaning of this channel should have a positive impact on the treatment plant effluent.

5.1.3 Sand Filters

Effluent filtration has resulted in effluent suspended solids concentrations which average 4 mg/l in summer and winter. BOD removal parallels suspended solids removal and is dependent on the proportion of solid/dissolved BOD in the secondary effluent. Filtration removes the pin floc which may be a problem in the secondary clarifiers.

The treatment plant operators have adopted a strategy of manually back washing the filters daily, even when there is not sufficient head loss to initiate a backwash automatically. The filters are periodically super-chlorinated and drained at least once a year to remove filter balls.

In January 1999 the sand media was completely replaced at a total cost of $4,410 dollars. As with the media in a sand filter for a water treatment plant, it is important that the sand be uniform with a uniformity coefficient of less than 2.0.

There is a reliance on the sand filter during wet weather periods when effluent concentrations close to 5 mg/l are required for compliance with effluent mass limits for both BOD and TSS. On day when the flows are greater than 4.0 MGD and exceed the hydraulic capacity of the filters, it is necessary to filter as much of the effluent as possible. A notched gate at the end of the filter allows for a water surface elevation of over a foot over the filters before bypassing..

5.2 POLYMER

The use of a polymer to aid secondary settling should be investigated. There are two situations where this application might be beneficial. Under high hydraulic loading conditions increasing the floc formation and settling of suspended solids in the secondary clarifiers would reduce the loading on the sand filters.

Polymer might also be used to aid in settling the pin floc which is often observable in the secondary clarifiers. It is recommended that the operators do some jar testing of polymers on the influent to the secondary clarifiers under various conditions. Polymer representatives are more than willing to provide information and samples for testing.

5.3 SOLIDS PROCESS DEFICIENCIES

Under present loading conditions the sludge lagoons are full in the spring and must be continually decanted back into the plant, increasing the concentration of what is left in the lagoon. The decant process significantly impacts the loading rates on the plant processes. Increased BOD in the decant from the lagoons contributes to higher than average effluent BOD concentrations in March and April.

Solids recyle from the lagoons contributes to an elevated solids inventory throughout the plant. Historically some of these solids have been stored in the secondary clarifiers during the winter by increasing the depth of blanket. The secondary sludge blanket may approach 10 ft in the spring when the solids inventory throughout the plant is high. This is not desirable and may lead to violations.

The potential for violating permitted mass loadings is especially high in May when the summer permit goes into effect and warmer water temperatures may lead to increased nitrification and bulking problems.

There are no mixers in the storage lagoons and the solids form a blanket several feet deep of very thick sludge at the bottom of the lagoons. It has been necessary to add water, and mixing is accomplished by recirculating the biosolids through a portable pump on the berm's edge.

CHAPTER 6: RECOMMENDED PLANT IMPROVEMENTS

6.1 IMPROVEMENTS TO THE LIQUIDS TREATMENT PROCESSES

6.1.1 Pretreatment

Prescreening

Screening the wastewater stream is recommended to remove coarse solids from entering the wastewater treatment facility and protects pumps valves and other appurtences. It is also very important in removing plastics and other material which may travel through the process. These materials are unsuitable for land application of biosolids.

The screenings unit would be placed in a channel preceding the degritter. A channel 15 ft. X 4'-2" X 4' deep, would need to be constructed for this purpose. The screenings unit screens the influent, and washes, dewaters, and compacts the solids. The solids are transported up an inclined conveyor to small dumpster or barrel which sits on a channel cover downstream of the equipment.

Table 6-1: Prescreening	
Screen (6.5 MGD Capacity)	$75,000
Channel	$35,000
Influent Sampler and associated piping	$2,000
Grit Drop Drop Box	$4,000
Estimated Costs	$116,000

Degritter

The grit conveyor has had several motor replacements due to exposure to wet and freezing conditions. It is recommended that an adequate cover be installed.

6.1.2 Trickling Filter

The peak treatment plant flow in 2020 is projected to be 5.7 MGD (3960 gpm) The trickling filter pumps have a combined capacity of 3200 gpm and would need to be enlarged to insure that all the influent receives secondary treatment. The existing distributor cannot handle this flow without being modified.

Higher flow capacity would require larger orifices on the secondary arms. At low flows all the flow is through the primary arms and a minimum head loss is required to start the distributor; therefore the primary arms cannot be adjusted and still maintain a 300 gpm minimum flow capability. Increased flow would increase the energy lost to friction in the piping system to the trickling filter, amounting to an estimated additional head loss of 6 ft .

To provide the firm capacity to pump peak instantaneous flows i.e with one of the largest pumps off line, it is necessary to upgrade three of the existing pumps. One of the upgraded pumps should be provided with a VFD to give it increased range and minimize surges in the secondary system.

Table 6-2: Pump Improvements Cost Estimate		
	Existing	Upgrade of two existing
	gpm	gpm
Pump #1	300	300
Pump #2	500	1600
Pump #3	1200	1600
Pump #4	1200	VFD(1,000-1600)
Total Capacity	3200	5100
Firm Capacity		3500
Cost of Pumps		$23,000
Variable Frequency Drive		$12,000
Cost of Distributor Modifications		$20,000
Total Estimated Construction Costs		$55,000

Trickling filter recirculation is currently adjusted manually with the throttling valve on the eight inch recirculation line to the effluent launderer of the primary clarifier. With the installation of a motor actuated valve and position sensor the operator will be able to monitor and control the position of this valve remotely.

6.1.3 Secondary Processes

The diffusers in the solids contact channel should be scheduled for inspection and replacement. There is adequate dissolve oxygen levels in the channels but mixing is uneven. All of the diffusers could be replaced at an estimated cost of $22,000.

Being able to control the mixed liquor suspended solids concentrations in the solids contact chamber is important to the efficiency of this channel. The return rates form the secondary clarifiers control the proportion of solids in the contact basin and secondaries. Operationally there is a need for more control over the return activated sludge (RAS) flow.

Control over RAS rates is achieved with the use of centrifugal solids handling pumps with high efficiency motors which can be controlled with a variable frequency drive (VFDs). The submersible pumps would be placed in the existing manholes. The variable frequency drives and motor control panel could be placed in the existing control building.

Table 6-3: RAS Pumps	
Submersible pumps with high efficiency motors (2)	$12,000
VFDs (2)	$16,000
Electrical	$4,000
Flow Meters (2)	$2,000
Construction Cost	$34,000

6.1.4 Dechlorination

The mixing study completed in 1998 concluded that the chlorine dosage required for effective disinfection leaves an effluent residual exceeding the chronic and acute toxicity limits within the mixing zone under controlled river conditions. This study is presented in Appendix II.

Of the available options, dechlorination by the addition of liquid sodium bisulphite is the solution which is recommended. A flow paced system can assure elimination of effluent residual with flow variations, and redundancy can be provided at a reasonable cost.

The current effluent chlorine analyzer monitors the plant effluent continuously. It is recommended that this analyzer be supplied by a new sample pump from the contact basin to allow the operators to monitor the pre-dechlorination residual.

Table 6-3: Cost of Dechlorination	
Chemical feed Pumps 2	$5,600
Bisulphite Analyser	$5,800
Chemical Crocks (2)	$700
Piping, valves and fittings	$2,000
Structural revisions/concrete	$5,000
Electrical power / wiring, L.S.	$2,500
Total Construction	$21,600

6.1.5 Conversion to Hypochlorite

The City of Estacada plans to switch from chlorine gas to hypochlorite for disinfection. Hypochlorite is the recommended option because of safety and operational considerations. Chlorine gas is a highly toxic gas which if not handled properly can injure or kill plant personnel and may require evacuation of facility neighbors. Safety protocols include the use of the buddy system and a self-contained breathing apparatus (SCBA)

Hypochlorite is considered a corrosive material and does not carry with it the same risk to personnel as does chlorine gas. As a solution it can be offloaded to storage tanks and pumped to the point of application with minimum risk of exposure to chlorine fumes.

The switch from chlorine gas to hypochlorite is made more economical by the fact that the Estacada water treatment plant will be switching to hypochlorite in the immediate future. Transportation is the largest fraction of the chemical costs, and the City can save on the cost per gallon of hypochlorite by ordering larger and more consistent volumes.

Estimated dosages are based on current average use. These dosages are applied to projected maximum month wet weather flow (MMWWF) in 2020 to obtain future needs. It is assumed that the neat chlorine is delivered at 12 %.

Table 6-4: Hypochlorite Volumes

	Winter Flows (1.7 mg/l)			Summer Flows (1.3 mg/l)		
	gpd	gal / week	gal / mo	gpd	gal / week	gal /mo.
Current	15	104	456	4	28	122
Projected	21	149	646	8	58	254

It is proposed that the existing chlorine building be converted for hypochlorite use The existing room could hold 2 polyethylene tanks, 2 feet in diameter and 5 feet high, with a combined volume of 235 gallons. At current winter flows this volume would be sufficient for biweekly deliveries and 10 day deliveries at projected flows.

The neat hypochlorite would be delivered into the existing 1 1/4 inch lines along with carrier water for delivery out to the contact basin. Potable water is used for the dilution water to minimize scale formation in the pipes. The velocity imparted by the carrier water should eliminate the accumulation of gas in the feed lines.

The gas it produced is the largest problem with hypochlorite. The neat hypochlorite metering pumps should not be placed in a suction lift condition where they can pull dissolved gas out of solution and air lock the pumps. The suction of the diaphragm pumps should be equipped with valves which release any accumulated gas pressure and the piping between the discharge of the pump and the dilution water should be minimized. It is also recommended that gate valves be used in place of ball valves.

Table 6-5: Cost of Hypochlorite Conversion	
Covered Tanks (2) 235 gallons	$2,000
Chlorine Metering Pumps (2)	$3,200
Piping and assorted valves	$1,500
Water flow meter and valve	$1,500
Containment Curbing	$1,500
Electrical	$2,000
Installation Cost	$11,700

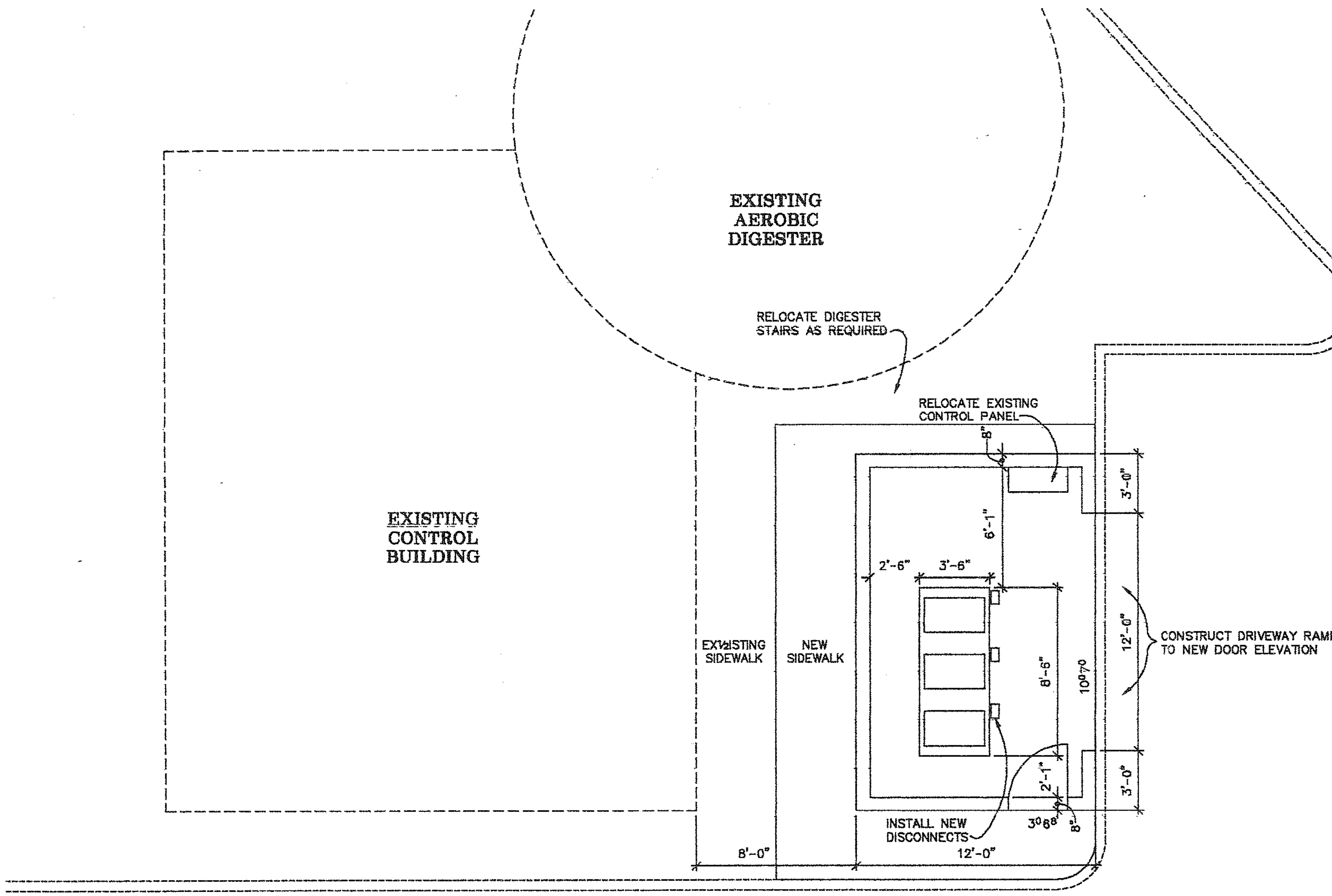
6.1.6 Blower building

There are three blowers, each with a capacity of 150 SCFM. The solids contact channel air diffusers are supplied by #1. Blower 2 supplies air to air lift pumps for the RAS and grit removal systems, and #3 serves the diffusers in the bottom of the digester.

It is recommended that the blowers be moved to a separate building due to concerns over the noise and heat levels in the current building. The building would be located next to the existing control building with a sidewalk in between as shown in Figure 6-1.

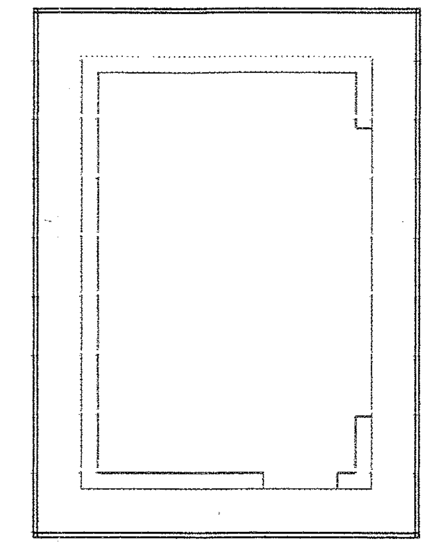
An additional blower is necessary to provide backup capacity for the solids contact channel and allow the return activated sludge (RAS) air lift system and grit removal systems to be supplied separately. The grit removal air system currently causes problems with the operation of the air-lift return activated sludge pumps when it starts up.

Table 6-6: Cost of Blower / Control Building	
New Blower Building and Existing Control Building	$55,000
Electrical power / wiring	$15,000
Relocate existing Equipment	$5,000
Purchase / install additional blower	$15,000
Covered Sidewalk	$6,000
Total Construction Cost	$96,000



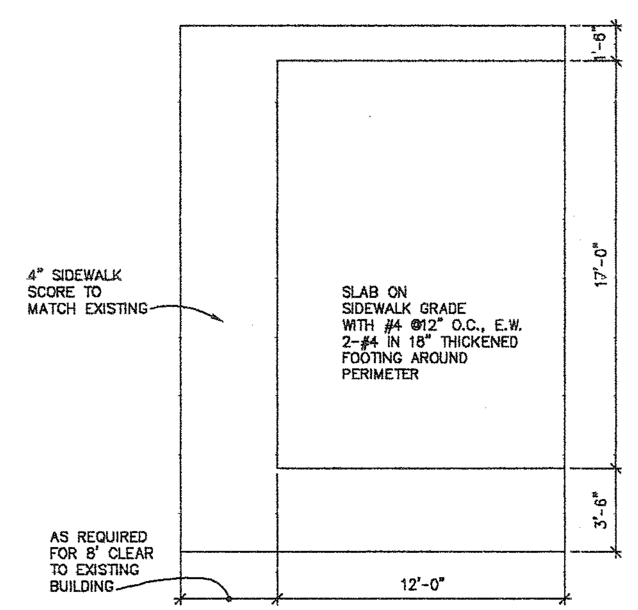
PLAN

SCALE: 1/8" = 1'-0"



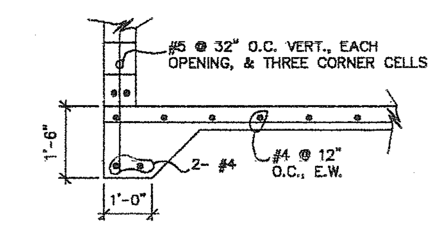
ROOF FRAMING PLAN

SCALE: 1/8" = 1'-0"



FOUNDATION PLAN

SCALE: 1/8" = 1'-0"



TYPICAL SLAB SECTION

SCALE: 1/4" = 1'-0"

CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
BLOWER BLDG./
CONTROL BLDG. - Figure 6-1
CLACKAMAS COUNTY, OREGON

CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

1301-6-1 02/28/2000 10:49

6.2 BIOSOLIDS HANDLING

The need for additional sludge storage capacity has been discussed in previous sections. Total dry solids production is estimated at 426 lbs/ day (155,490 lbs/year) The present storage capacity of the ponds is less than six months even allowing for average concentration of greater than 6%.

Under present loading conditions the ponds are full in the spring and the supernatant must be continually decanted back into the plant. The increased loading rates, combined with a high solids inventory in the secondary clarifiers can lead to elevated BOD and SS concentrations in the effluent.

Additional biosolids storage is necessary during the six month period from November through April. At solids loadings projected for 2020, the present ponds have a storage capacity of 3.4 months.

Several options were evaluated, including: modifying the existing lagoons for increased storage, construction of a concrete storage tank, wet-season land application of biosolids, and a biosolids dewatering process to reduce sludge volume.

6.2.1 Liquid Biosolids

6.2.1.1 Irrigation

If biosolids are applied during the winter months it would not be necessary to store a total of six months of production. The current Biosolids Management Plan allows for year around application of solids. During the wet season, wet ground conditions limit the size of trucks and the method of application.

It would be feasible to use the City truck with a tank of 3500 gallons and a distributor with a large nozzle. Irrigation equipment called a hose traveler has a large wheel with several hundred feet of coiled hose on a movable cart. The hose uncoils as it moves along the ground. This equipment has been used for municipal sludges and would be suitable for this application. A solids handling slurry pump would be a necessary component of the irrigation system.

Table 6-7: Irrigation of Biosolids	
Irrigation Traveler	$15,000
Biosolids Pump with power take-off	$5,000
Tractor (5510)	$30,000
Estimated Costs	$50,000

6.2.1.2 Increased Storage with Mixing

Sludge storage capacity needs to be increased to provide six months worth of storage or four months if winter application methodology is adopted. Storage needs to be adequate to handle projected sludge loadings which it is assumed will increase at a rate of 2.3% per year.

Table 6-8: Solids Loading and Storage						
	6 Months			4 Months		
	lbs	4% gals.	6% gals.	lbs	4% gals.	6% gals.
1999	78,000	233,813	155,875	52,000	155,875	103,917
5 Years	87,000	260,791	173,861	58,261	174,645	116,430
10 Yrs	97,915	293,511	195,674	65,277	195,674	130,449
15 Yrs.	109,706	328,854	219,236	73,137	219,236	146,157
20 Yrs.	122,916	368,452	245,635	81,944	245,635	163,757

Increased storage capacity could be created by modifying the existing ponds. This modification assumes that a uniform concentration in the ponds is maintained by mixing. At an average concentration of 4%, the projected need for 4 months storage is 245,000 gallons. A 3 ft. vertical wall around the large pond could accommodate the increase in required capacity.

The main problem with current operation of the sludge lagoons has been a lack of mixing. The biosolids tend to concentrate in the far end of the pond and do not flow into the sludge collection manhole to be pumped. Mixing methods which employ air will have a tendency to cause foaming of the lime stabilized biosolids. Therefore the most suitable method of keeping lime stabilized solids mixed is the use of pumps for recirculation.

Table 6-9: Storage Methods					
Storage		Mixing		Pump	Total
Description	Cost	Method	Cost	Cost	Cost
New Gravity Thickener	$80,000	Clarifier Drive	45,000	$15,000	$140,000
New Rectangular Thickener	$145,000	Gantry Dredge	$65,000		$210,000
Pond Enlargement	$80,000	Submersible	24000	$10,000	$114,000

6.2.2 Dewatering

Dewatering of biosolids reduces the total volume which must be handled, stored and hauled. Methods of dewatering include: belt filter presses, vaccum filter, centrifuge, drying bed, vertical press, and rotary press. The methodologies evaluated as most applicable to solids handling at the Estacada treatment plant included belt filter presses, vertical presses, and rotary presses.

Changing from a liquid to a higher concentration biosolid product involves changes in operation A front end loader would be needed to load a dump truck from the storage bunkers. A manure spreader and tractor would be required for field application. The suitability of the current sites for land application of the drier solids would need to be evaluated, new permit applications would need to be reviewed by DEQ, and authorizations obtained.

Filter Belt Press

A filter belt press could be expected to achieve 20% dry solids with polymer addition and a consistent feed of 2.5% to 4%. Feeding a belt press directly from the storage lagoons is not feasible because of the wide range of solids concentrations. Field testing has confirmed this problem.

It would be necessary to provide a feed tank which could store and thicken solids. This tank would need capacity to store one week to ten days of the plants solids production, and would be fed directly from the lime stabilization tank or the western storage lagoon, #1.

The cost of a traditional belt filter press may be comparable to a concrete storage tank but associated equipment costs and a building with an odor control system add significantly to the estimated costs. The square footage of the building for the belt filter press, and associated equipment is estimated at 900 SF. The thickened biosolids come off the belt at a height of approximately six ft. and the belt would be placed on a platform and raised an additional 6 ft. allowing for a conveyor to load a truck directly.

Wet weather application of dewatered biosolids would not be feasible , and six months of dry storage would be needed for the cake product. At projected solids loadings, and assuming a solids concentration of 20%; the necessary capacity is estimated at 9800 cubic ft.

Covered storage bunkers could be constructed in the footprint of Sludge Storage Lagoon #2. The lagoon would need to be partially filled to provide a more level access for loaders. These costs are included in Table 6-11. Costs reflect the cost of improvements and equipment at the treatment plant. They do not include the cost of equipment for transporting and land applying cake sludge

Table 6-10:Cost of a Belt Filter Press	
Building	$65,000
Belt Filter Press	$150,000
Odor Control System	$60,000
Feed Pump	$10,000
Polymer System	$15,000
Feed Tank	$30,000
Electrical (bldg. wiring and secondary power)	$40,000
Control system and SCADA Modifications	$20,000
Mechanical (piping)	$10,000
Concrete Work	$10,000
Covered Storage Bunkers (6 Months)	$80,000
Total Costs	$490,000

Vertical Press

Dewatering alternatives to belt filter presses include a vertical press. This equipment employs a vertical screw surrounded by a screen. Water is removed as the solid/liquid stream is carried up, and an adjustable plug in the end restricts cake passage and squeezes the mass. Pilot testing of the press on the lime stabilized and thickened solids was completed in December of 1999, and was only marginally successful.

Table 6-11: Estimated Costs of Vertical Press	
Building	$46,000
Dewatering Device	$60,000
Feed Tank	$30,000
Odor Scrubber	$40,000
Electrical (bldg. wiring and secondary power)	$25,000
Control system and SCADA Modifications	$15,000
Mechanical (piping)	$10,000
Concrete Bunkers to Provide 6 Months Storage	$80,000
Total Costs	$306,000

Rotary Press

The rotary press uses compressive forces similar to that of a belt press to separate liquid from the biolsolids. Biosolids are first pumped into a pressurized chamber where polymer is added to flocculate the solids. From the flocculation chamber the chemically treated solids enter the rotary press. The required inlet pressure is 3-11 PSI..

Once the solids are in the press, a paddle wheel within the unit generates additional compressive force on the solids as they are rotated through the press channel. The inlet pressure of the pump and the restrictive action of the outlet dewaters the solids, resulting in a dry cake.

Table 6-12: Cost of a Rotary Press	
Building	$45,000
Rotary Press (single channel)	$120,000
Feed Pump	$10,000
Polymer System	$15,000
Feed Tank	$30,000
Electrical (bldg. wiring and secondary power)	$25,000
Control system and SCADA Modifications	$15,000
Mechanical (piping)	$10,000
Covered Storage Bunkers (6 Months)	$65,000
Total Costs	$335,000

Municipalities with mixed aerobically digested sludges report cake dryness of 25%-30%. Operators at three municipalities who were contacted concerning the operation of the rotary press said that it is clean, quiet, and generates few odors.

Of the three dewatering alternatives the rotary press is less expensive than the belt filter press in terms of installation and operating costs and has a better performance record on municipal sludges than the vertical press.

Contract Dewatering

A third dewatering alternative is to contract with a private company with mobile dewatering equipment. Preliminary investigation indicates that the cost of this alternative is approximately $0.12 / gallon which would be $92,000 per year at projected sludge yield. The cake would still need to be stored and land applied by the City.

6.2.3 Evaluation of Alternatives for Biosolids Handling

In evaluating the alternatives, switching to a dewatering process for biosolids would require a larger capital expenditure than improving the liquid biosolids storage and handling. The reduction of biosolid volume by dewatering decreases the number of trips for land application; but to justify the increased capital expenditure and offset increased operating costs, the decrease in hauling costs must be significant. In Estacada the proximity of sites suitable for land application argue for retaining a liquid biosolid product.

It is recommended that Estacada continue with liquid biosolid storage and handling. In the short term the need for storage can be reduced by land application of some of the solids during the winter months. Land application is proven technology that could be implemented at Estacada.

The least expensive way of increasing the storage capacity for liquids biosolids would be to enlarge the large pond with a 3 foot vertical wall of. Mixing in the ponds will be accomplished with a fixed submersible pump in the corner of each pond to set up a pattern of recirculation. There would be several levels of discharge from the rectangular pump vault or precast manhole .The pumps are set out into the sludge ponds and accessed via a catwalk to a pump platform on the pump vault.

Table 6-13: Modified Sludge Ponds	
3 ft. Wall around large pond	$60,000
Submersible chopper pumps with recirculation (2)	$24,000
Portable self priming pump (electrical)	$10,000
Pump vault and sumps (2)	$5,000
Catwalk and platform (2)	$9,000
Electrical	$6,000
Total Construction	$114,000

A submersible chopper pump is recommended for sludge withdrawal because of its ability to handle a range of solids concentrations (up to 8%) The pump would be placed in the existing solids manhole.

EPA RELIABILITY

	CLASS I

DESIGN POPULATION 2020

	3545

DESIGN CAPACITY

AVERAGE DRY WEATHER FLOW (ADWF) Max DWF = 1.1 MGD	0.54 MGD
AVERAGE WET WEATHER FLOW (AWWF)	1.50 MGD
PEAK WET WEATHER FLOW (PWWF)	4.5 MGD

DESIGN LOADINGS

BIOCHEMICAL OXYGEN DEMAND (BOD)	1080 LB/DAY
TOTAL SUSPENDED SOLIDS	945 LBS/DAY

REQUIRED EFFLUENT QUALITY

DRY WEATHER BOD/SS	10/10 MG/L
WET WEATHER BOD/SS	20/20 MG/L
DRY WEATHER MASS LOAD LIMITS MO. AVE.	45 LBS
WET WEATHER MASS LOAD LIMITS MO. AVE.	90 LBS

INFLUENT CHARACTERISTICS

BIOCHEMICAL OXYGEN DEMAND (BOD5)	
AVE WET WEATHER CONCENTRATION	110 MG/L
AVE DRY WEATHER CONCENTRATION	220 MG/L
LOADING	700 LBS/DAY
TOTAL SUSPENDED SOLIDS	
AVE WET WEATHER CONCENTRATION	95 MG/L
AVE DRY WEATHER CONCENTRATION	200 MG/L
LOADING	625 LBS/DAY

FLOWS 2020

MAXIMUM MONTH DRY WEATHER FLOW MMDWF-5	1 MGD
MAXIMUM MONTH WET WEATHER FLOW MMWWF-5	1.6 MGD
PEAK DAY FLOW PDAF-5	4.1 MGD
PEAK INSTANTANEOUS FLOW PIF-5	4.9 MGD

LOADING 2020

BIOCHEMICAL OXYGEN DEMAND (BOD5)	1103 LBS/ DAY
TOTAL SUSPENDED SOLIDS (TSS)	985 LBS/ DAY

INFLUENT SCREEN

TYPE	ROTATING RAKE
SCREEN	1/4"
SCREW COMPACTER/ DEWATERING	1 HP
DRIVE MOTOR	1 HP

GRIT REMOVAL SYSTEM

VORTEX GRIT REMOVAL CHAMBER	
DESIGN CAPACITY	2.5 MGD
AIR LIFT PIMP TO CONVEYOR	

PRIMARY CLARIFIERS

TOTAL AREA	1150 SF
2-12'X8'RECTANGULAR	
OVERFLOW RATE	
@ MMDWF-5	869 G/SF/D
@ MMWWF-5	1391 G/SF/D
@ PDAF-5	3565 G/SF/D
@ PIF-5	4260 G/SF/D

TRICKLING FILTER PUMPS

WETWELL CAPACITY	2300 GAL/FT, 11FT
PUMPS (4), FIRM CAPACITY PIF-5	3400 GPM (4.9 MGD)
PUMP 1	300 GPM @ 30'
PUMP 2	1600 GPM @ 38'
PUMP 3 ,4	1600 GPM @ 38'
MOTORS FOR 3, 4	15 HP
PUMP TO HIGH EFFICIENCY MOTOR SUITABLE FOR VFD OPERATION	15 HP
VFD (1)	PUMP 2

TRICKLING FILTER

CAPACITY OF DISTRIBUTER ARM	4.9 MGD
VOLUME OF MEDIA	43000 CF
ORGANIC LOADING	20 LBS BOD/1000 CF

SOLIDS CONTACT CHAMBER

VOLUME	25600 GALONS
DETENTION TIME	
@ MMDWF-5 YEAR 2020	36 MINUTES
@ MMWWF-5 YEAR 2020	23 MINUTES
FINE AIR DIFFUSERS	120 FLEXIBLE MEMBRANE DISC

SECONDARY CLARIFIERS

TOTAL AREA	4250 SF
2-52' DIAMETER CIRCULAR	
TOTAL FLOCCULATION WELL AREA	500 SF
2-18' DIAMETER CIRCULAR	
OVERFLOW RATE	
@ MMDWF-5 YEAR 2020	235 G/SF/D
@ MMWWF-5 YEAR 2020	376 G/SF/D
@ PDAF-5 YEAR 2020	965 G/SF/D
@ PIF-5 YEAR 2020	1152 G/SF/D

RETURN SLUDGE PUMPS

SUBMERSIBLE NON-CLOG	50 - 400 GPM @ 25'
HIGH EFFICIENCY MOPTOR SUITABLE FOR VFD OPERATION	2.7 HP
FLOW MEASUREMENT OVER V-NOTCH WEIR	

WASTING PUMP

SUBMERSIBLE TORQUE FLOW	150GPM @ 20'
BATCH WASTE DFROM SUMP WITH VOLUME OF 18000 GALLONS	

BLOWERS

POSITIVE DISPLACEMENT BLOWERS	
BLOWER 1,CHANNEL AIR	150 SCFM @ 4.5 PSI
BLOWER 2, AIR LIFT PUMPS AND GRIT REMOVAL PUMP	150 SCFM @ 5.5 PSI
DIGESTER REDUNDANCY	120 SCFM @ 9.0 PSI
BLOWER 3, DIGESTER	120 SCFM @ 9.0 PSI
BLOWER 4, GRIT REMOVAL PUMP AND DIGESTER REDUNDANCY	150 SCFM @ 5.5 PSI

CHLORINE CONTACT CHAMBER

VOLUME	64,000 GAL
CONTACT TIME	
@ MMDWF-5 YEAR 2020	92 MINUTES
@ MMWWF-5 YEAR 2020	58 MINUTES
@ PDAF-5 YEAR 2020	22 MINUTES
@ PIF-5 YEAR 2020	19 MINUTES
LENGTH/WIDTH RATIO	225/4 = 56
FLASH MIXER	3 AXIAL FLOW S.S. BLADES
MOTOR	1725 RPM

DISINFECTION

HYPOCHLORITE STORAGE TANKS (2)	FIBERGLASS REINFORCED PLASTIC
CAPACITY	235 GALLONS
METERING PUMPS (2)	MOTOR DRIVEN
DIAPHRAGM	VITRON -R
CAPACITY	2.5 GPH
ONE PUMP TO BE ACTIVATED BY EMERGENCY BYPASS	
DILUTION WATER ROTAMETERS	
WATER	POTABLE

DECHLORINATION

CHEMICAL FEED PUMPS (2)	0.75 GPH
DECHLORINATION ANALYZER	6-20 MA SIGNAL

SAND FILTRATION

TOTAL AREA	
DIMENSIONS 16' X 40', MEDIA 11" DEEP.	
AUTOMATIC BACKWASHING SANFD FILTER WITH TRAVELING BRIDGE	
DESIGN HYDRAULIC LOADING RATE AVE. - PEAK	2 GPM/SF - 5 GPM/SF
HYDRAULIC LOADING RATE	
@ MMDWF-5 YEAR 2020	1.1 GPM/SF
@ MMWWF-5 YEAR 2020	1.7 GPM/SF
@ PDAF-5 YEAR 2020	4.5 GPM/SF
@ PIF-5 YEAR 2020	5.3 GPM/SF

AEROBIC DIGESTION

VOLUME	70,000 GALLONS
PRIMARY SOLIDS LOADING	
YEAR 2000	7 LBS/ 1,000 CF
YEAR 2020	11.3 LBS/ 1,000 CF
DETENTION TIME	
YEAR 2000	52 DAYS
YEAR 2020	33 DAYS

LIME STABILIZATION

VOLUME	22,000 GALLONS
DETENTION TIME	24HRS AT PH 11.5
LOADING IN YEAR 2000	
DIGESTED PRIMARY SLUDGE, 190 LBS @ 1.7 %	1340 GPD
WASTE ACTIVATED SLUDGE, 175 LBS @ 0.7%	3000 GPD
LIME SLURRY 64 LBS @ 6%	120 GPD
LOADING IN YEAR 2220	
DIGESTED PRIMARY SLUDGE, 295 LBS @ 1.7 %	2080 GPD
WASTE ACTIVATED SLUDGE, 275 LBS @ 0.7%	3276 GPD
LIME SLURRY 100 LBS @ 6%	200 GPD

PRIMARY SLUDGE PUMPS

PLUNGER TYPE SLUDGE PUMPS	2
CAPACITY	86 GPM @ 25'
FLOW MEASURED AS (TOTAL STROKES/STROKES PER MINUTE)xGPM	

SLUDGE STORAGE PONDS

VOLUME POND 1	195000 GALLONS
VOLUME POND 2	50000 GALLONS
STORAGE	140000 GALLONS
YEAR 2000 4%	190 DAYS
YEAR 2020 4%	120 DAYS

LAND APPLICATION EQUIPMENT

TYPE	TRAVELING IRRIGATION WHEEL
ENGINE FOR TRAVELING CART	5 HP
SLURRY PUMP	OPEN IMPELLER
UTILIZES POWER TAKE OFF	1000 RPM

FLOW METERING

INFLUENT	3.5 MGD
EFFLUENT V-NOTCH WEIR	0-9 MGD
ULTRASONIC LEVEL TRANSMITTER (2)	0-24 INCHES
CONTINUOUS RECORDER, INDICATOR, TOTALIZER AND SAMPLER	4.20 MA

SAMPLERS

DIVERTER TYPE SAMPLERS (2)	24 SAMPLES/DAY
SELF PRIMING PUMPS (20)	5.8 GPM
LOCATION	INFLUENT CHANNEL
	EFFLUENT MANHOLE

STAND-BY GENERATOR

CAPACITY	100 KW
TOTAL RUNNING LOAD	57 KW
AUTOMATIC TRANSFER SWITCH	
EMERGENCY OPERATIONS	TRICKLING FILTER PUMPS 1, 2, & 3
	SLUDGE PUMP #1
	BLOWERS 1 & 2
	DIGESTER MIXER
	CRITICAL LIGHTING
	FLASH MIXER

OUTFALL

LENGTH	650 FT/ 80 FT.
MATERIAL	CONCRETE/STEEL
SUBMERGENCE	6 FT
MANHOLES	3

EXISTING PLANT AND PROPOSED IMPROVEMENTS CAPACITY SUMMARY

CITY OF ESTACADA

WASTE WATER FACILITIES PLAN

Figure 6-2

CLACKAMAS COUNTY, OREGON

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1301-6-2 06/08/2000 14:05

CHAPTER 7: COLLECTION SYSTEM

7.1 EXISTING SYSTEM

The existing collection system is divided into four subsystems. Subsystem #1 is the oldest subsystem, and the main was extensively repaired when infiltration caused the collapse of the pipe channel along third street. In 1996, the 12 inch line between SW Wade Street and Main Street was replaced.

Table 7-1: Existing Subsystems		
Subsystem	Year Built	Area Served
1	1911	Downtown portion of Estacada
2	1935	Major residential area
3	1963	Lakeshore Drive area, and southeast hills
4	1961, 1977,	Northwest section, School Trunk to Cemetery Rd.

All four systems have been extended since their original construction. The School Trunk in Subsystem 4 was constructed in 1977 to Cemetery Rd. Recent developments along Cemetery Rd include Foothills I, and III, and Valley View Terrace. Currently a new 300 home development is proposed land annexed to the north of the Foothills development.

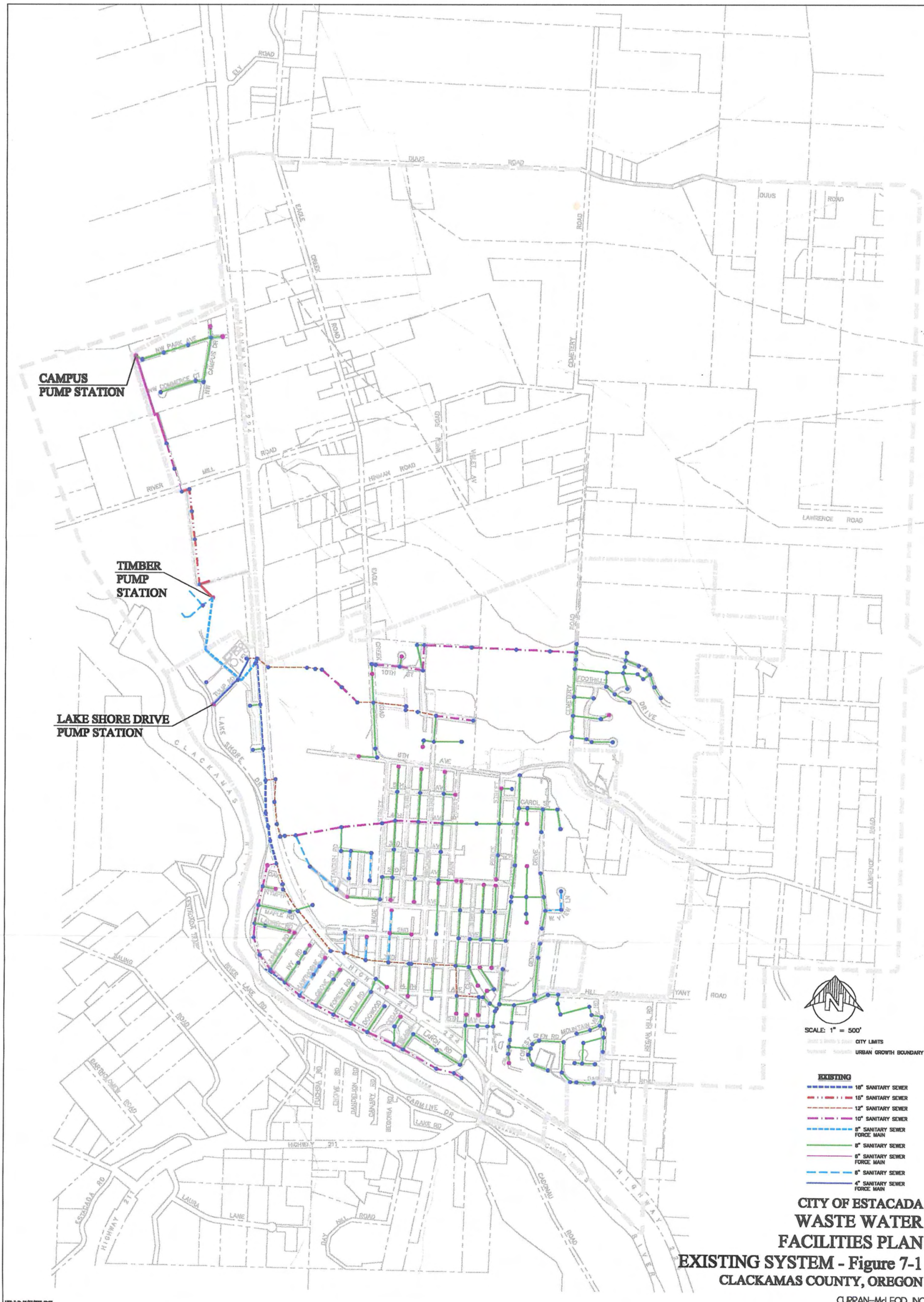
In 1995 the main line for Subsystem 1 was connected to the line from Regan Hills area at 4th and Shafford, shown in Figure 8-1 as manhole 10A. This connection took pressure off of the Subsystem 3 mainline and the Lakeshore Drive pump station. ?? NO-

Recent development in the Regan Hill area includes Cazadero Heights located east of Espinosa and south of Regan Hill Rd. The Sewer Capital Improvement Plan includes plans to extend interceptors along Regan Hill Rd. and east of Espinosa to the UGB. This area will drain into Subsystem 1. NO- To Lakeshore

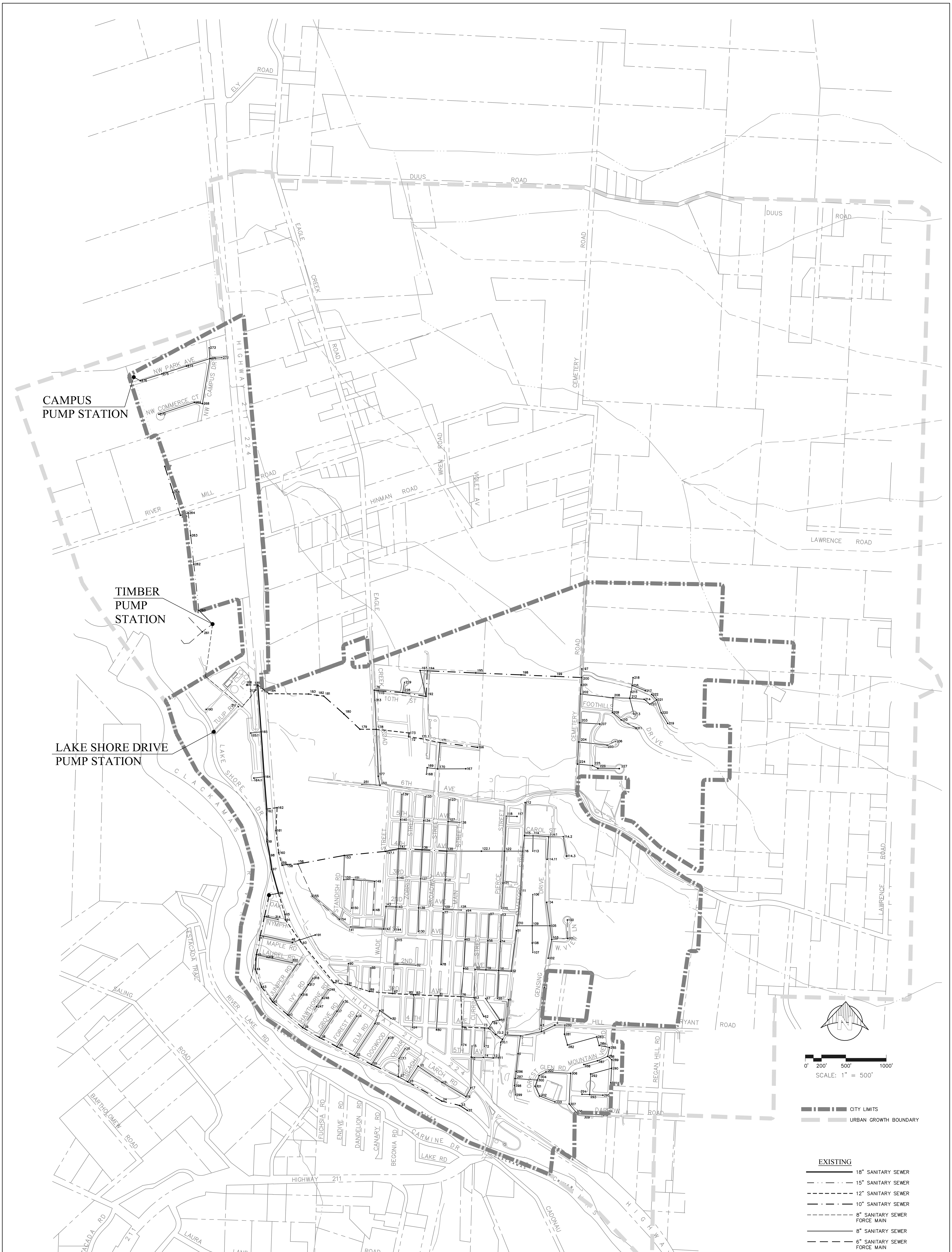
7.2 SEWER CAPITAL IMPROVEMENT PROGRAM

Recommendations for extensions to the existing trunk lines, and new interceptors to serve the Urban Growth Boundary (UGB) are based on the Comprehensive Plan developed by the City of Estacada. and follow the land use designations previously discussed.

As proposed development is approved, annexation must occur before City public utilities such as sewer and water are built are built. It is necessary to include, the Urban Growth Management Boundary (UGMA), the area between the City limits and the UGB, in all facilities planning. This is especially true of sewer interceptors which must be adequately sized to handle all upstream flow.



CAMPUS
PUMP STATION
TIMBER
PUMP
STATION
LAKE SHORE DRIVE
PUMP STATION
NW PARK AVE
NW CAMPUS DR
NW COMMERCE CT
ELY ROAD
DUUS ROAD
EAGLE CREEK ROAD
CEMETERY ROAD
RIVER MILL ROAD
HINMAN ROAD
LAWRENCE ROAD
10TH ST
6TH AVE
CAROL ST
FOOTHILL DRIVE
LAKE SHORE DR
CLACKAMAS
HILL ROAD
YANT ROAD
REGAN HILL RD
CARMINE DR
LAKE RD
HIGHWAY 211
LAURA LANE
BARTHOLOMEW ROAD
SCALE: 1" = 500'
CITY LIMITS
URBAN GROWTH BOUNDARY
EXISTING
18" SANITARY SEWER
15" SANITARY SEWER
12" SANITARY SEWER
10" SANITARY SEWER
8" SANITARY SEWER FORCE MAIN
8" SANITARY SEWER
6" SANITARY SEWER FORCE MAIN
6" SANITARY SEWER
4" SANITARY SEWER FORCE MAIN
CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
EXISTING SYSTEM - Figure 7-1
CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS



CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
EXISTING SYSTEM - Figure 7-1
CLACKAMAS COUNTY, OREGON

CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

7.2.1 Design Flows

To determine design flows for the existing main lines and proposed new interceptors, factors for the average number of Equivalent Dwelling Units per acre (EDUs / acre) were applied to each zoning designation and multiplied by the base sewage flow per residential unit, 290 gpd / EDU. The zoning factors were developed in 1994, City of Estacada Systems Development Charge findings

The average base flow estimated from the average of dry weather flows excluding May and October is 0.36 MGD. The flow contributed per dwelling units was calculated by dividing the total average base flow by the total number of EDUs. Based on current billings registers the total number of EDUs served by the system is 1250. The total base flow divided by 1250 EDUs gives an estimate of the flow contributed per dwelling unit as 290 gallons.

To estimate the diurnal contribution to flow a peaking factor of 3 was applied to the base flows. The peaking factor was not applied to industrial contributions to flow. The factor for industrial contribution of 6 EDUs / acre is equivalent to 1752 gpd / acre which is conservative for most industrial applications.

In this Facilities Plan, interceptors were extended to the UGB and sized based on 100% build-out. Factors for the average number of equivalent dwelling units per acre were applied to each zoning designation, and it was assumed that only 70 % of the total area will be available for residential, commercial, and industrial development. The area available for each land use was estimated as 70% of the total, based upon an estimated 30% of land for public use including streets, schools and parks.[6]

The land use designations for each of the subareas to be drained by new interceptors are shown in Table 7-2 and the sub-basins drained by these interceptors are shown in Figure 7-2, Sewer CIP.

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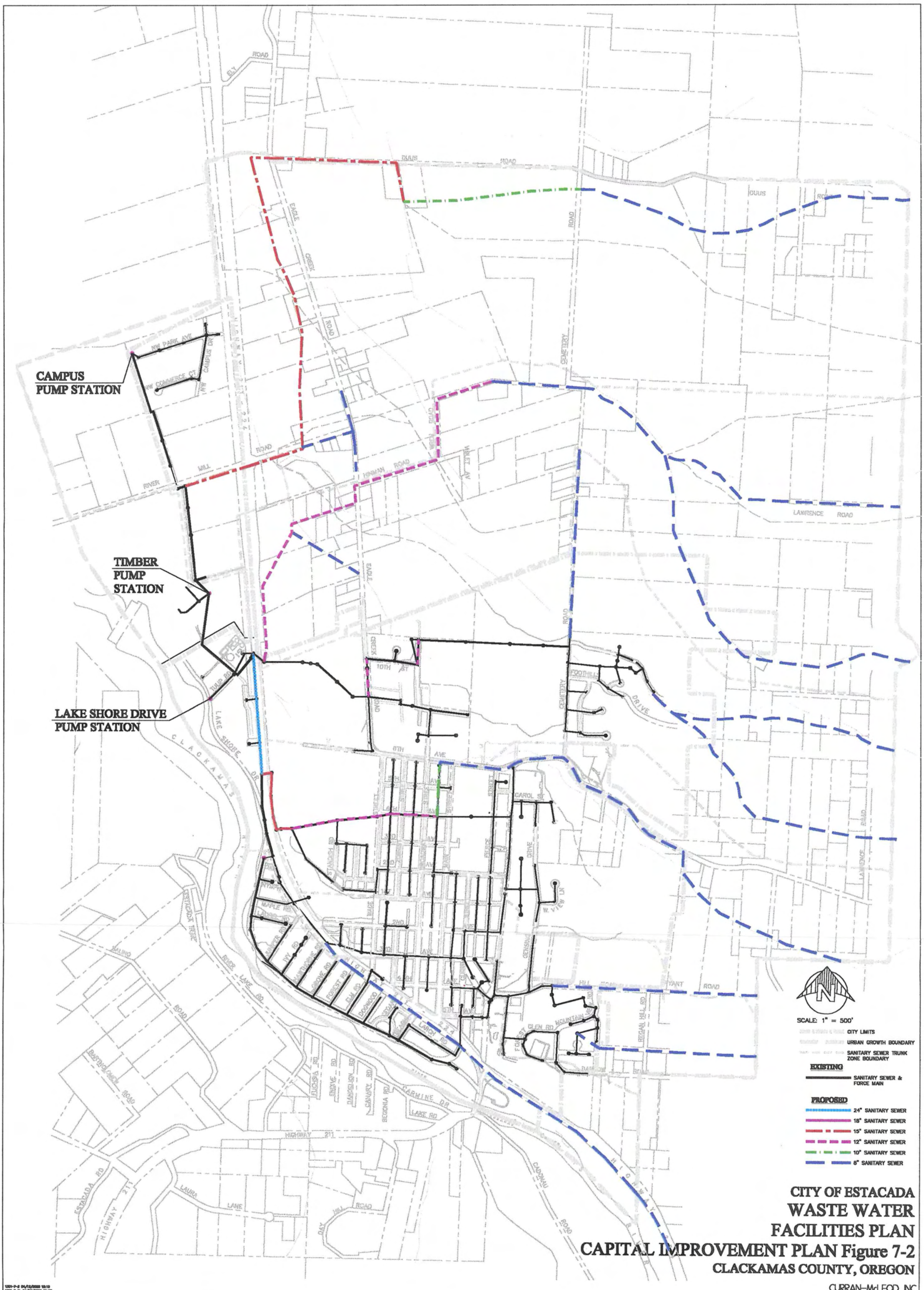
[6] *City of Estacada Comprehensive Plan*, Table 18 (c), Page 89.



CAMPUS PUMP STATION
TIMBER PUMP STATION
LAKE SHORE DRIVE PUMP STATION
DUUS ROAD
LAWRENCE ROAD
EAGLE CREEK ROAD
CEMETERY ROAD
RIVER MILL ROAD
HINMAN ROAD
LAKE SHORE DR
CLACKAMAS
6TH AVE
10TH ST
FOOTHILL DRIVE
CAROL ST
MAPLE RD
2ND
HIGHWAY 211
HIGHWAY 224
CARMINE DR
LAKE RD
RIVER LAKE RD
N
SCALE: 1" = 500'
CITY LIMITS
URBAN GROWTH BOUNDARY
SANITARY SEWER TRUNK ZONE BOUNDARY
EXISTING
SANITARY SEWER & FORCE MAIN
PROPOSED
24" SANITARY SEWER
18" SANITARY SEWER
15" SANITARY SEWER
12" SANITARY SEWER
10" SANITARY SEWER
8" SANITARY SEWER
CITY OF ESTACADA
WASTE WATER
FACILITIES PLAN
CAPITAL IMPROVEMENT PLAN Figure 7-2
CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

CITY OF ESTACADA

WASTE WATER

FACILITIES PLAN

CAPITAL IMPROVEMENT PLAN Figure 7-2

CLACKAMAS COUNTY, OREGON

CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

Table 7-2: Sewage Contribution Within City Limits									
EDU / acre	4	6	9	2	6			Projected Base Flow	Peak Flow
Zoning	R1	R2	MR	C	I	Total			
Subsystem	Acres	Acres	Acres	Acres	Acres	Acres	EDU	MGD	MGD
1	49	21		47		117	291	20% 0.09	0.26
2	98		1	11	38	147	452	29 0.13	0.40
3	54	5	14	3		75	262	18 0.08	0.23
4	64		6	50	14	134	343	22 0.10	0.30
S. Tulip Rd					45	45	189	13 0.06	0.17
Total	264	26	20	111	97	518	1537	0.45	1.35

Table 7-3: Sewage Contribution from UGMA									
EDU / acre /Acre/	4	6	9	2	6			Base Flow	Peak Flow
Zoning	R1	R2	MR	C	I	Total			
Subsystem	Acres	Acres	Acres	Acres	Acres	Acres	EDU	MGD	MGD
Regan Hill	55					55	154	0.05	0.13
Hwy 224 East	60					60	168	0.05	0.15
Coupland	138	60				198	638	0.19	0.56
Duus Rd	301				197	498	1670	0.49	1.46
Timber Park	0				132	132	554	0.16	0.49
River Mill Rd.	47				18	65	207	0.06	0.18
Hinman Rd.	344			45	5	394	1047	0.31	0.92
Cemetery Rd.	63					63	176	0.05	0.15
Foothills W. Ex.	127					127	356	0.10	0.31
Totals (w/o Airport)	1135				352	1592	4971	1.45	4.35

7.2.2 Existing Interceptors

The design flow for the existing main lines includes the peak base flows, and an infiltration / inflow component. The peak base flow is based on buildout within the city limits to the densities allowed by existing zoning.

The existing trunk lines are each assumed to contribute a proportion of the total estimated infiltration. The average percentage of the total infiltration contributed by each basin is based on flow measurements done in 1998 and 1999. The design flows in Table 7-4 do not include flows from projected growth outside the City limits.

Table 7-4: Wastewater Contributions from Existing Basins

Main Line	Base Flow MGD (Peak)	Infiltration		Total Flow
		%	MGD	MGD
1	.26	39	1.37	1.63
2	.40	26	0.91	1.31
3	.23	15	0.53	0.76
4	.30	20	0.70	1.00
Totals	1.19	100	3.50	4.69

530 gpm

The total infiltration is assumed to be 3.5 MGD based on the Peak Instantaneous Flow (PIF) minus the peak base flow. The PIF was 4.6 MGD, from the analysis in Chapter 2. The peak base flow, is estimated as the base flow of 0.36 MGD times a peaking factor of 3, or 1.1 MGD .

In general the existing mainlines are adequately sized to handle projected flows within the city limits not including flow from new interceptors which will be built to service the UGB. The capacity of the existing main lines is presented in Table 8-4.

Main line #4 has one section of 8 inch pipe from manhole 178-176 which may be a potential prblem potential problem area. Based on a slope of .004 the line has a capacity of 0.53 MGD which is insufficient the current sewage contribution at peak flow of 0.80 MGD

The trunk line for flows from mains 1, 2 and 3 (MH 96 - MH185) begin to surcharge at flows greater than 2.5 MGD. Surcharging of the interceptor at peak flows results in an estimated increase in the hydraulic grade line of 2.5 ft at MH 163. Since there is greater than 6 ft of freeboard between the crown of the pipe and the ground there is not currently a problem with overflows. Surcharging should be monitored as growth occurs.

Table 7-5: Capacity of Trunk Lines to Handle Flows Without New Interceptors						
				Existing Capacity	Existing Flows	Remaining Capacity
Subsystem	MH-MH	In.	Slope	MGD	MGD	MGD
1	96-76	12	0.084	2.27	1.63	0.64
2	163-159	12	0.003	1.31	1.31	0.00
2	156-147A	10	0.006	1.53	1.31	0.22
3	49-22	10	0.007	1.27	0.76	0.51
4	185 -182	12	0.015	3.03	1.00	2.03
4	182-179	10	0.013	1.73	1.00	0.73
4	178-176	8	0.004	0.53	0.80	-0.27
4	192-194	10	0.004	0.96	0.80	0.16
4	194-200	10	0.015	1.86	0.80	1.06
1, 3	96-163	18	0.001	2.53	2.53	0.00
1, 2, 3	163-185	18	0.001	2.53	3.84	-1.31
1, 2, 3, 4	185-STP	18	0.005	4.95	4.84	0.11

7.2.3 New Interceptors

The proposed sewers to service the UGB were sized, sloped, and located to maintain scouring velocities (2.5 ft/sec) at design flow. The drainage basins delineated in the City of Estacada Storm Drainage Plan aided in locating the proposed interceptors where the topography could be used to best advantage in the design of the gravity system. The minimum slopes shown in Table 8-2 were employed except where it was appropriate tp assume steeper slopes based on the grade.

Table 7-6: Minimum Gravity Sewer Line Grades	
Pipe Size	Minimum Grade
8"	0.40%
10"	0.28%
12"	0.22%
15"	0.15%
18"	0.12%

The new interceptors were designed using Mannings equation for gravity flow and a Mannings n for PVC of 0.012. For public sewers the minimum line sign is 8 inches. Where sewer grades are greater than the minimum required , they have been based on the existing ground slope, or as-built drawings where available.

The unit costs in the following table include pipe cost, excavation, bedding, and backfill; manholes, road repair, access roads where necessary, a construction contingency, and estimated engineering costs. All new sewers are assumed to be PVC.

Table 7-7: New Interceptors

Interceptor	Design	Slope	Size	Full	P/F	Length	Unit	Cost
	MGD		in.	MGD	ratio	LF	$/LF	
Duus Rd	1.46	0.0015	15	1.74	0.97	6000	$94	$564,000
Duus East of Currin Creek	1.00	0.0100	10	1.52	0.66	2200	$86	$189,200
Duus East of Cemetery Rd	1.20	0.0500	8	1.87	0.64	4350	$70	$304,500
River Mill	0.14	0.0022	8	0.53	0.27	1700	$70	$119,000
Duus + R.M	1.60	0.0030	15	2.46	0.78	1550	$84	$130,200
Highway Bore	1.68	0.0030	15	2.46	0.78	80	$172	$13,760
Hinman Hyw - Wren Rd.	0.92	0.0022	12	1.16	0.79	5600	$80	$448,000
Hinamn S. to Eagle Cr. Rd.	0.39	0.0100	8	0.84	0.68	1000	$70	$70,000
Hinman East ,Currin Cr.	0.63	0.0100	8	0.84	0.75	1200	$70	$84,000
Hinman East Cemetery Rd	0.63	0.0500	8	1.88	0.34	10183	$70	$712,810
Cemetery Rd.	0.15	0.0040	8	0.53	0.28	2400	$70	$168,000
East Extension	0.32	0.0040	8	0.53	0.60	6752	$70	$472,640
Coupland Rd.	0.48	0.0100	8	1.16	0.41	5273	$66	$347,987
Coupland to 4th	0.70	0.0040	10	0.96	0.81	735	$70	$51,450
Regan Hills	0.14	0.0040	8	0.53	0.26	4908	$70	$343,560
Hwy 224, East	0.15	0.0040	8	0.53	0.28	5891	$70	$412,370
Estimated Cost at Ultimate Buildout, using 1999 dollars								$4,431,477

The Timber Park interceptor and pump station and the Industrial Campus pump station were recently completed to service the Estacada Industrial Campus which was annexed in 1998. The proposed Duus Rd River interceptor would flow into the proposed new interceptor along River Mill Road which runs under Highway 224 and west into the Timber Park Interceptor. The Duus Road interceptor which collects flow from the industrially zoned areas to the East of Highway 224, and along Eagle Creek Road should be15 inch pipe.

The combined flow from the Timber Park, Duus and River Mill interceptors would flow into the existing 15 inch interceptor leading south to the Timber Park pump station. At full build out the pumps at this pump station will need to be upgraded to a capacity of 2.1 MGD (1500 gpm).

Development to the north of the existing city limits would be split between the proposed Duus and Hinman Rd interceptors. The Hinman Road interceptor would serve the area north of the City limits to River Mill Rd. and east of Highway 224 to the UGB. Between Eagle Creek Rd. and Highway 224 this area is commercially zoned. The proposed interceptor would flow south along 224 to the existing highway crossing to the treatment plant. Since this section is at minimum grade, a 12 inch interceptor is recommended. In the eastern section of this subbasin, the proposed interceptor would follow Currin Creek.

The proposed interceptor along Cemetery Rd. would serve new residential development north of the Foothills area. The subbasin for this interceptor extends north to a ridge which separates it from the Hinman Subbasin to the north. To the east of the Foothills area, new development would be served by an 8 inch interceptor called Foothills East which would tie into the existing line on Hill Way. This proposed line would separate into northern and southern branches extending to the eastern UGB.

The Regan Hills sanitary sewer system would serve a total area of 123 acres of low and medium density residentially zoned property, east of Espinosa Road and Gensing Drive to the UGB and North of Darrow Road to the City Limits. The Estacada Heights development along Forest Glen and Mt. View Roads is the most recent development in this area. Proposed interceptors would tie into existing 8 inch lines along Regan HillRd. and north of Darrow Rd.

The proposed interceptor, Highway 224 East, would serve the area east of Short Street and north of Highway 224 to Darrow Rd. Originally this line was planned to connect with trunk line #1 where it heads north along the east side of 224. A pump station would accomplish the same thing, but gravity sewer is recommended.

7.2.4 Necessary Improvements to Existing Trunk Lines

The projected design flow for the existing main lines includes the peak base flows, the estimated flow from the proposed interceptors and an infiltration component. The existing trunk lines are each assumed to have a proportion of the total estimated infiltration The average percentage of the total infiltration contributed by each basin is based on flow measurements done in December 1999.

Table 7-8: Projected Design Flow for Existing Main Lines						
Main Line	Base Flow MGD (Peak)	Flow Projections		Infiltration		Total Flow
		Interceptor	MGD	%	MGD	MGD
1	.26	Reagan Hill	0.13	39	1.37	1.76
2	.40	Coupland	0.56	26	0.91	1.87
3	.23	Hwy 224 E.	0.15	15	0.53	0.91
4	.30	Cemetery Rd.	0.15	20	0.70	1.46
		Foothills W.	0.31			

Table 7-9: Capacity of Existing Main Lines to Handle Flow at Future Peak										
				Capacity	Design	Remaining	Upgrade for Flows at Buildout			
Main	MH-MH	Slope	In.	MGD	MGD	MGD	In.	LF	$/LF	Cost
1	96 - 76	0.084	12	2.27	1.76	0.51	-			
1	76 - 75	0.004	12	1.56	1.31	0.25	-			
2	163 - 159	0.003	12	1.31	1.87	-0.56	15	900	$82	$73,800
2	159 - 160	0.010	12	2.48	1.87	0.61	-			
2	156-147A	0.006	10	1.53	1.87	-0.34	12	1200	$78	$93,600
2	147 - 135	0.010	10	1.52	1.79	-0.27	12	800	$78	$62,400
2	135 - 122	0.015	8	1.03	0.93	0.10	-			
2	122 - 116	0.035	8	1.57	0.93	0.64	-			
3	49 - 22	0.007	10	1.27	0.91	0.36	-			
4	185 - 182	0.015	12	3.03	1.46	1.57	-			
4	182 - 179	0.013	10	1.73	1.38	0.35	-			
4	179 - 178	0.013	12	2.82	1.38	1.44				
4	178 - 176	0.004	8	0.53	1.26	-0.73	12	480	$78	$37,440
4	176 - 192	0.004	10	1.52	1.26	0.26				
4	192 - 194	0.013	10	0.96	1.26	-0.30	12	160	$78	$12,480
4	194 -200	0.015	10	1.87	1.26	0.61	-			
1, 3	96-163	0.001	18	2.53	2.67	-0.14	-			
1, 2, 3	163-185	0.001	18	2.53	4.54	-2.01	24	760	$92	$69,920
1, 2, 3, 4	185-STP	0.005	18	4.95	6.00	-1.05	21	100	$90	$9,000
Totals						0.00				$191,240

for sections of line which may be under capacity are designed assuming full pipe flow and minimum slopes except where it was appropriate to assume steeper grades based on the ground slope.

The design flows for the existing main lines are based upon conditions after interceptors are extended to the UGB and buildout has been achieved based on existing zoning. Upgrades have been included in the CIP if Design Flow/ Existing Capacity exceeds 1. In fact some surcharging of lines during peak flows is acceptable if there is enough capacity to prevent overflows. Surcharging of manholes on lines which do not have sufficient capacity for peak flows should be monitored.

Coupeland Road Interceptor will contribute additional flow to basin #2 and at ultimate buildout the projected flows may cause surcharging of the 10 inch lines. The projected peak flow is only slightly less than existing capacity but if the projected flows are realized surcharging could cause overflow at the upstream manhole.

The trunk line for mains #1,#2, and #3 should be monitored. It is estimated that projected peak flows through the existing 18" line will raise the hydraulic grade line as much as 6 ft at MH 96, and the interceptor from MH 163 to the plant will need to be replaced.

After the downstream interceptor is replaced the 18" interceptor for #1, and #3 (MH 96 -MH 163) will be only slightly under capacity at peak flows with a surcharge of less than a foot at the upstream manhole.

7.3 PUMP STATIONS

7.3.1 Lakeshore Pump Station

The Lakeshore Drive Pump Station serves subbasin #3 with 75 acres of single and multi-family residential and several acres of commercially zoned area. The base flow is estimated as 0.08 MGD (55 gpm) with a PIF of 0.23 MGD (159 gpm) . The ~~two~~ pumps each have a rated capacity of 150 gpm at 30 ft. of head with a maximum capacity of 300-350 gpm at higher wet pit levels.

As cited previously the only occasion when the Lakeshore Drive Pump station has overflowed was on February , 1996 when two one -in - 25 year storm events over two days resulted in raw sewage and rainwater surging from street manholes, inundating the cul de sac, and overflowing the curbs into the Clackamas River.

A series of drawn downs was done on both pumps to establish current pumping capacity. The rate of fill was measured numerous times over the course of the tests to determine the average flow into the pump station. This rate was added to the draw-down rate to determine individual pump capacity.

The complete results of the draw down test are included in Appendix #4. Both pumps are pumping very close to design capacity, the measured pumping rates were on or just below the design pump curve.

Pump # 2 pumped 150 gpm against 30 feet TDH, but the pumping rate dropped off rapidly as the level in the wet well decreased. At the shut off point it was barely keeping up with the influent flow of 66 gpm. The stop float should be raised because at the lower level pump #2 is pumping close to its shut off head.

Table 7-10: Lakeshore Drive Sewage Pump Station Design Data	
Area Served	75 acres of single and multi family residential including 3 acres which are commercially zoned, 262 EDU
Pump Station	
Type:	Dry Pit , vertically mounted, torque - flow pump
Pump type:	Constant Speed, 1170 rpm , recessed impeller, vortex flow
Design Capacity	150 gpm each at 30 ft. total dynamic head, verified
Pump Hp (each)	5 HP
Level Control Type	Bubbler tube
Overflow	415 ft.
Overflow Discharge	Upstream manhole
Average time to Overflow	100 gpm, 30 minutes
Auxillary Power	none
Pump bypass	Emergency bypass vault and 6 inch line to upstream manhole for use with gasoline powered 500 gpm emergency pump.
Pump fuel tank capacity	1 hour
Alarm Telemetry type	Autodialer to STP
Force Main	
Length, Type	100 ft. 6 -inch ductile iron
Profile	Continuously ascending at approximately 7% slope
Discharge manhole	#26 on 18 inch interceptor on Hwy 224
Air Release	none
Vacuum Release	none
Average detention	2.7 min (100 gpm:)

During a high flow event corresponding to the five-year storm, the PIF at the Lakeshore Drive pump station is estimated to be as high as 0.75 MGD (520 gpm). This estimate is based on 15% of the peak day I/I , 3.5 MGD, plus a peak base flow of 0.23 MGD (159 gpm).

Both dry well pumps are required to pump a flow of 520 gpm. Each pump can deliver 260 gpm at 28.5 ft total dynamic head. Taking into account frictional losses, the wet well would increase to a level of 409.5 feet, 6.5 feet below the overflow elevation.

Firm capacity for the five year storm and reserve capacity during high flow events greater than the five year storm is supplied by a portable 500 gpm pump which pumps from the wet-well into the six inch force main. There is a separate pipe vault which provides for connection to the force main.

It is recommended that in addition to a high level alarm, there should be an indication from the auto-dialer when more than one pump is running and an alarm to indicate when overflow occurs. Class one reliability is required for this pump station. It requires the ability to respond to emergencies and maintain required pumping rates at all times.

One option which the City has investigated for providing emergency power is to install a manual transfer switch at this pump station so that the auxiliary portable generator currently used at the on Timber Park pump station could be employed at both sites. Installation of this transfer switch is recommended.

The emergency pump has small fuel storage tank on the which requires hourly refills. Since this pump functions as a redundant pump it should be replaced with one with a larger fuel tank. The emergency pump is considered a dedicated pump.

Base flows in this basin are not expected to change. The thrust of this report has been that an aggressive I/I program is necessary to prevent an increase in I/I and that peak flows may be reduced. Therefore total flows are not expected to increase. The pump station must have the ability to pump current peak flows with one pump off line. This redundancy is provided by the emergency pump.

The force main discharges to manhole 98 on Highway 224. This manhole was inspected and found to be in excellent condition. The sides of the manhole were probed with a screw driver and showed no signs of deterioration. The manhole which serves as the wet well for the pump station was observed to be structurally sound. The ladder in this manhole is rusted and needs to be repaired.

7.3.2 Timber Park Pump Station

The Timber Park Pump Station is a new pump station designed in the short term to serve 126 acres of Light Industrial Development with a projected peak flow of 0.46 MGD (320 gpm) including flow from the Campus Pump Station. Ultimately the Timber Park Pump station will also receive flow from the proposed Duus Rd and River Rd. interceptors which serve industrially zoned areas east of the highway. Flow from these interceptors will flow into the 15-inch interceptor upstream of the pump station on River Road.

The combined peak flow of the Timber Park, Duus Road, and River Road interceptors is projected to be 2.06 MGD (1430 gpm). Although the duplex pumping station at Timber Park is presently equipped with pumps rated at 550 gpm each, the 8 inch force main and wet well were designed for peak flows of two to three times this amount and will have the capacity for the projected peak flows.

The pump station plan review submitals and design data sheets for the Timber Park Pump and Campus Pump stations are included Appendix III.

7.3.3 Campus Pump Station

The Campus Pump station, located north of River Mill Rd and west of Highway 224, is designed to serve 30 acres of light industrial businesses. The pump station is not expected to serve any additional area. If additional industrial property to the west should be developed, an additional pump station would be required. The duplex pump station has two submersible pumps rated at 120 gpm each.

Table 7-10: Lakeshore Drive Sewage Pump Station Design Data	
Area Served	75 acres of single and multi family residential including 3 acres which are commercially zoned, 262 EDU
Pump Station	
Type:	Dry Pit , vertically mounted, torque - flow pump
Pump type:	Constant Speed, 1170 rpm , recessed impeller, vortex flow
Design Capacity	150 gpm at 30 ft. total dynamic head
Maximum Capacity	300 gpm -350 gpm (from pump curve)
Pump Hp (each)	5 HP
Level Control Type	Bubbler tube
Overflow	416 ft.
Overflow Discharge	To street
Average time to Overflow	100 gpm, 15 minutes
Auxillary Power	none
Pump bypass	Emergency bypass vault and 6 inch line to upstream manhole for use with gasoline powered 300 gpm emergency pump.
Pump fuel tank capacity	1 hour
Alarm Telemetry type	Autodialer to STP
Force Main	
Length, Type	180 ft. 6 -inch ductile iron
Profile	Continuously ascending at aproximately 7% slope
Discharge manhole	#46 on 18 inch interceptor on Hwy 224
Air Release	none
Vacuum Release	none
Average detention	2.7 min (100 gpm:)

During a high flow event corresponding to the five-year storm, the PIF at the Lakeshore Drive pump station is estimated to be as high as 0.91 MGD (632 gpm). This estimate is based on 16% of the peak day I/I , 3.5 MGD, plus a peak base flow of 0.23 MGD. At these flows, the level in the manhole is above the starting level of 406 ft but below the overflow at 416 feet. At a reduced dynamic head condition the pump curve, included in Appendix 3, indicates a maximum pumping rate of 350 gpm.

Reserve capacity during high flow events greater than the five year storm can be supplied by a portable 500 gpm pump which can be set up to pump from the wet-well into the six inch force main. There is a separate pipe vault which provides for connection to the force main downstream of the dry pit.

The the thrust of this report has been that peak flows can be reduced by an aggressive I/I program. The design capacity of a single pump is equal to the average $MMWWF_{10}$, and because this basin is built out this flow is not expected to change. Upgrading to a higher capacity pump is not recommended at this time.

It is recommended that in addition to high level alarms, there should be an indication from the auto-dialer when more than one pump is running. Another deficiency that should be noted is the small fuel storage tank on the emergency pump which requires that it be filled on an hourly basis

One option which the City has investigated for providing emergency power is to install a manual transfer switch at this pump station so that the auxiliary portable generator currently used at the on Timber Park pump station could be employed at both sites.

7.3.2 Timber Park Pump Station

The Timber Park Pump Station is a new pump station designed in the short term to serve 126 acres of Light Industrial Development with a projected peak flow of 0.46 MGD (320 gpm) including flow from the Campus Pump Station. Ultimately the Timber Park Pump station will also receive flow from the proposed Duus Rd and River Rd. interceptors which serve industrially zoned areas east of the highway. Flow from these interceptors will flow into the 15 inch interceptor upstream of the pump station on River Road.

The combined peak flow of the Timber Park, Duus Road, and River Road interceptors is projected to be 2.06 MGD (1430 gpm). Although duplex pumping station at Timber Park is the presently equipped with pumps rated at 550 gpm each the 8 inch force main and wet well were designed for peak flows of two to three times this and will have the capacity for the projected peak flows.

The pump station plan-review submitals and design data sheets for the Timber Park Pump and Campus Pump stations are included Appendix III.

7.3.3 Campus Pump Station

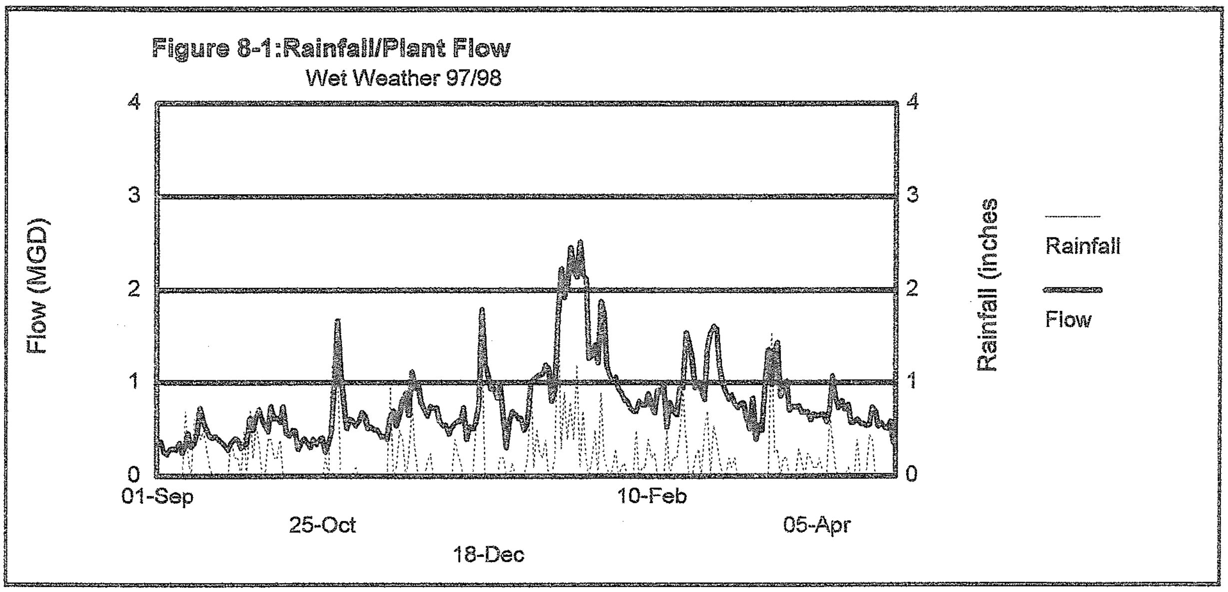
This pump station, located north of River Mill Rd and west of Highway 224 is designed to serve 30 acres of light industrial businesses. The pump station is not expected to serve any additional area. If additional industrial property to the west should be developed an additional pump station would be required. The duplex pump station has two submersible pumps rated at 120 gpm each.

CHAPTER 8: INFLOW AND INFILTRATION

8.1 CURRENT MAGNITUDE OF INFLOW AND INFILTRATION

Wastewater flow rates are important factors in evaluating the capacity of an existing collection system. This occurs during the wet weather season when high groundwater coupled with a significant rainfall event lead to massive infiltration and inflow (I/I) of stormwater into the collection system. Flow to the wastewater treatment plant ranges from an Average Dry Weather Flow of 0.43 MGD to a Peak Daily Flow of 3.9 MGD; nearly 10:1.

High flows exert a significant stress on the treatment plant process. The lowered detention time in the solids contact basin and increased hydraulic loading on the effluent sand filters results in a lowered efficiency for these processes. During an extended high flow event, compliance for permitted mass loadings may be difficult to achieve, emphasizing a need for the City to have an aggressive approach to reducing I/I.



8.1.1 Infiltration

The City is currently experiencing a serious infiltration problem due to the inevitable deterioration of the collection network over time. Infiltration becomes a problem when groundwater rises above the level of the sewage collection system, effectively submerging the pipe network. When this occurs, any deteriorated portions of the collection system are subject to leakage. This deterioration includes cracked or broken pipes, which is inevitable in any sewer system, and unsealed pipe or manhole joints.

According to the EPA, infiltration is excessive if the highest average daily flow over a period of 7 to 14 days during a period of high groundwater and dry weather markedly exceeds 120 gpcd. The flow at the Estacada Treatment Plant during periods of when there is no rainfall, but when the groundwater table can be assumed to be elevated, from November - April, has averaged 0.75 MGD over the last three years. Given the present population of 2190, there is an average per capita flow during these no rainfall periods of 342 gpcd. This is far in excess of accepted threshhold levels.

Peak flows occur after periods of rainfall which raise the ground-water table and may dramatically increase infiltration when service laterals are submerged. Flow monitoring during the 1986 study indicated that infiltration rates increased dramatically following rainfall events and peak infiltration was much higher than the average value. Smoke testing found 85 faulty service connections that were significant contributors to peak infiltration. Repair of these service connections resulted in an estimated removal of 1.22 MGD.

8.1.2 Inflow

Inflow is the result of stormwater discharged into the collection system directly through roof and area drains, combined stormwater and sanitary sewer networks, and holes in manhole lids. Inflow occurs during and/or immediately following a storm event. Although infiltration comprises the majority of the excess flow, the rapid response of the treatment plant flows to rainfall events indicate that there is a significant inflow effect.

Wet weather flow events early in the rainy season illustrate the magnitude of inflow because the ground water level is below the level of the collection system. The wet - weather period of 1999/2000 was preceded by dry antecedent conditions and a base flow averaging 0.32 MGD. Treatment plant flow for several weeks in October and November of 1999 is shown in Figure 8-2. During this period 1-inch of rainfall inflow added more than 200,000 gpd and the two day total from one day of rain was nearly one-half million gallons.

The pattern of flow indicates that inflow amounts are cumulative. Flows do not immediately return to base conditions following a rainfall event. Although this delayed component is attenuated, the total amount of inflow is proportional to the rainfall

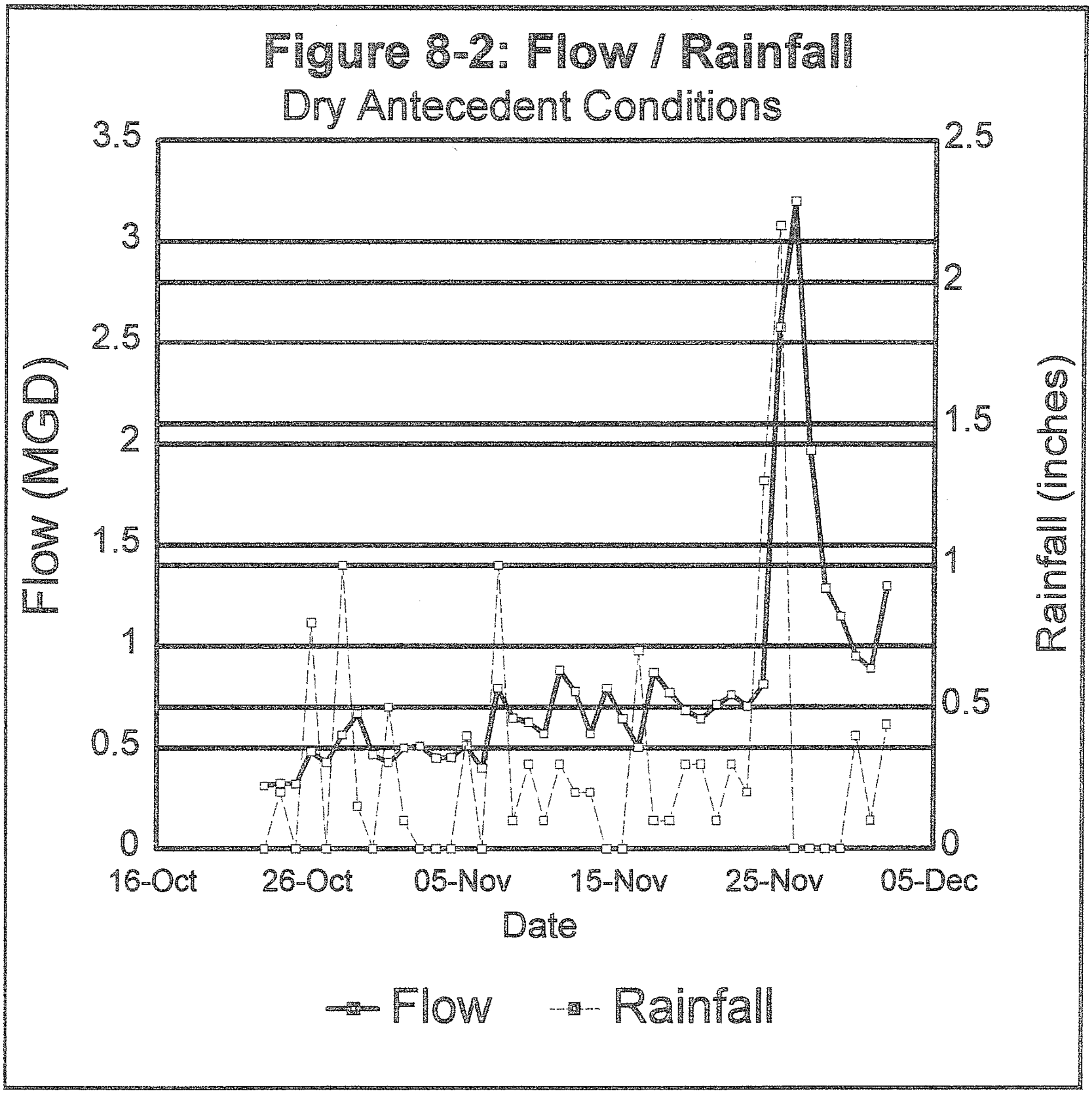


Figure 8-2: Flow / Rainfall
Dry Antecedent Conditions
3.5
3
2.5
2
1.5
1
0.5
0
Flow (MGD)
2.5
2
1.5
1
0.5
0
Rainfall (inches)
16-Oct
26-Oct
05-Nov
15-Nov
25-Nov
05-Dec
Date
Flow
Rainfall

8.1.3 I/I Analysis by Basin

The purpose of the flow monitoring was to quantify the proportion of rainfall induced Inflow and Infiltration (I/I) contributed by each basin, and to determine where the City should focus its control program. Locating specific inflow sources and pipes which need to be repaired or replaced requires smoke testing and television inspection of the area in question which was beyond this study scope.

Examination of flow records indicates that the treatment plant flows will return to base flow conditions following a rainfall event until there has been an average cumulative rainfall of approximately 12 inches. This is the point at which the ground is saturated above the level of most of the collection system and there is a steady rate of infiltration even during periods without rainfall.

Flow monitoring was done after the ground was saturated so that measurements would include both inflow and infiltration. Treatment plant flows had been greater than 1 MGD for the week preceding the first day of sampling on December 1, 1999 following a Thanksgiving Day storm with treatment plant flows of 2.58 MGD and 3.2 MGD November the 24th and the 25th.

On the first day of sampling there had been no rainfall for the preceding 42 hours and treatment plant flows were averaging 0.9 MGD. The flows measured the morning of December 1 are indicative of rates of infiltration.

On the following morning sampling was done following a 5 hour period of heavy rainfall. Treatment plant flows averaged 1.55 MGD during the 2 hr sampling period. The basin flows measured on December 2 therefore include inflow and infiltration.

Flows were measured at downstream manholes on the four main lines and the relative contribution of each basin to I/I was calculated as a percentage of the total flow. The flow monitoring was done within a relatively short time frame and it was assumed that the base flow contribution did not change significantly. Base flow was assumed to be 0.37 MGD divided equally between the four basins.

The results of the flow monitoring are presented in the following table, and compared to historical data of basin contributions to excess flow. Trunk line #1 contributed the highest percentage of flow and also showed the greatest increase in flow immediately following the rainfall, indicating significant inflow.

Flow from Subbasin #3 flows into the pumpstation on Lakeshore Drive and was measured in the manhole preceding the pump station. Flow from this subbasin averaged 16 % of the total flow. Flow from Subbasin #3 was subtracted from the totals in Table 8-1 in order to compare to earlier data which did not include Lakeshore Drive measurements

Table 8-1: Flow Monitoring Summary				
	Subsystem Flows			Total Flow (MGD)
	1 (MGD)	2 (MGD)	4 (MGD)	
December 7, 1999	0.645	0.491	0.210	1.345
	41 48%	31 36%	.13 16%	
December 2, 1999	0.407	0.250	0.240	0.897
	39 46%	24 28%	23 26%	
December 1, 1999	0.250	0.170	0.165	0.585
	37 43 %	25 29 %	24 28 %	
February 21, 1989	0.41	0.26	0.23	0.92
	45%	28%	27%	
February 17, 1989	0.66	0.45	0.29	1.40
	47%	32%	21%	
April 4,8, 1988	0.37	0.45	0.23	1.05
	35%	43%	22%	
March 24, 25, 1988	0.43	0.54	0.27	1.24
	35%	43%	22%	
March 23, 1988	0.24	0.30	0.26	0.80
	30%	38%	32%	
1986 (Facilities Plan)	0.75	3.05	0.90	4.70
	16%	65%	19%	

1.560
1.04 w/25
0.678

184

Flows at MH-178 above the mill property on main line #4 were measured on both days . The difference between the flow measured at this manhole and MH-148 downstream of the lumber mill indicates the amount of I/I contributed as the line passes though the lumber mill property. Similar measurements were made at MH 147A at 4th and Wade on main line #4. 2

Table 8-2: Flow Measurements Indicating Flow Contribution Across Mill Property						
	Main #2			Main #4		
	MH 163	MH 147 A	Difference	MH 184	MH 178	Difference
	MGD	MGD	MGD	MGD	MGD	MGD
December 1,1999	0.17	0.11	0.06 ~~0.66~~	0.165	0.107	0.058
December 2, 1999	0.25	0.25	0	0.25	0.14	0.11

On December 2, flow monitoring followed a five hour period of steady rainfall. There was an increase of 100,000 gallons in the flow contributed across the lumber yard on this day, indicating a possible inflow contribution.

Television inspection reports from 1996 indicated that there were holes and a pulled joint in the alley east of Currin Street. Downstream of this line at MH-66 significant infiltration was observed and the following flows were recorded: December 1: 7.67 gpm, and December 2: 37 gpm.

8.2 I/I REDUCTION

Estacada has had a continuing commitment to reducing I/I reduction in the collection system. Earlier infiltration abatement programs targeted subbasin #2 and the improvement in this subbasin can be seen in the decreased percentage of excess flow contributed by this basin. between 1986 and 1988.

Phase one of the I/I abatement project in 1987 included removal of inflow from roof drains, and leaking service connections. Infiltration reduction resulted from replacement of broken pipe joints, manhole repairs, and chemical grouting of 6900 ft. of line.

Between 1988-1989, phase two of the project included replacement of 1330 ft of pipeline along Pierce Street, replacement of service laterals, and 1140 ft. of pipeline along N.W Wade Street, and sliplining of 1000 ft of Main #2 through the lumber mill property.

Improvements to the collection system since 1990 include rehabilitation work in Subbasin #1 resulting in the replacement of the entire main line along South 3rd Street and south to 4th Street east of Main Street. As part of the 3rd St. sewer and water improvements in 1995, the main line for Subsystem 1 was connected to the line from Regan Hill area at 4th and Shafford. Improvements to the collection system are itemized in Table 8-2.

Table 8-3: I/I Sewer Projects and Repairs		
Project Name	Cost	Date Completed
Post Office Alley Utility Rehabilitation	$108,130	March 99
3rd Ave Rehab between Main & Wade	$139,065	February 98
TV inspection of 1435 LF $0.45	$646	February 97
N. Broadway 360' PVC lined	$26,450	June 96
Cleaning & TV inspection of 1401 LF $1.07	$1,499	June 96
3rd Ave., Sewer and Water Improvement	$28,679	January 95
Total Project Costs	$304,469	
Repairs and New Services	$2,472	

Television inspection in 1994, and 1996 of several laterals along 3rd St. in basin #1, indicated the potential for significant infiltration due to cracked pipes and pulled joints. The results of the TV inspection are summarized in Table 8 -3.

Estacada has had an I/I control program which includes a program of cleaning the entire system and television inspection of problem areas. Basin #3 was recently completed (1997) and it is estimated that it will take 5-7 years to complete the entire system.

Table 8-4: Recent TV Inspections			
Date	MH	Location	Description of Problems
5-25-94	87-86	S.W Third	Pulled or offset joints (6), CC (4) LC (1)
5-27-94	177-178	N.W. Wade	Pulled joints (3), Pinhole leaks (2)
8-27-94	76-63	East of 3rd & Main	BP (1), Pulled joint (1)
3-26-96	64-66	SE 2nd near M. Church	Holes (2) Seperated Joint (1)
3-26-96	83	Pine Cone Alley-Third	Holes (3), BP(1), Seperated Joint (1)

CC: Circumferential Crack, LC: Longitudinal Crack, BP: Broken Pipe

8.2.1 Control Methods

There are several technologies available for the repair and rehabilitation of sanitary sewer pipe without excavation. These include slip lining, reconstruction with cured in place resins, pipe reconstruction with folded PVC, and chemical grouting. The choice of appropriate technology depends upon factors such as the degree of deterioration and structural integrity of the existing pipe.

The City of Estacada has had positive experience with the sliplining of existing pipe which was used to repair 1,000 ft of Main Line #2 which runs through Estacada Lumber Mill. The chemical grouting of lines in the 1987 I/I abatement program was less successful and at various times grout fragments have showed up at the treatment plant.

8.2.2 Recommendations

High rates of I/I were observed in basin #1, and contributed 43-45 % of the total flow during monitoring in December 1999. It is recommended that this basin be targeted for continued attention. Since the main line is recent in construction , efforts should target laterals in the older sections of the basins.

Smoke testing to identify inflow sources should be included in addition to continuing the ongoing program of cleaning and inspection in this basin. Specific lines should be targeted for repair based on careful review of television inspection videos of the problem areas.

There is an inflow component of I/I which is observed at the treatment plant 20 minutes after heavy rainfall. Line #4 which runs through the mill property has been identified as a potential inflow source. Investigation of this area should begin with inspection of all manholes on lines #2, and #4 through the mill property.

Smoke testing is an inexpensive method of identifying inflow sources and can be followed by TV inspection of the problem areas. Major sources of infiltration due to cracked lines may also be identified by smoke testing.

CHAPTER 9: CAPITAL IMPROVEMENT PLAN

9.1 RECOMMENDED IMPROVEMENTS TO THE TREATMENT SYSTEM

Improvements to the liquids processes of the existing treatment plant are necessary to provide pretreatment, pump projected peak flows, and comply with discharge regulations regarding chlorine. It is also recommended that the existing blowers be relocated to improve operating conditions in the control building while also providing an opportunity to increase blower capacity.

Improvements to the sludge storage ponds and purchase of biosolids application equipment are necessary to provide for projected biosolid production. These improvements are a priority because without adequate storage or disposal too many solids are carried in the secondaries or recycled with the decant from the sludge storage ponds.

Improvements to the solids handling system are also an operational priority. Currently, lack of mixing in the sludge storage pond causes the accumulation of a very concentrated layer of solids which is difficult to pump with the existing equipment.

Table 9-1: Proposed Improvements to Wastewater Treatment Plant

		Construction	Contingency	Engineering	Total
Biosolids Irrigation	2	$50,000	$5,000	$11,000	$55,000
Sludge Storage Modifications	3	$114,000	$11,400	$25,080	$150,480
Dechlorination	1	$22,000	$2,200	$4,840	$29,040
Recirculation Controller	4	$4,500	$450	$990	$5,940
Ras Pumps	5	$34,000	$3,400	$7,480	$44,880
Trickling Filter Pump Upgrade	6	$55,000	$5,500	$12,100	$72,600
Headworks Modifications	7	$116,000	$11,600	$25,520	$153,120
Blower Building / Control Bld.	8	$96,000	$9,600	$21,120	$126,720
Hypochlorite	10	$12,000	$1,200	$2,640	$15,840
Total Improvements					$653,620

The diffusers in the solids contact channel should be scheduled for inspection and replacement. There is adequate dissolve oxygen in the channels but mixing is uneven. All of the diffusers could be replaced at an estimated cost of $22,000. It is not essential to replace all diffusers at once and the work can be scheduled over several budget years.

9.2 COLLECTION SYSTEM

9.2.1 Sewer Improvements

The collection system capital improvement plan includes recommendations for extensions to the existing trunk lines, and new interceptors to serve properties within the Urban Growth Boundary (UGB). The interceptors are sized for ultimate buildout based on the land use designations developed in the City's land use plan.

Included in the capital improvement plan are upgrades to existing subsytem interceptors which will be necessary after the areas served by the proposed interceptors are developed.

Table 9-2: Proposed Interceptors

Interceptor	Design (MGD)	Estimated Cost	EDUs
Duus Rd	1.46	$1,057,700	1670
River Mill (east)	0.14	$119,000	207
Duus + R.M	1.60	$143,960	1877
Hinman	0.92	$1,314,800	1047
Cemetery Rd.	0.15	$168,000	176
East Extension	0.32	$472,640	356
Coupland Rd.	0.48	$399,437	638
Regan Hills	0.14	$343,560	154
Hwy 224, East	0.15	$412,370	168
New Interceptor Cost		$4,431,467	4416
Upgrades to existing lines		$191,240	287
Total		$4,622,707	4703

9.2.2 I/I Control Plan

Inflow and infiltration have resulting in peak days flows which stress the secondary treatment system and make it difficult to meet daily mass pounds limit for BOD. It is a recommended that Estacada continue to implement an aggressive program of I/I control. A strong investment now will reduce the stresses on the treatment processes and extend the life of the plant.

Identifying inflow sources has the greatest impact in terms of I/I removed per dollar spent. Main line #4 which runs through the Estacada Lumber Company logging yards was identified as a potential inflow source by the flow monitoring program. This and other suspected inflow sources should be identified with smoke testing followed by television inspection of problem areas.

I/I related improvements are appropriately funded from the Operation Maintenance and Replacement (OMR) budget. Estacada has had in place a program of line cleaning followed by TV inspection where excessive accumulation of silt and grit indicates a problem.

Money for smoke testing and additional inspection should be added to the OMR budget .The budget should also include money for replacement or repair of sewer lines An annual budget of $50,000 per year is recommended for effective I/I control.

9.3 FINANCIAL PLAN

9.3.1 System Development Charges

Local governments may impose System Development Charges (SDCs) for financing specified capital improvements in accordance with Oregon Revised Statutes (ORS) 223.297-223.314. Fees may be based on reimbursement of value of capital improvements already constructed and anticipated costs of specified improvements to be constructed.

SDCs may be assessed or collected at the time of increased usage of a capital improvement or issuance of a development permit, building permit or connection to the capital improvement. SDC revenue can be used for any capital improvement project for the utility for which they were collected, including pre-design studies, design, construction, and installation of capital improvements but not for the costs of operation or routine maintenance.

The objective of reimbursement fees is to assure that future users contribute an equitable share of the capital cost of existing facilities. Reimbursement fees are based on the current value of the unused capacity of the treatment plant prorated to future users. To determine the reinbursement fee, the city must consider ... "the cost of the existing facility or facilities , prior contributions by existing users,
the value of unused capacity, rate making principles employed to finance publicly owned capital improvements and other relevant factors..."(ORS 223.304(1)) .

The methodology which has been adopted by the City is to take the depreciated value of the treatment plant less the federal grant divided by the number of EDUs served at 100% capacity.

An alternative methodology is to evaluate the current value of the treatment plant by using the original cost value evaluated in terms of today's dollars less the federal grant. Instead of depreciating the original value, the historical rate of inflation is applied to the original cost. This methodology results in a larger reimbursement fee and one that is more indicative of the actual value of the remaining capacity .

For comparison, we used the first two methods both methods of determining the treatment plant value were used in calculating the reimbursement fee. The results are included in the following tables as current methodology, and alternative methodology. Any change to the current methodology for calculating the reimbursement fee would need to be formally adopted by the City Council.

A third option for evaluating the current value of the treatment plant is to estimate today cost of replacement. This is probably the most accurate method of evaluating current value should the City choose to change its methodology.

Our analysis shows that the wastewater treatment plant processes will perform within the limits of the NPDES permit up to an average monthly flow of 1.5 MGD. The projected Maximum Month Wet Weather ($MMWW_5$) flow is 1.49 MGD in 2020 when there are 1970 EDUs served. At this point the plant is estimated to be at 100% capacity.

9-4: Reimbursement Fee for STP Current Methodology

		Project Cost	Federal Grant	Reimbursable Cost	Value Remaining	Users EDUs	Fee / EDU
1989	STP Expansion	$1,811,816	$1,039,789	$117,027	$308,800	1970	$157
1994	Lime Stabilization	$50,000	-	-	$43,970	3565	$12
Reimbursement Fee for Sewage Treatment Plant							$169

~~2,951~~
67
3,018

9-5: Reimbursement Fee for STP Alternative Methodology

		Project Cost	Federal Grant	Value Remaining	Present Value @3%	Users EDUs	Fee / EDU
1989	STP Expansion	$1,811,816	$1,039,789	$772,027	$1,068,666	1970	~~$542~~
1994	Lime Stabilization	$50,000			$57,964	3565	$16
Reimbursement Fee for Sewage Treatment Plant							~~$559~~

1127
+16
1115

Improvement fees are based on identified projects and estimated costs of construction of future capital improvements.

"Improvement fees shall be spent only on capacity increasing capital improvements including expenditures relating to repayment of debt for such improvements. An increase in system capacity may be established if a capital improvement increases the level of performance or service provided by the existing facilities. The portion of such improvements funded by improvement fees must be related to current or projected development."(ORS 223.307)

Table 9-6: Improvement Fee for STP

	Cost	Edu	$ / EDU	Recovered cost
Prescreening	$153,120	1970	$78	$55,963
Trickling Filter Pump Upgrade	$72,600	1970	$37	$26,534
Recirculation Valve	$5,940	1970	$3	$2,171
RAS Pumps	$44,880	1970	$23	$16,403
Dechlorination	$29,040	1970	$15	$10,614
Hypochlorite	$15,840	1970	$8	$5,789
Blower Building / Control Bld.	$126,720	1970	$64	$46,314
Spray Gun Application	$55,000	720	$76	$55,000
Sludge Storage Modifications	$150,480	720	$209	$150,480
	$653,620		$513	$369,267

The capital improvement plan identifies improvements to the treatment plant which should be implemented within the next 10 years. The improvements are based upon projected flows and loadings at 1970 EDU (2020 projection). The solids improvements which directly increase capacity are paid for by new users up to the design capacities of 1970 EDU. The cost of improvements which increase the level of performance of the existing plant are borne by existing and future users.

Sewer improvement fees are based on the proposed new interceptors and existing system upgrades at system buildout. The total cost of the sewer capital improvements is paid for by future system users. Total improvement cost is divided by the total EDU at buildout minus existing EDUs minus EDU in the existing improvement district. The improvement district for the River Mill Industrial Area Infrastructure Expansion includes the 132 acres of industrially zoned area west of Highway 224 (designated as the Timber Park subsytem in Table 7-3).

Table 9-7: Sewer Improvement Fee		
Total Improvement Cost	EDU	$ / EDU
$4,622,707	4703	$983

Table 9-8: Value of Existing Interceptors			
Existing Interceptors	LF	$ / LF	Value
10"	5176	$72	$372,672
12"	6854	$78	$534,612
18"	2578	$92	$237,176
			$1,144,460

The sewer reimbursement fee is based on interceptors within the existing sytem which will be used to service the entire UGB. The replacement value of existing lines greater than 8" is 1.14 million dollars. This amount is divided by the total number of EDUs at buildout excluding the Timber Park Improvement District and Lakeshore Drive.

Table 9-9: System Development Charges					
	Reimbursement		Improvement	Total	
	Current Methodology	Alternative Methodology		Current Methodology	Alternative Methodology
Sewage Treatment Plant	$173	$559	$462	$635	$1,021
Collection System		$200	$966	$966	$1,166
Total SDC				$1,601	$2,187

9.3.2 Rate Analysis

Estacada bases sewer rates on an estimate of the relative contribution of each connection in equivalent dwelling units (EDUs) at a rate of $16.40 per month per EDU.

Table 9-10: Summary of Equivalent Dwelling Units (EDU)		
	Connections	EDUs
Residential Connections	657	665
Apartments	7	87
Commercial	105	332
Schools	4	133
Public Institutions	9	10
Industrial	3	23
Totals	785	1250

Table 9-11: EDU Conversions	
Dwelling	1 EDU / dwelling for single and multiple dwellings, apartments,& trailers
Schools	G.S.: 1 EDU / 12.5 students, H.S.: 1 EDU / 12.5 students
Motels	With kitchen 1 EDU / unit, Without kitchen 1 EDU / 3 units
Hospitals	1 EDU / 2.5 beds
Industrial/Commercial	1 EDU / 10 employees or 1 EDU / establishment
Offices	1 EDU / 10 employees
Churches	1 EDU / church
Service Stations	< 4 pumps 1 EDU, 5-12 pumps 2 EDUs, > 13 pumps 3 EDUs
Garages	1 EDU / garage
Restaurants	1 EDU / 10 seating spaces
Laundries	1 EDU / machine
Grocery Stores	2 EDU / 3,000 square ft.

Revenue from monthly sewer fees pays for operation, maintenance, and replacement (OMR)of the treatment plant and collection system. Of the fees collected, 10% are placed in the system reserve fund. Based on the actual revenues and expenditures in 1999, it was necessary to use all of the money collected plus additional from the system improvement reserve.

The base fee should be reviewed on an annual basis to assure the solvency of the enterprise or system reserve fund. It is appropriate to include an increase in operations funding to account for inflation.

Table 9-12: Budget Summary			
	Treatment	Collection	
Expenditures	$202,157	$59,076	$261,233
Collection for Services	$219,988	$25,000	$244,988
Other Fees (excluding SDCs)	$10,415	$1,266	$11,681
Total Revenue 1999	$230,403	$26,266	$256,669
Excess of revenues over (under) expenditures			($4,564)

The sewer system reserve fund currently has retained earnings of $172,949. Of this SDC reserves total $68,000 and this money is being used to pay for instrumentation system at the wastewater treatment plant. SDC money will be available for additional reimbursable improvements in 2001. It is estimated that SDC charges will be collected at an average of $16,000 to $20,000 per year.

It is practicable to implement these improvements on a pay as you go basis over a five year period with an average annual improvement cost of $130,000. It is assumed that in 2001 there will be $20,000 dollars per year available from SDCs leaving $110,000 per year as user costs.

These improvement costs are expected to increase with inflation which is offset by the projected growth rate of 2.3%. The number of EDUs increase at approximately the same rate as the cost.

There is currently $ 10,000 per year budgeted for I/I control. It is recommended that this amount be increased to $50,000 per year. This is a rate increase of $2.66 / EDU / month

Table 9-13: Adjustments to Rates						
Treatment Plant Improvement Cost		$$ / EDU /Mo.				
Annual Cost	EDUS	Improvements	I/I Control	Base Rate	Inflation Adjustment	Total
$110,000	1250	$7.33	$2.66	$16.40	$1.00	$27.39

Alternatively, the city could choose to spread the cost of the improvements over a longer time period by borrowing the money. Low interest loans and partial grant finding may be available in the form of Community Development Block Grants (CDBG) administered by Clackamas County Community Development or funding through the Water /Wastewater Financing Program of Oregon Economic and Community Development (OECDD).

At an interest rate of 5% the annual cost of a $650,000 loan over 20 years is $52,157. Assuming that 20,000 per year of SDC money will be available, the increase in rates for treatment plant improvements would be $2.14 / EDU / month. Including an increase in rates for I/I control, the rate paid by current users would be $22.26 per month. However, in 20 years, the City will have spent $1,043,000 in debt payment or 393,000 in interest.

We recommend that a pay as you go approach be taken and funding provided from rates. Priorities and budgets can be prepared to accomplish needed biosolids projects first and dechlorination and blower building projects to follow.

CHAPTER 10: FUTURE CONSIDERATIONS

10.1 TREATMENT PLANT EXPANSION

The capacity of the Estacada wastewater treatment plant to meet discharge limitations within the twenty year planning period has been discussed in detail. With an aggressive approach to I/I and implementation of the capital improvement plan, it is expected that the plant will continue to meet permit until average wet weather flows exceed the wet weather design capacity of the treatment plant at 1970 EDUS.

At 1970 EDUs the City of Estacada will be at less than one third of ultimate build-out which is estimated to be 6,500 EDUs. There is some room for expansion of secondary treatment and effluent filtration at the current plant site but there is not enough space to triple or even double the capacity.

The City needs to consider sites for future plant expansion. One potential location would be nearby land in Timber Park currently owned by PGE.

10.2 EXCESS FLOW STORAGE AND IRRIGATION

Because of restrictions on discharge to the Clackamas River imposed by the Three Basin Rule, the mass discharge limits are not expected to increase. For the purpose of analyzing the long range implications of these discharge limits on development, it is assumed that as future treatment plant expansions occur with growth it will be possible to produce treated effluent with BOD and SS concentrations which average 5 mg/l in both the summer and winter months. Permits requiring effluent concentration of 5 mg/l are not uncommon and are well within the range of current technologies.

Table 10.1 Currently Permitted Mass Loading

		Monthly Ave		Weekly Averages		Daily Maximum	
		lbs / day	@ (5 mg/l) Flow MGD	lbs / day	@ (5 mg/l) Flow MGD	pounds/day	@ (5 mg/l) Flow MGD
May1 - Oct. 31	BOD	45	1.079	68	1.63	90	2.16
	SS	45	1.079	68	1.63	90	2.16
Nov. 1 - April 30	BOD	90	2.16	135	3.23	180	4.32
	SS	90	2.16	135	3.23	180	4.32

The maximum pound limits for wet weather flows will be limiting first. Currently the peak day flow of 3.9 MGD is 10 times the base flow of 0.36 MGD. Even with an aggressive approach to I/I, wet weather flows will continue to more than double dry weather flows. When it is not possible to meet effluent pounds limits it may be feasible to store the excess treated effluent for irrigation during the summer months.

To estimate how much storage would be required, projected base flows were added to plant flows for the wet weather season 1998/1999 to create a realistic flow pattern. Using the load limit of 180 lbs per day and the practicable performance of 5.0 mg/l results in a flow limit of 2.16 MGD. Flows greater than 2.16 MGD would need to be stored.

Table 10-2 Storage of Wet Weather Flows in Excess of Mass Disharge Limits

EDU	Year @ 2.3%	Base Flow	Storage		
		MGD	MG	Acre-ft.	Acres @6ft Depth
2500	2000	.72	6	17	3
3750	2049	1.09	11	35	6
4375	2055	1.28	32	98	16
5000	2061	1.46	50	153	26
6508	2073	1.89	102	313	35

To store 30 MG (92 acre ft), 15 acres would be required. Land north of the UGB has been considered as a possibility because of its suitability for development for large scale irrigation. Larger volumes of storage become infeasible.

Treatment technologies which can reduce produce treated effluent to 2 - 3 mg/l include advanced tertiary treatment with chemical addition, mixed media effluent filtration, and ongoing advances in membrane filtration techniques. Many of these are developing technologies which will be tested in the next twenty years.

The cost of treating to these stringent levels is expected to decrease. Storage and irrigation is an alternative for achieving mass loadings for the next fifty years, but employing more advanced treatment methods will ultimately be necessary.

If effluent is treated to a achieve an effluent consistently below 3 mg/l, the average monthly wet-weather mass loading of 90 pounds can be met at buildout, without storage. This assumes that peak flows are brought under control with an I/I program. A flow of 3.59 MGD, the average discharge which would be allowed at an effluent concentration of 3 mg/l, is less than the Projected $MMWWF_5$ at buildout.

Advance tertiary treatment will ultimately need to be implemented and with new technologies, and some of the treatment techniques employed in the water industry, storage will not be necessary.

However, these costs may be deferred and beneficial use may be made of the reclaimed wastewater by storage and irrigation.

The advantage of having land available for storage and irrigation are that the capital and operational costs associated with treating to these low levels could be forestalled. In the short term City owned land could be used for application of biosolids.

10.3 LAND FOR BIOSOLIDS APPLICATION

The use of City owned land for the application of biosolids was discussed as part of a strategy to use biosolids for a greater part of the year eliminating the need for large storage capacities at the treatment plant. Currently the availability of land for the application of biosolids is dependent upon the on the land owners needs and the ability to deliver the biosolids when crops dictate application. If the City owned its own land for biosolids application, a more consistent operation could be possible.

Table 10-3 : Projected Biolsolids Land Application Needs

EDUs	Solids Production lbs/year	Available Nitrogen lbs/ year	Acres
1250	155490	2934	29
1970	245052	4624	46
2500	310980	5868	59
3750	466470	8801	88
5000	621960	11735	117
6508	809543	15274	153

To be able to apply solids year round, at least half of these site would need to be in upland regions where the ground is not saturated during the winter months. In general biosolids and domestic septage must be applied at rates and methods which prevent the occurrence of runoff, erosion, leaching and nuisance conditions.

10.4 FINANCIAL CONSIDERATIONS

Investment in land for irrigation and winter storage of excess discharge and or for application of biosolids is an investment which would increase the capacity of the treatment plant. The capital expenditures would be eligible for SDC reimbursement. The improvement cost (or reimbursement cost after the property is purchased) would be determined by dividing the capital cost by the number of EDUs served by the available storage.

APPENDIX I

NATIONAL POLLUTION DISCHARGE
ELIMINATION SYSTEM (NPDES) WASTE DISCHARGE PERMIT

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM
WASTE DISCHARGE PERMIT

Department of Environmental Quality
2020 S.W. Fourth Avenue, Suite 400, Portland, OR 97201-4987
Telephone: (503) 229-5263

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

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ISSUED TO:
City of Estacada
PO Box 958
Estacada, OR 97023

PLANT TYPE AND LOCATION:
Trickling Filter/Solids Contact WTP
Adjacent to Timber Park and Highway 211/224
Estacada, Oregon.
Treatment System Class: III
Collection System Class: II

SOURCES COVERED BY THIS PERMIT

Type of Waste	Outfall No.	Outfall Location
Domestic Sewage	001	R.M. 23.6

RECEIVING STREAM INFORMATION:
Basin: Willamette
Sub-Basin: Clackamas
Receiving Stream: Clackamas River
Hydro Code: 22N-CLAC 23.6 D
County: Clackamas

EPA REFERENCE NO : OR-002057-5

Issued in response to application no. 992330 received January 24, 1997.

Tom Bispham

Tom Bispham, Administrator
Northwest Region

February 3, 1998
Date

PERMITTED ACTIVITIES

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

Unless authorized by another NPDES permit, each other direct and indirect waste discharge to public waters is prohibited.

SCHEDULE B
Minimum Monitoring and Reporting Requirements
(unless otherwise approved in writing by the Department)

The permittee shall monitor the parameters as specified below at the locations indicated. The laboratory used by the permittee to analyze samples shall have a quality assurance/quality control (QA/QC) program to verify the accuracy of sample analysis. If QA/QC requirements are not met for any analysis, the results shall be included in the report, but not used in calculations required by the permit. When possible, the permittee shall re-sample in a timely manner for parameters failing the QA/QC requirements, analyze the samples, and report the results.

1. Influent

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD or l/s)*	Daily	Reading
Flow Meter Calibration*	Annually	Verification
pH	3/Week	Grab
Biochemical Oxygen Demand (BOD_5)	2/Week	24-hr Composite
Total Suspended Solids (TSS)	2/Week	24-hr Composite

2. Outfall Number 001 (Wastewater Treatment Plant Outfalls)

Item or Parameter	Minimum Frequency	Type of Sample
Total Flow (MGD or l/s)*	Daily	Reading
Flow Meter Calibration*	Annually	Verification
pH	3/Week	Grab
Total Residual Chlorine	Daily	Grab
Biochemical Oxygen Demand (BOD_5)	2/Week	24-hr Composite
Total Suspended Solids (TSS)	2/Week	24-hr Composite
Average Percent Removal (BOD_5 & TSS)	Monthly	Calculation
E. Coli (See Note 1/)	Weekly	Grab

*Total flow measurement required only at one site, whichever is most appropriate.

3. Biosolids Management

Item or Parameter	Minimum Frequency	Type of Sample
Biosolids analysis including: Total Solids (% dry weight), Volatile Solids (% dry weight), pH (standard units), Biosolids nutrients for NH_4-N; NO_3-N; TKN; Total Phosphorus; Potassium (% dry weight), Biosolids trace inorganics for As; Cd; Cu; Hg; Mo; Ni; Pb; Se; Zn (mg/kg dry weight)	Annually	Composite samples to be representative of the biosolids product to be land applied (See Note 2/)
Quantity and type of alkaline product used to stabilize biosolids (when required to meet Federal pathogen and vector attraction reduction requirements in 40 CFR § 503.32(b)(3) and 40 CFR § 503.33 (b)(6).	Each occurrence	Measurement
Initial time when solids that received alkaline agent ascended to pH 12.	Each batch	Date, time and actual pH measurement
2 hours after initial alkaline addition and sustained at pH 12.	Each batch	Date, time and actual pH measurement
24 hours after initial alkaline addition and pH 11.5 was achieved.	Each batch	Date, time and actual pH measurement

SCHEDULE A

1. **Waste discharge limitations not to be exceeded after permit issuance.**

a. Outfall Number 001:

Parameters	Limitations				
	Average Effluent Concentrations		Mass Loading*		
	Monthly	*Weekly*	*Monthly Average lb/day*	*Weekly Averages lb/day*	*Daily Maximum lbs*
May 1 - October 31					
Biochemical Oxygen Demand (BOD5)	10 mg/l	15 mg/l	45	68	90
Total Suspended Solids (TSS)	10 mg/l	15 mg/l	45	68	90
November 1 - April 30					
Biochemical Oxygen Demand (BOD5)	20 mg/l	30 mg/l	90	135	180
Total Suspended Solids (TSS)	20 mg/l	30 mg/l	90	135	180
Year-round					
pH	Shall be within the range of 6.0 - 9.0 s.u.				
Escherichia coli (E. coli)	30 day log mean of 126 organisms per 100 ml and no single sample shall exceed 406 organisms per 100 ml. If a single sample exceeds 406 organisms per 100 ml, additional sampling per Schedule B is mandated.				
BOD5 and TSS percent removal efficiency	Shall not be less than 85 percent monthly average				

*Mass loading based on an average dry weather design flow equaling 0.54 MGD

b. Notwithstanding the effluent limitations established by the permit, except as provided for in OAR 340-45-080, no wastes shall be discharged and no activities shall be conducted which will violate water quality standards as adopted in OAR 340-41-445 except in the defined mixing zone:

The mixing zone shall not extend beyond a radius of seventy-five(75) feet from the point of discharge.

Item or Parameter	Minimum Frequency	Type of Sample
Record of locations where biosolids are applied on each DEQ authorized land application site (Site location maps to be maintained at the treatment facility for review upon request by DEQ).	Each occurrence	Date, quantity (gallons) and locations where biosolids were applied recorded on site location map.

Notes:

1/ E. coli monitoring must be conducted according to any of the following test procedures as specified in **Standard Methods for the Examination of Water and Wastewater, 19th Edition**, or according to any test procedure that has been authorized and approved in writing by the Director or his authorized representative:

Method	Reference	Page	Method Number
mTEC agar, MF	Standard Methods, 19th Edition	9-28	9213 D
NA-MUG, MF	Standard Methods, 19th Edition	9-63	9222 G
Chromogenic Substrate, MPN	Standard Methods, 19th Edition	9-65	9223 B
Colilert QT	Idexx Laboratories, Inc.		

If a single bacteria sample exceeds 406 organisms per 100 ml, then five consecutive re-samples shall be taken at four hour intervals beginning as soon as practicable (preferably within 28 hours) after the original sample was taken. If the log mean (geometric average) of the five re-samples is less than or equal to 126 organisms per 100 ml, a violation shall not be triggered.

2/ Composite samples to be representative of the final stabilized product prior to land application. Inorganic pollutant monitoring must be conducted according to **Test Methods for Evaluating Solid Waste, Physical/Chemical Methods**, Second Edition (1982) with Updates I and II and Third Edition (1986) with Revision I (EPA Publication SW-846).

4. Reporting Procedures:

a. Monitoring results shall be reported on approved forms.

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

c. Monitoring reports shall also include a record of all applicable breakdowns and bypassing. The quantity and method of use of all biosolids removed from the treatment facility and any biosolids management plan updates shall be included in the biosolids report submitted to the Department by February 19 of each year that describes solids handling activities for the previous year and includes but is not limited to, all monitoring data and the required information outlined in OAR 340-50-035(6)(a-e).

SCHEDULE C
Compliance Conditions and Schedules

1. No later than one year after issuance of this permit, the permittee shall submit a written plan for evaluating the dispersion, mixing and dilution of effluent at the outfall. The evaluation shall determine the ability of the discharge to comply with the water quality standards for total chlorine residual (no more than 0.019 mg/l within the mixing zone and no more than 0.011 mg/l at the edge of the mixing zone). Based on the results of the study, the Department will reopen the permit to include an appropriate total residual chlorine limit if necessary to achieve compliance with water quality standards. In lieu of the aforementioned, the permittee may propose and provide a schedule for removing chlorine from the discharge during the term of this permit.

2. The permittee shall have in place a program to identify and reduce inflow and infiltration into the sewage collection system. An annual report shall be submitted to the Department by February 1 each year detailing sewer collection maintenance activities that have been done in the previous calendar year and outlining those activities planned for the current year.

3. By no later than 12 months after permit issuance, the permittee shall submit either an engineering evaluation which demonstrates the design average wet weather flow, or a request to retain the existing mass load limits. The design average wet weather flow is defined as the average flow between November 1 and April 30 when the sewage treatment facility is projected to be at design capacity for that portion of the year. Upon acceptance by the Department of the design average wet weather flow determination, the permittee may request a permit modification to include higher winter mass loads based on the design average wet weather flow.

4. Within 180 days of permit modification to include higher winter mass load limits as specified in Condition 4 of this Schedule, the permittee shall submit to the Department for review and approval a proposed program and time schedule for identifying and reducing inflow. Within 60 days of receiving written Department comments, the permittee shall submit a final approvable program and time schedule. The program shall consist of the following:

   a. Identification of all overflow points and verification that sewer system overflows are not occurring up to a 24-hour, five-year storm event or equivalent;

   b. Monitoring of all pump station overflow points;

   c. A program for identifying and removing all inflow sources into the permittees sewer system over which the permittee has legal control; and

   d. If the permittee does not have the necessary legal authority for all portions of the sewer system or treatment facility, a program and schedule for gaining legal authority to require inflow reduction and a program and schedule for removing inflow sources.

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

SCHEDULE D
Special Conditions

1. All biosolids shall be managed in accordance with the current biosolids management plan approved by the Department and the site authorization letters issued by the Department. The biosolids management plan shall be kept current and remain on file with the permit. No substantial changes shall be made in solids management activities which significantly differ from operations specified under the approved plan without the prior written approval of the Department.

2. This permit may be modified to incorporate any applicable standard for biosolids use or disposal promulgated under section 405(d) of the Clean Water Act, if the standard for biosolids use or disposal is more stringent than any requirements for biosolids use or disposal in the permit, or controls a pollutant or practice not limited in this permit.

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

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

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

   b. The permittee's wastewater system may not be without supervision (as required by Special Condition 3.a. above) for more than thirty (30) days. During this period, and at any time that the supervisor is not available to respond on-site (i.e. vacation, sick leave or off-call), the permittee must make available another person who is certified at no less than one grade lower than the system classification.

   c. If the wastewater system has more than one daily shift, the permittee shall have the shift supervisor, if any, certified at no less than one grade lower than the system classification.

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

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

   f. Upon written request, the Department may grant the permittee reasonable time, not to exceed 120 days, to obtain the services of a qualified person to supervise the wastewater system. the written request must include justification for the time needed, a schedule for recruiting and hiring, the date the system supervisor availability ceased and the name of the alternate system supervisor(s) as required in 3.b. above.

4. The permittee shall notify the DEQ Northwest Region office (phone: 229-5263), in accordance with the response times noted in the General Conditions (Schedule F), of any malfunction so corrective action can be coordinated between the permittee and the Department.

SCHEDULE F
General Conditions

SECTION A. STANDARD CONDITIONS

1. Duty to Comply

The permittee must comply with all conditions of this permit. Any permit noncompliance constitutes a violation of Oregon Revised Statutes (ORS) 468B.025 and is grounds for enforcement action; for permit termination, suspension, or modification; or for denial of a permit renewal application.

2. Penalties for Water Pollution and Permit Condition Violations

Oregon Law (ORS 468.140) allows the Director to impose civil penalties up to $10,000 per day for violation of a term, condition, or requirement of a permit.

Under ORS 468.943, unlawful water pollution, if committed by a person with criminal negligence, is punishable by a fine of up to $25,000 or by imprisonment for not more than one year, or by both. Each day on which a violation occurs or continues is a separately punishable offense.

Under ORS 468.946, a person who knowingly discharges, places or causes to be placed any waste into the waters of the state or in a location where the waste is likely to escape into the waters of the state, is subject to a Class B felony punishable by a fine not to exceed $200,000 and up to 10 years in prison.

3. Duty to Mitigate

The permittee shall take all reasonable steps to minimize or prevent any discharge or sludge use or disposal in violation of this permit which has a reasonable likelihood of adversely affecting human health or the environment. In addition, upon request of the Department, the permittee shall correct any adverse impact on the environment or human health resulting from noncompliance with this permit, including such accelerated or additional monitoring as necessary to determine the nature and impact of the noncomplying discharge.

4. Duty to Reapply

If the permittee wishes to continue an activity regulated by this permit after the expiration date of this permit, the permittee must apply for and have the permit renewed. The application shall be submitted at least 180 days before the expiration date of this permit.

The Director may grant permission to submit an application less than 180 days in advance but no later than the permit expiration date.

5. Permit Actions

This permit may be modified, suspended, revoked and reissued, or terminated for cause including, but not limited to, the following:

a. Violation of any term, condition, or requirement of this permit, a rule, or a statute;

b. Obtaining this permit by misrepresentation or failure to disclose fully all material facts; or

c. A change in any condition that requires either a temporary or permanent reduction or elimination of the authorized discharge.

The filing of a request by the permittee for a permit modification or a notification of planned changes or anticipated noncompliance, does not stay any permit condition.

6. Toxic Pollutants

The permittee shall comply with any applicable effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants within the time provided in the regulations that establish those standards or prohibitions, even if the permit has not yet been modified to incorporate the requirement.

7. Property Rights

The issuance of this permit does not convey any property rights of any sort, or any exclusive privilege.

8. Permit References

Except for effluent standards or prohibitions established under Section 307(a) of the Clean Water Act for toxic pollutants and standards for sewage sludge use or disposal established under Section 405(d) of the Clean Water Act, all rules and statutes referred to in this permit are those in effect on the date this permit is issued.

SECTION B. OPERATION AND MAINTENANCE OF POLLUTION CONTROLS

1. Proper Operation and Maintenance

The permittee shall at all times properly operate and maintain all facilities and systems of treatment and control (and related appurtenances) which are installed or used by the permittee to achieve compliance with the conditions of this permit. Proper operation and maintenance also includes adequate laboratory controls, and appropriate quality assurance procedures. This provision requires the operation of back-up or auxiliary facilities or similar systems which are installed by a permittee only when the operation is necessary to achieve compliance with the conditions of the permit.

2. Duty to Halt or Reduce Activity

For industrial or commercial facilities, upon reduction, loss, or failure of the treatment facility, the permittee shall, to the extent necessary to maintain compliance with its permit, control production or all discharges or both until the facility is restored or an alternative method of treatment is provided. This requirement applies, for example, when the primary source of power of the treatment facility fails or is reduced or lost. It shall not be a defense for a permittee in an enforcement action that it would have been necessary to halt or reduce the permitted activity in order to maintain compliance with the conditions of this permit.

3. Bypass of Treatment Facilities

a. Definitions

(1) "Bypass" means intentional diversion of waste streams from any portion of the treatment facility. The term "bypass" does not include nonuse of singular or multiple units or processes of a treatment works when the nonuse is insignificant to the quality and/or quantity of the effluent produced by the treatment works. The term "bypass" does not apply if the diversion does not cause effluent limitations to be exceeded, provided the diversion is to allow essential maintenance to assure efficient operation.

(2) "Severe property damage" means substantial physical damage to property, damage to the treatment facilities or treatment processes which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of a bypass. Severe property damage does not mean economic loss caused by delays in production.

b. Prohibition of bypass.

(1) Bypass is prohibited unless:

(a) Bypass was necessary to prevent loss of life, personal injury, or severe property damage;

(b) There were no feasible alternatives to the bypass, such as the use of auxiliary treatment facilities, retention of untreated wastes, or maintenance during normal periods of equipment downtime. This condition is not satisfied if adequate backup equipment should have been installed in the exercise of reasonable engineering judgment to prevent a bypass which occurred during normal periods of equipment downtime or preventative maintenance; and

(c) The permittee submitted notices and requests as required under General Condition B.3.c.

(2) The Director may approve an anticipated bypass, after considering its adverse effects and any alternatives to bypassing, when the Director determines that it will meet the three conditions listed above in General Condition B.3.b.(1).

c. Notice and request for bypass.

(1) Anticipated bypass. If the permittee knows in advance of the need for a bypass, it shall submit prior written notice, if possible at least ten days before the date of the bypass.

(2) Unanticipated bypass. The permittee shall submit notice of an unanticipated bypass as required in General Condition D.5.

4. <u>Upset</u>

a. Definition. "Upset" means an exceptional incident in which there is unintentional and temporary noncompliance with technology based permit effluent limitations because of factors beyond the reasonable control of the permittee. An upset does not include noncompliance to the extent caused by operation error, improperly designed treatment facilities, inadequate treatment facilities, lack of preventative maintenance, or careless or improper operation.

b. Effect of an upset. An upset constitutes an affirmative defense to an action brought for noncompliance with such technology based permit effluent limitations if the requirements of General Condition B.4.c are met. No determination made during administrative review of claims that noncompliance was caused by upset, and before an action for noncompliance, is final administrative action subject to judicial review.

c. Conditions necessary for a demonstration of upset. A permittee who wishes to establish the affirmative defense of upset shall demonstrate, through properly signed, contemporaneous operating logs, or other relevant evidence that:

(1) An upset occurred and that the permittee can identify the causes(s) of the upset;

(2) The permitted facility was at the time being properly operated;

(3) The permittee submitted notice of the upset as required in General Condition D.5, hereof (24-hour notice); and

(4) The permittee complied with any remedial measures required under General Condition A.3 hereof.

d. Burden of proof. In any enforcement proceeding the permittee seeking to establish the occurrence of an upset has the burden of proof.

5. <u>Treatment of Single Operational Event</u>

For purposes of this permit, A Single Operational Event which leads to simultaneous violations of more than one pollutant parameter shall be treated as a single violation. A single operational event is an exceptional incident which causes simultaneous, unintentional, unknowing (not the result of a knowing act or omission), temporary noncompliance with more than one Clean Water Act effluent discharge pollutant parameter. A single operational event does not include Clean Water Act violations involving discharge without a NPDES permit or noncompliance to the extent caused by improperly designed or inadequate treatment facilities. Each day of a single operational event is a violation.

6. Overflows from Wastewater Conveyance Systems and Associated Pump Stations

a. Definitions

(1) "Overflow" means the diversion and discharge of waste streams from any portion of the wastewater conveyance system including pump stations, through a designed overflow device or structure, other than discharges to the wastewater treatment facility.

(2) "Severe property damage" means substantial physical damage to property, damage to the conveyance system or pump station which causes them to become inoperable, or substantial and permanent loss of natural resources which can reasonably be expected to occur in the absence of an overflow.

(3) "Uncontrolled overflow" means the diversion of waste streams other than through a designed overflow device or structure, for example to overflowing manholes or overflowing into residences, commercial establishments, or industries that may be connected to a conveyance system.

b. Prohibition of overflows. Overflows are prohibited unless:

(1) Overflows were unavoidable to prevent an uncontrolled overflow, loss of life, personal injury, or severe property damage;

(2) There were no feasible alternatives to the overflows, such as the use of auxiliary pumping or conveyance systems, or maximization of conveyance system storage; and

(3) The overflows are the result of an upset as defined in General Condition B.4. and meeting all requirements of this condition.

c. Uncontrolled overflows are prohibited where wastewater is likely to escape or be carried into the waters of the State by any means.

d. Reporting required. Unless otherwise specified in writing by the Department, all overflows and uncontrolled overflows must be reported orally to the Department within 24 hours from the time the permittee becomes aware of the overflow. Reporting procedures are described in more detail in General Condition D.5.

7. Public Notification of Effluent Violation or Overflow

If effluent limitations specified in this permit are exceeded or an overflow occurs, upon request by the Department, the permittee shall take such steps as are necessary to alert the public about the extent and nature of the discharge. Such steps may include, but are not limited to, posting of the river at access points and other places, news releases, and paid announcements on radio and television.

8. Removed Substances

Solids, sludges, filter backwash, or other pollutants removed in the course of treatment or control of wastewaters shall be disposed of in such a manner as to prevent any pollutant from such materials from entering public waters, causing nuisance conditions, or creating a public health hazard.

SECTION C. MONITORING AND RECORDS

1. Representative Sampling

Sampling and measurements taken as required herein shall be representative of the volume and nature of the monitored discharge. All samples shall be taken at the monitoring points specified in this permit and shall be taken, unless otherwise specified, before the effluent joins or is diluted by any other waste stream, body of water, or substance. Monitoring points shall not be changed without notification to and the approval of the Director.

2. Flow Measurements

Appropriate flow measurement devices and methods consistent with accepted scientific practices shall be selected and used to ensure the accuracy and reliability of measurements of the volume of monitored discharges. The devices shall be installed, calibrated and maintained to insure that the accuracy of the measurements is consistent with the accepted capability of that type of device. Devices selected shall be capable of measuring flows with a maximum deviation of less than ± 10 percent from true discharge rates throughout the range of expected discharge volumes.

3. Monitoring Procedures

Monitoring must be conducted according to test procedures approved under 40 CFR Part 136, unless other test procedures have been specified in this permit.

4. Penalties of Tampering

The Clean Water Act provides that any person who falsifies, tampers with, or knowingly renders inaccurate, any monitoring device or method required to be maintained under this permit shall, upon conviction, be punished by a fine of not more than $10,000 per violation, or by imprisonment for not more than two years, or by both. If a conviction of a person is for a violation committed after a first conviction of such person, punishment is a fine not more than $20,000 per day of violation, or by imprisonment of not more than four years or both.

5. Reporting of Monitoring Results

Monitoring results shall be summarized each month on a Discharge Monitoring Report form approved by the Department. The reports shall be submitted monthly and are to be mailed, delivered or otherwise transmitted by the 15th day of the following month unless specifically approved otherwise in Schedule B of this permit.

6. Additional Monitoring by the Permittee

If the permittee monitors any pollutant more frequently than required by this permit, using test procedures approved under 40 CFR 136 or as specified in this permit, the results of this monitoring shall be included in the calculation and reporting of the data submitted in the Discharge Monitoring Report. Such increased frequency shall also be indicated. For a pollutant parameter that may be sampled more than once per day (e.g., Total Chlorine Residual), only the average daily value shall be recorded unless otherwise specified in this permit.

7. Averaging of Measurements

Calculations for all limitations which require averaging of measurements shall utilize an arithmetic mean, except for bacteria which shall be averaged as specified in this permit.

8. Retention of Records

Except for records of monitoring information required by this permit related to the permittee's sewage sludge use and disposal activities, which shall be retained for a period of at least five years (or longer as required by 40 CFR part 503), the permittee shall retain records of all monitoring information, including all calibration and maintenance records of all original strip chart recordings for continuous monitoring instrumentation, copies of all reports required by this permit, and records of all data used to complete the application for this permit, for a period of at least 3 years from the date of the sample, measurement, report or application. This period may be extended by request of the Director at any time.

9. Records Contents

Records of monitoring information shall include:

a. The date, exact place, time and methods of sampling or measurements;

b. The individual(s) who performed the sampling or measurements;

c. The date(s) analyses were performed;

d. The individual(s) who performed the analyses;

e. The analytical techniques or methods used; and

f. The results of such analyses.

10. Inspection and Entry

The permittee shall allow the Director, or an authorized representative upon the presentation of credentials to:

a. Enter upon the permittee's premises where a regulated facility or activity is located or conducted, or where records must be kept under the conditions of this permit;

b. Have access to and copy, at reasonable times, any records that must be kept under the conditions of this permit;

c. Inspect at reasonable times any facilities, equipment (including monitoring and control equipment), practices, or operations regulated or required under this permit, and

d. Sample or monitor at reasonable times, for the purpose of assuring permit compliance or as otherwise authorized by state law, any substances or parameters at any location.

SECTION D. REPORTING REQUIREMENTS

1. Planned Changes

The permittee shall comply with Oregon Administrative Rules (OAR) 340, Division 52, "Review of Plans and Specifications". Except where exempted under OAR 340-52, no construction, installation, or modification involving disposal systems, treatment works, sewerage systems, or common sewers shall be commenced until the plans and specifications are submitted to and approved by the Department. The permittee shall give notice to the Department as soon as possible of any planned physical alternations or additions to the permitted facility.

2. Anticipated Noncompliance

The permittee shall give advance notice to the Director of any planned changes in the permitted facility or activity which may result in noncompliance with permit requirements.

3. Transfers

This permit may be transferred to a new permittee provided the transferee acquires a property interest in the permitted activity and agrees in writing to fully comply with all the terms and conditions of the permit and the rules of the Commission. No permit shall be transferred to a third party without prior written approval from the Director. The permittee shall notify the Department when a transfer of property interest takes place.

4. Compliance Schedule

Reports of compliance or noncompliance with, or any progress reports on interim and final requirements contained in any compliance schedule of this permit shall be submitted no later than 14 days following each schedule date. Any reports of noncompliance shall include the cause of noncompliance, any remedial actions taken, and the probability of meeting the next scheduled requirements.

5. Twenty-Four Hour Reporting

The permittee shall report any noncompliance which may endanger health or the environment. Any information shall be provided orally (by telephone) within 24 hours, unless otherwise specified in this permit, from the time the permittee becomes aware of the circumstances. During normal business hours, the Department's Regional office shall be called. Outside of normal business hours, the Department shall be contacted at 1-800-452-0311 (Oregon Emergency Response System).

A written submission shall also be provided within 5 days of the time the permittee becomes aware of the circumstances. If the permittee is establishing an affirmative defense of upset or bypass to any offense under ORS 468.922 to 468.946, and in which case if the original reporting notice was oral, delivered written notice must be made to the Department or other agency with regulatory jurisdiction within 4 (four) calendar days. The written submission shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected;

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance; and

e. Public notification steps taken, pursuant to General Condition B.7.

The following shall be included as information which must be reported within 24 hours under this paragraph:

a. Any unanticipated bypass which exceeds any effluent limitation in this permit.

b. Any upset which exceeds any effluent limitation in this permit.

c. Violation of maximum daily discharge limitation for any of the pollutants listed by the Director in this permit.

The Department may waive the written report on a case-by-case basis if the oral report has been received within 24 hours.

6. Other Noncompliance

The permittee shall report all instances of noncompliance not reported under General Condition D.4 or D.5, at the time monitoring reports are submitted. The reports shall contain:

a. A description of the noncompliance and its cause;

b. The period of noncompliance, including exact dates and times;

c. The estimated time noncompliance is expected to continue if it has not been corrected; and

d. Steps taken or planned to reduce, eliminate, and prevent reoccurrence of the noncompliance.

7. Duty to Provide Information

The permittee shall furnish to the Department, within a reasonable time, any information which the Department may request to determine compliance with this permit. The permittee shall also furnish to the Department, upon request, copies of records required to be kept by this permit.

Other Information: When the permittee becomes aware that it failed to submit any relevant facts in a permit application, or submitted incorrect information in a permit application or any report to the Department, it shall promptly submit such facts or information.

8. Signatory Requirements

All applications, reports or information submitted to the Department shall be signed and certified in accordance with 40 CFR 122.22.

9. Falsification of Reports

Under ORS 468.953, any person who knowingly makes any false statement, representation, or certification in any record or other document submitted or required to be maintained under this permit, including monitoring reports or reports of compliance or noncompliance, is subject to a Class C felony punishable by a fine not to exceed $100,000 per violation and up to 5 years in prison.

10. Changes to Indirect Dischargers - **[Applicable to Publicly Owned Treatment Works (POTW) only]**

The permittee must provide adequate notice to the Department of the following:

a. Any new introduction of pollutants into the POTW from an indirect discharger which would be subject to section 301 or 306 of the Clean Water Act if it were directly discharging those pollutants and;

b. Any substantial change in the volume or character of pollutants being introduced into the POTW by a source introducing pollutants into the POTW at the time of issuance of the permit.

c. For the purposes of this paragraph, adequate notice shall include information on (i) the quality and quantity of effluent introduced into the POTW, and (ii) any anticipated impact of the change on the quantity or quality of effluent to be discharged from the POTW.

11. Changes to Discharges of Toxic Pollutant - **[Applicable to existing manufacturing, commercial, mining, and silvicultural dischargers only]**

The permittee must notify the Department as soon as they know or have reason to believe of the following:

a. That any activity has occurred or will occur which would result in the discharge, on a routine or frequent basis, of any toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels:

(1) One hundred micrograms per liter (100 µg/l);

(2) Two hundred micrograms per liter (200 µg/l) for acrolein and acrylonitrile; five hundred micrograms per liter (500 µg/l) for 2,4-dinitrophenol and for 2-methyl-4,6-dinitrophenol; and one milligram per liter (1 mg/l) for antimony;

(3) Five (5) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

b. That any activity has occurred or will occur which would result in any discharge, on a non-routine or infrequent basis, of a toxic pollutant which is not limited in the permit, if that discharge will exceed the highest of the following "notification levels":

(1) Five hundred micrograms per liter (500 µg/l);

(2) One milligram per liter (1 mg/l) for antimony;

(3) Ten (10) times the maximum concentration value reported for that pollutant in the permit application in accordance with 40 CFR 122.21(g)(7); or

(4) The level established by the Department in accordance with 40 CFR 122.44(f).

SECTION E. DEFINITIONS

1. BOD means five-day biochemical oxygen demand.

2. TSS means total suspended solids.

3. mg/l means milligrams per liter.

4. kg means kilograms.

5. m^3/d means cubic meters per day.

6. MGD means million gallons per day.

7. Composite sample means a sample formed by collecting and mixing discrete samples taken periodically and based on time or flow.

8. FC means fecal coliform bacteria.

9. Technology based permit effluent limitations means technology-based treatment requirements as defined in 40 CFR 125.3, and concentration and mass load effluent limitations that are based on minimum design criteria specified in OAR 340-41.

10. CBOD means five day carbonaceous biochemical oxygen demand.

11. Grab sample means an individual discrete sample collected over a period of time not to exceed 15 minutes.

12. Quarter means January through March, April through June, July through September, or October through December.

13. Month means calendar month.

14. Week means a calendar week of Sunday through Saturday.

15. Total residual chlorine means combined chlorine forms plus free residual chlorine.

16. The term "bacteria" includes but is not limited to fecal coliform bacteria, total coliform bacteria, and E. coli bacteria.

17. POTW means a publicly owned treatment works.

APPENDIX II

MIXING ZONE STUDY

October 15, 1998

CURRAN-McLEOD, INC.
CONSULTING ENGINEERS
6655 S.W. HAMPTON STREET, SUITE 210
PORTLAND, OREGON 97223
PHONE (503) 684-3478
FAX (503) 624-8247

MEMORANDUM

TO: Bill Strawn, Public Works Superintendent

FROM: Patrick D. Curran, P.E.

RE: **ESTACADA WASTEWATER TREATMENT PLANT MIXING ZONE ANALYSIS**

Bill:

Based on data obtained by your plant operating staff on the 7th of October, I have generated calculations of mixing zone phenomena of that date and conditions. The numbers indicate that two possibilities exist to assure that chlorine residuals will be within state limits in the mixing zone:

1. Extend the pipeline to the most vigorous part of the main channel; or,

2. Install a dechlorination system at the measuring manhole near the effluent filter.

The calculations show that at 700 cfs in the Clackamas River, compliance chlorine residuals of 0.019 mg/L in the Zone of Initial Dilution and 0.011 mg/L in the mixing zone (at 75 feet) are not achievable without jeopardizing disinfection.

Methods

The general mixing zone equation was manipulated to present the ratio of flows as a measure of the conductivity readings. The conductivity readings are multiplied by ten except for the sewage conductivity which is multiplied by 100.

Equation: $C_R Q_R + C_S Q_S = C_T Q_T$ where:

$C_R =$ River Conductivity (background)
$Q_R =$ River flow at the sampling point
$C_S =$ Sewage conductivity
$Q_S =$ Sewage flow
$C_T =$ River + sewage conductivity at the sampling points
$Q_T =$ River flow plus sewage flow at the sampling points

Background river conductivity was assumed to be the average of the lowest data points.

6.08
6.02
6.06
6.02
25.18/4 average 6.04

$\underline{C_R = 6.04}$

Q_R is unknown at the sampling points and is indicated by a ratio of the conductivities at each sampling point.

C_S is the sewage conductivity, in this case, the average of three samples showing 4.08, 4.30 and 4.31. The average is 4.23.

$\underline{C_S = 423}$

The sewage flow on October 7th averaged 0.65 MGD for the day or approximately 1.0 cfs. The flow distribution at each sampling point is diffused and measurable only by the ratio of conductivities.

C_T = River plus plant flow conductivity measured at the sampling points.

Q_T = River flow and sewage flow at the sampling points.

The derivation of the dilution ratio results in a ratio as follows:

$$\text{dilution ratio} = \frac{Q_R}{Q_S} = \frac{C_T - C_S}{C_R - C_T}$$

The table following shows the results of the calculations.

Mixing Zone Pts.	70 feet from outfall			75 feet from outfall		
	reading	dilution	max Cl_2	reading	dilution	max Cl_2
#1 opposite pipe 1' deep	N/A	---	---	69.8	37.6	0.41
opposite pipe 6' deep	65.5	70.1	0.77	65.2	74.5	0.82
#2 Lt @ 10:00 1' deep	67.6	49.4	0.54	74.6	24.5	0.27
Lt @ 10:00 6' deep	60.8	905.5	9.96	64.7	83.3	0.92
#3 Rt. @ 2:00 1' deep	72.2	29.7	0.33	79.2	18.3	0.20
Rt. @ 2:00 6' deep	60.2	>1,000	---	69.4	39.3	0.43
#4 Rt. @ 1:00 1' deep	69.3	39.7	0.44	68.2	45.5	0.50
Rt. @ 1:00 6' deep	60.6	>1,000	---	64.5	87.4	0.96
#5 Lt. @ 11:00 1' deep	70.4	35.5	0.39	62.0	225.6	2.48
Lt. @ 11:00 6' deep	61.6	301.1	3.31	60.2	>1,000	---

Zone of Initial Dilution									
	reading	dilution	max Cl_2	reading	dilution	max Cl_2	reading	dilution	max Cl_2
#6 @ 7.5' 1' deep	108.5	6.5	0.12						
@ 7.5' 6' deep	74.6	24.5	0.47						
#7 @ 7.5' 1' deep	80.1	17.4	0.33	78.0	19.6	0.37	88.8	11.8	0.22
6' deep	66.9	54.8	1.04	64.4	89.6	1.7	106.0	6.9	0.13
At outfall 1' deep	86.6	12.8	0.24						
6' deep	171.5	2.3	0.04						

Results

The shaded data points indicate those points where normal chlorine residual (0.4 mg/L) would exceed toxicity limits in the mixing zone (0.019 mg/L at the ZID and 0.011 mg/L at 75 feet).

The sampling data clearly show that the effluent plume surfaces almost immediately at the point of discharge and remains close to or at the water surface. This is not surprising since the effluent is usually at a higher temperature than the river water and warm water rises. This situation might be less dramatic in August and September; however, even summer releases from Cazadero Dam are seldom from warm surface water and more from deeper, cooler water.

Mixing is largely due to dispersion and less to diffusion. This means that installation of a multi-port diffuser on the existing pipeline would not improve mixing conditions.

Effective mixing requires agitation and energy to physically interfere with natural buoyancy conditions. This may be accomplished by injection of wastewater effluent into the main flow channel of the river. Extension of the effluent pipeline might satisfy the mixing conditions. Clearly, if a large proportion of the river base flow was available for dilution, chlorine residual would not be a problem. However, mixing connotes an energy exchange and effluent pipe velocity may not be sufficient to physically inject the plume into the river channel. The velocity in the outfall pipe is normally low and may exceed 1.0 feet per second only in the winter. A reduced port diffuser in the main channel might convert some of the available effluent head to velocity head and improve mixing. However, head losses at the orifices might compromise the capacity of the outfall pipe. These variables can be physically determined by measuring the characteristics of the outfall pipe.

On the day the samples were taken, the Clackamas River waters released from Cazadero Dam totaled 700 cfs. Low flows in the summer often are less than 1,000 cfs, but are fully managed between releases from Cazadero Dam and re-regulating at River Mill Dam. Low flows at River Mill Dam show a seven-day historic low flow of 507 cfs in 1958. However, recent summer daily flows are seldom less than 800 cfs.

Conclusions

1. The NPDES permit along with state water quality standards call for disinfection of the treated effluent to an average daily bacterial level of 126/100 ml of *E. Coli.* Maximum day is 406/100 ml.

2. Chlorine disinfection of the treated effluent requires a residual averaging 0.4 mg/L to reliably achieve the state bacterial limits.

3. The effective chlorine dosage leaves an effluent residual exceeding the chronic and acute toxicity limits within the mixing zone under controlled river flows.

4. The existing conditions do not appear to demonstrate compliance with state limitations of residual chlorine in the mixing zone. As a result, the City of Estacada should plan to effect improvements to:

   a. Extend the outfall to the main channel and install a diffuser; or,

   b. Install a dechlorination system to chemically reduce the chlorine residual; or,

   c. Install ultraviolet irradiation disinfection.

Estimated Costs

Extend the Outfall

1.	Mobilization, bond, permits, insurance	$ 5,000
2.	Trench excavation, 200' @ $22.00	4,400
3.	Foundation, stabilization, 150 cy @ $15.00	2,250
4.	Install 21-inch pipe with anchors, 200' @ $100.00/foot	20,000
5.	21-inch diffuser manifold, L.S.	2,500
6.	Three 6-inch diffusers, 3 @ $500 each	1,500
7.	Overflow facility, L.S.	7,500
8.	Concrete structures/thrust control, L.S.	5,000
9.	Erosion protection/special backfill, L.S.	5,000
	Total Construction	$53,150

Dechlorination

1.	Mobilization, etc.	$ 2,500
2.	Furnish and install chem feed pumps/controls (2)	5,600
3.	Chemical crocks (2)	1,400
4.	Misc. piping, valves, fittings, tubing	2,100
5.	Structural revisions/concrete, L.S.	5,000
6.	Electrical power/wiring, L.S.	4,500
7.	Service/training, 2 days @ $500	1,000
	Total Construction	$22,100

Ultraviolet Irradiation Disinfection

1.	Equipment only	$200,000
2.	Installation	30,000
3.	Concrete Channels	65,000
4.	Excavation/backfill	15,000
5.	Piping	21,000
6.	Electrical	35,000
	Total Construction	$366,000

Cost Summary

1.	Outfall Extension/Diffuser	$53,150
	Construction contingency	5,000
	Engineering, permits, administration	16,000
	Total Project	$74,150
2.	Dechlorination	22,100
	Construction contingency	2,000
	Engineering, administration	9,000
	Total Project	$33,100
3.	Ultraviolet Irradiation Disinfection	$366,000
	Construction Contingency	30,000
	Engineering, administration	42,000
	Total Project	$428,000

Reliability

Outfall extension: dependent on effluent release and river energy for mixing; at low flows, the wastewater velocities for mixing may be insufficient to assure mixing energy.

Dechlorination: electrically powered pumps, controls and sensors; dose controllers/flow meter pacing/response sensors; all have redundancy except at maximum flows; power loss is offset by the engine-generator; control failures jeopardize performance and reliability.

Ultraviolet Irradiation: reliability level same as dechlorination for controllers, pacing and sensors; electrical demand may exceed the capacity of the existing engine-generator; ultraviolet system draws an estimated 50 kW electrical demand.

Recommendation

The *ultraviolet irradiation* option enjoys the advantage of eliminating chemical disinfection and leaving no residual effects. Some bacterial regrowth has been documented in the literature; however, pathogenicity of the regrowth has not been demonstrated. Ultraviolet irradiation may be a realistic option at the end of the 20-year plant life and in the context of the next upgrade of wastewater treatment. There is little merit at this time to expend the dollars for UV disinfection if a substantially less costly option is available.

Extension of the outfall and diffuser-enhancement of mixing has favorable appeal but the wide variation of summer/winter flows presents challenging engineering problems. Division of State Lands/Corp of Engineers permits might present costly constraints. And, the accumulation of silt along the floor of the impoundment demands a rigorous analysis of pipe foundation.

There is no assurance that the mixing will always achieve the levels of completion necessary to avoid chlorine toxicity. Fish populations may be at even greater risk by introducing chlorinated effluent into the main channel instead of the present practice of dispersing the effluent near the surface and over a greater area.

Dechlorination appears to have the most appeal. It is possible to step the levels of control to coincide with the plant flow variations and provide system redundancy at a reasonable cost. Mixing and reaction can be accomplished in a short time and the controls can be programmed to assure the elimination of effluent chlorine residual with flow variations.

We would recommend the installation of dechlorination technology to assure compliance with mixing zone residual chlorine in the Clackamas River.

Additional Considerations

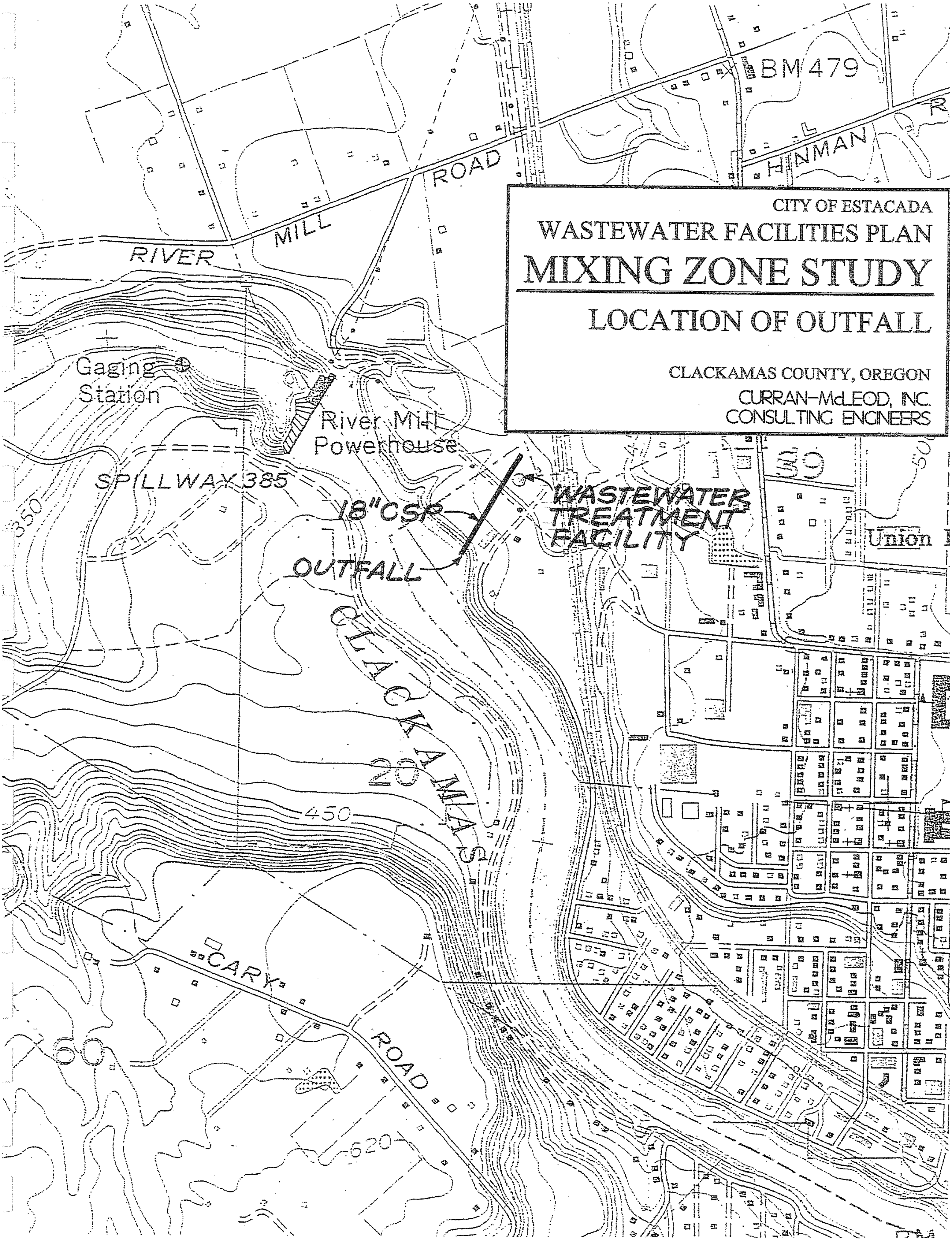
Two additional alternatives — long shot options — are also available. The first is to petition DEQ for an adjustment to the mixing zone. This may be possible if additional mixing zone sampling shows that the effluent effects a short distance further from the pipe are greatly diminished. It would be possible to make a strong case for extending the mixing zone to 100 feet or 125 feet if additional sampling supports this thesis.

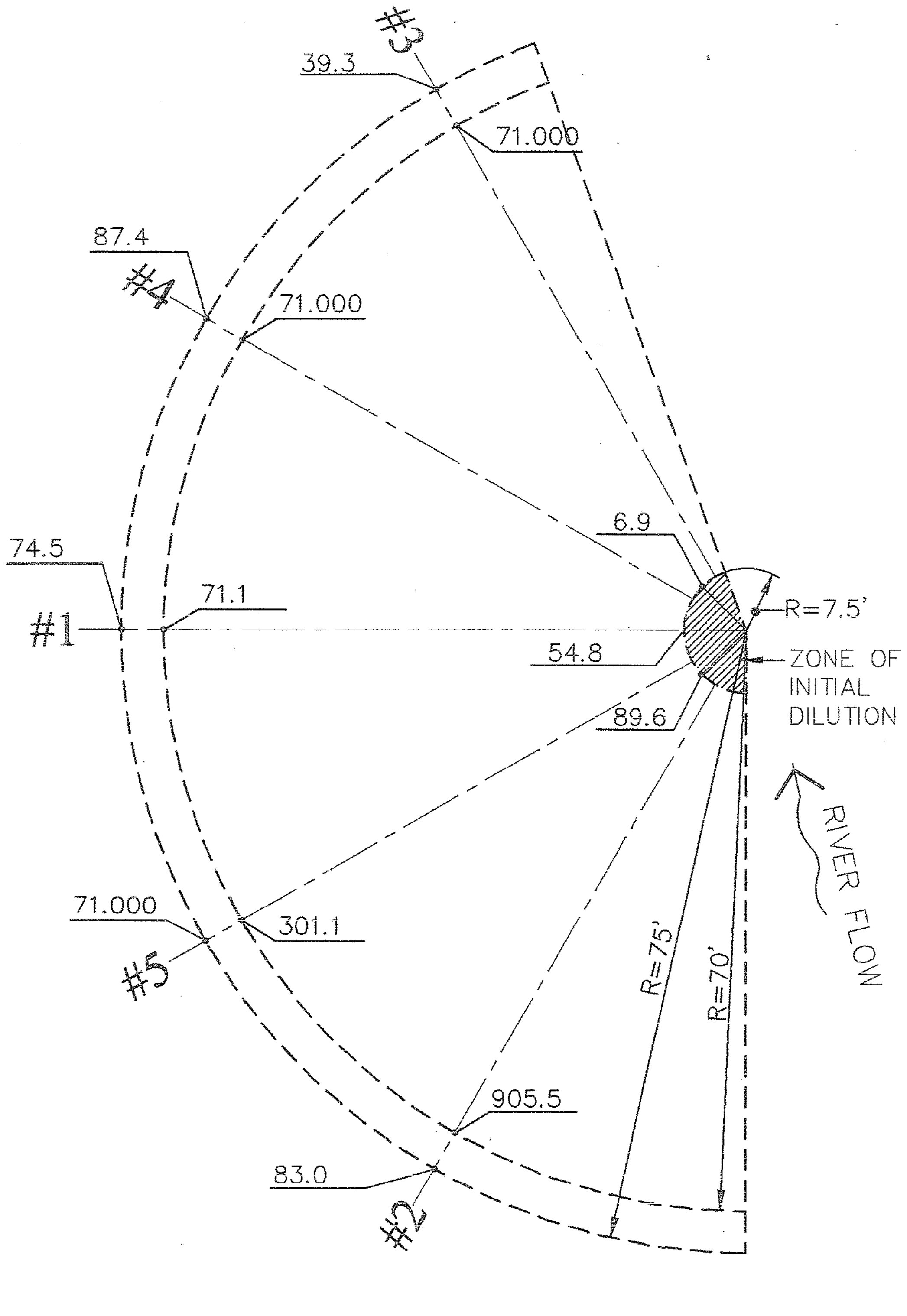
The chlorine residual in the ZID is still a problem but that might be overcome by inserting a flow diverter in the outfall pipe mouth.

The obvious benefits would be to assure permit compliance by changing the guidelines without serious or measurable environmental consequences. There are no rules, codes or statutes governing the establishment of mixing zone dimensions. Such a strategy is practicable and probably realistic. However, mixing zones are protected by DEQ with an almost religious zeal. Extending a mixing zone might find substantial staff resistance due to the potential for setting an unpopular precedent.

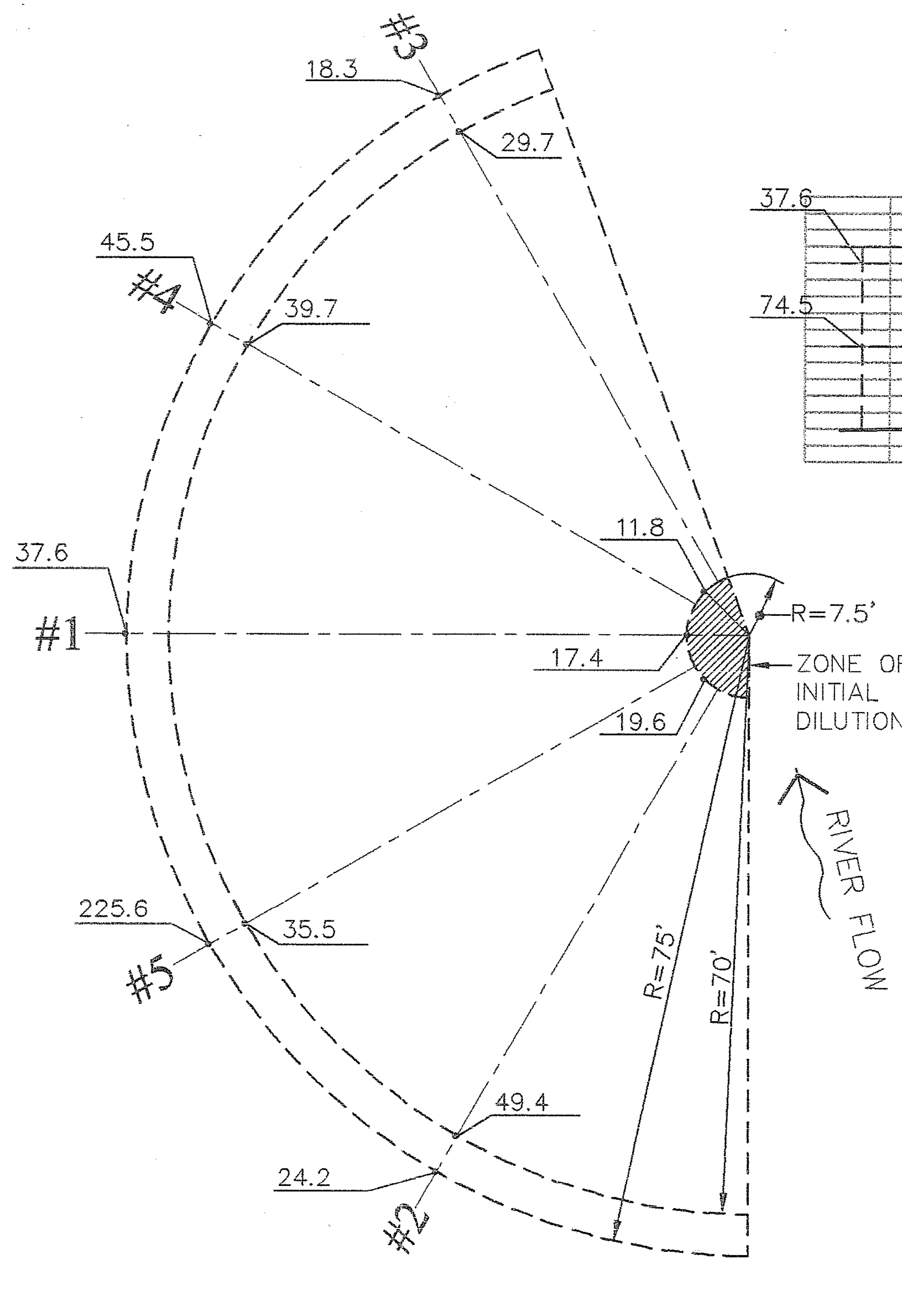
A second long-shot option would be to extend the outfall to the forebay of River Mill Dam to take advantage of violent agitation below the dam. There is no doubt that perfect mixing could be achieved. However, this same successful mixing might present a subchronic chlorine toxicity barrier to fish accessing the pool and adjacent reaches of the stream.

Pat

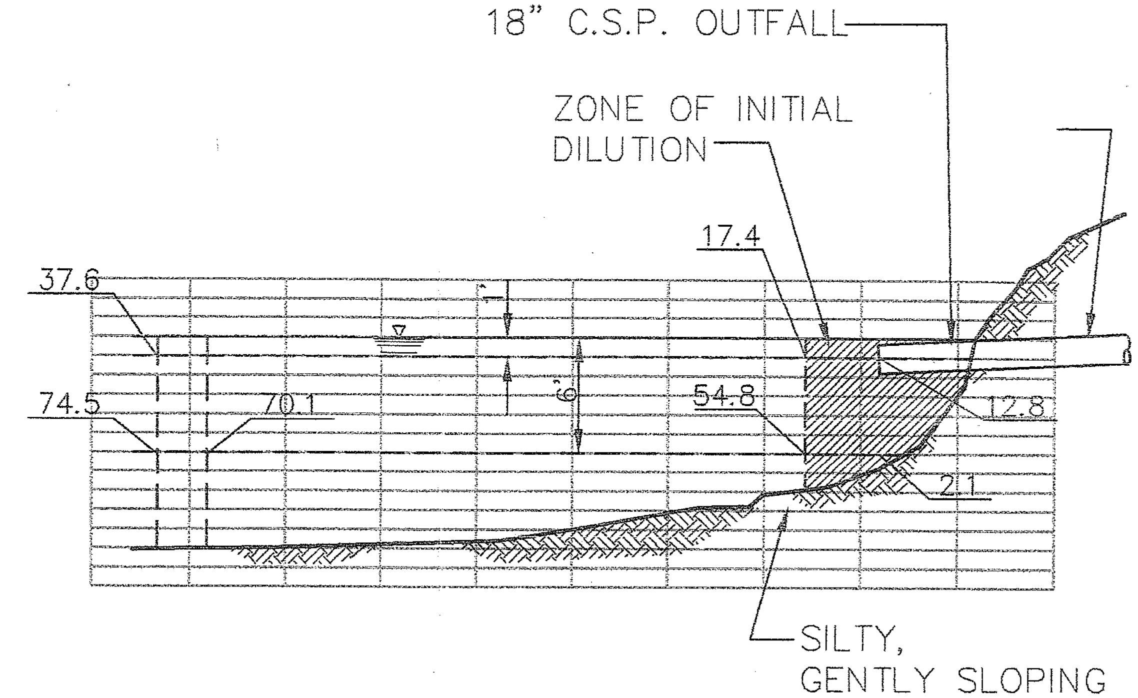

CITY OF ESTACADA
WASTEWATER FACILITIES PLAN
MIXING ZONE STUDY
LOCATION OF OUTFALL
CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS
BM 479
HINMAN
RIVER MILL ROAD
Gaging Station
River Mill Powerhouse
SPILLWAY 385
18"CSP
OUTFALL
WASTEWATER TREATMENT FACILITY
Union
CLACKAMAS
450
350
620
CARY ROAD



PLAN VIEW
at 6' DEEP



PLAN VIEW
at 1' DEEP



PROFILE

CITY OF ESTACADA
WASTEWATER FACILITIES PLAN

MIXING ZONE STUDY

SAMPLING POINTS AND CALCULATED DILUTION RATIO

CLACKAMAS COUNTY, OREGON
CURRAN-McLEOD, INC.
CONSULTING ENGINEERS

301DILUT 04/26/2000 12:59

APPENDIX III

PUMP STATIONS

SEWAGE PUMP STATION DESIGN DATA

Owner/Facility	City of Estacada — Lakeshore Drive Pump Station
Address/Location	Lakeshore Drive and Oak
Area Served	75 acres of single and multi family residential including 3 acres which are commercially zoned, 262 EDUs

<u>Pump Station</u>

Type:	Dry Pit , vertically mounted, torque-flow
Pump type:	Constant Speed, 1170 rpm ,recessed impeller, vortex flow.
Design Capacity	150 gpm at 30 ft. total dynamic head
Pump Hp (each)	5 HP
Level Control Type	Bubler tube
Overflow	415 ft.
Overflow Discharge	Upstream manhole
Average time to Overflow	At 100 gpm, 30 minutes
Auxillary Power	none
Alarm Telemetry type	Autodialer to WTP

<u>Force Main</u>

Length, Type	100 ft. 6-inch ductile iron
Profile	Continuously ascending at aproximately 7% slope
Discharge manhole	#96,18 inch interceptor on Hwy 224
Air Release	none
Vacuum Release	none
Average detention	2.7 minutes at 100 gpm,

A Division of Western Machinery Company

CABLE ADDRESS "WEMACHY"   EXbrook 7-1717 • 650 FIFTH STREET • SAN FRANCISCO

DATE RECEIVED	DATE WANTED		
5-9-63	wk of 6-17-63	22297	9633

SOLD TO Zimmer & Francescon

ADDRESS 1715 Fifteenth St. Place, Moline, Illinois

SHIP TO Same

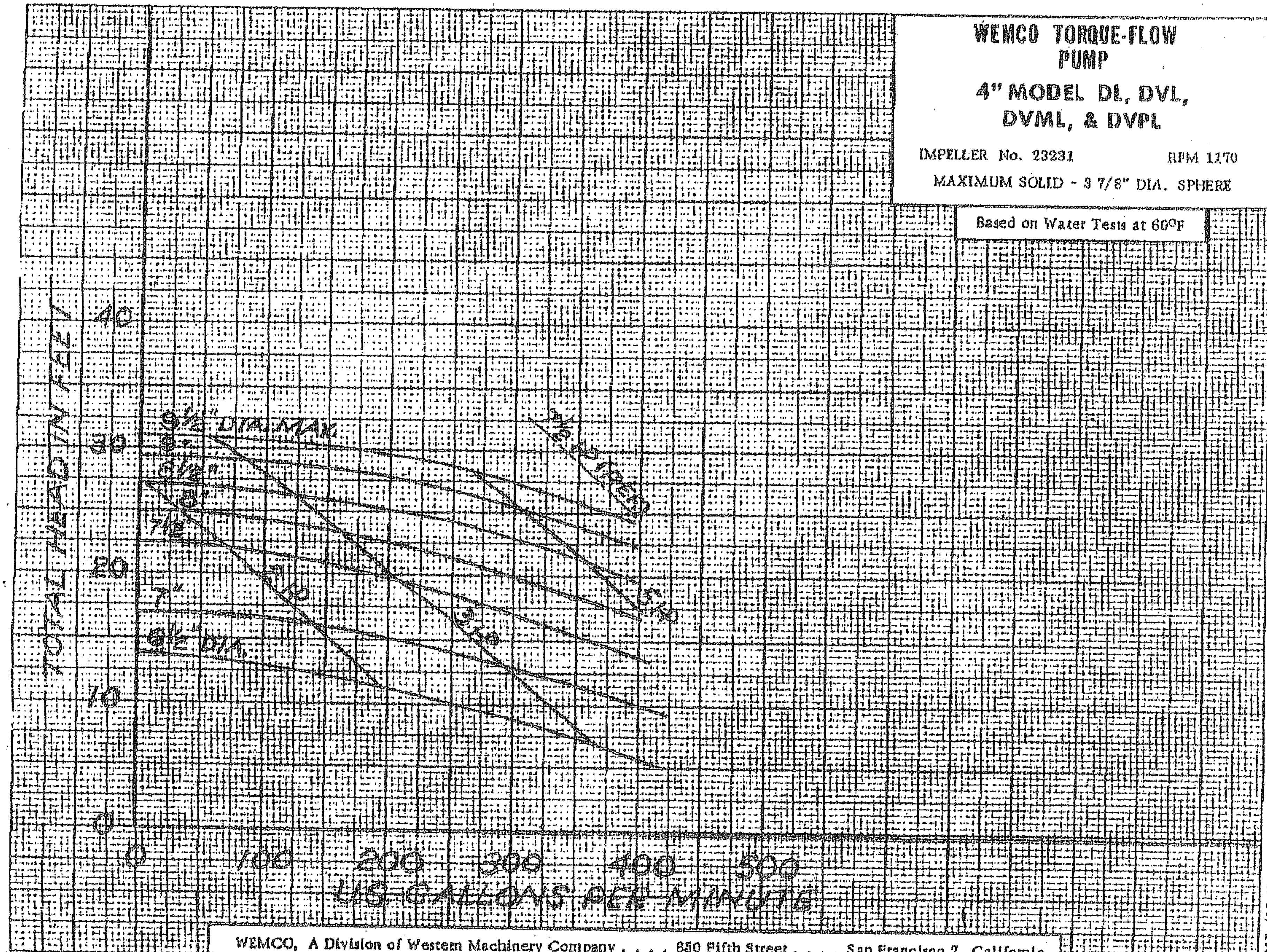
c/o Gabco Boiler & Equipment Company

ADDRESS 1400 N.W. 14th Avenue, Portland 9, Oregon

ITEM	QUANTITY	DESCRIPTION
1	2	4" x 4" MODEL DVML TORQUE FLOW PUMPS, S/N 6392297-1 & 2 PUMP S/N 1: VARIABLE ASSEMBLY #15734ARG1 CW Rotation PUMP S/N 2: VARIABLE ASSEMBLY #15734ALG1 CCW Rotation SEAL: Grease lubricated packing - Spring loaded grease cup CONSTRUCTION: Cast Iron PUMP S/N 1: SUCTION & DISCHARGE: Per Dwg. 42328 Arrgt.A PUMP S/N 2: SUCTION & DISCHARGE: Per Dwg. 42328 Opposite of Arrgt. A CONDITIONS: 150 GPM of sewage against 30 feet TDH at 1170 RPM IMPELLER: 23231 trimmed to 9 1/2".
2	2	VERTICAL SOLID SHAFT MOTORS 5 HP, 1170 RPM, Fr. 254UP, open dripproof, 3/60/220 volts, special shaft extension per WEMCO dwg. #44219 one right hand threaded shaft, one left hand threaded shaft. MARK: "P 471"

Released for Production

Prepaid Freight on Above

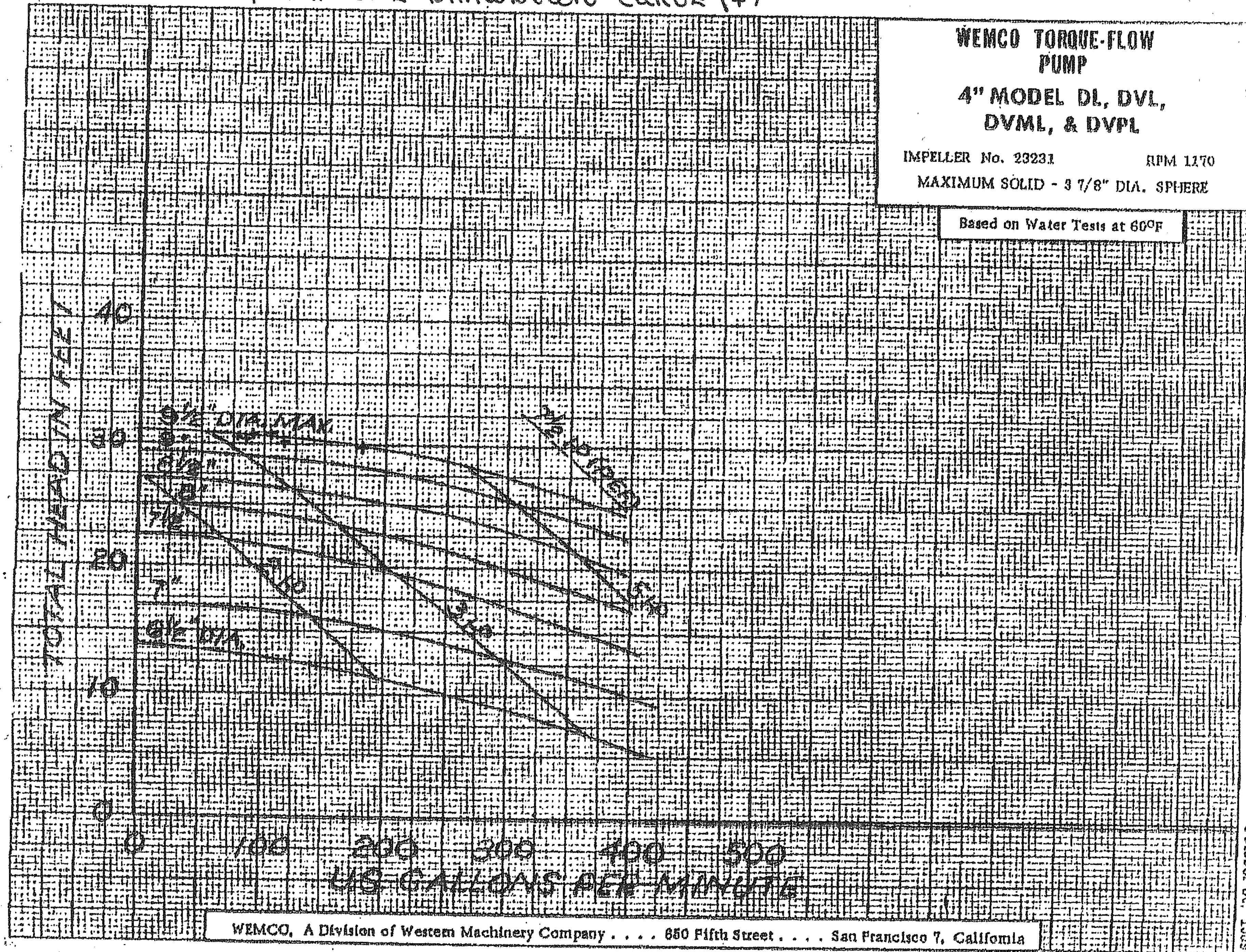


Supersedes P10-D114 dated 2/15/59

P10D-D61
October 30, 1959

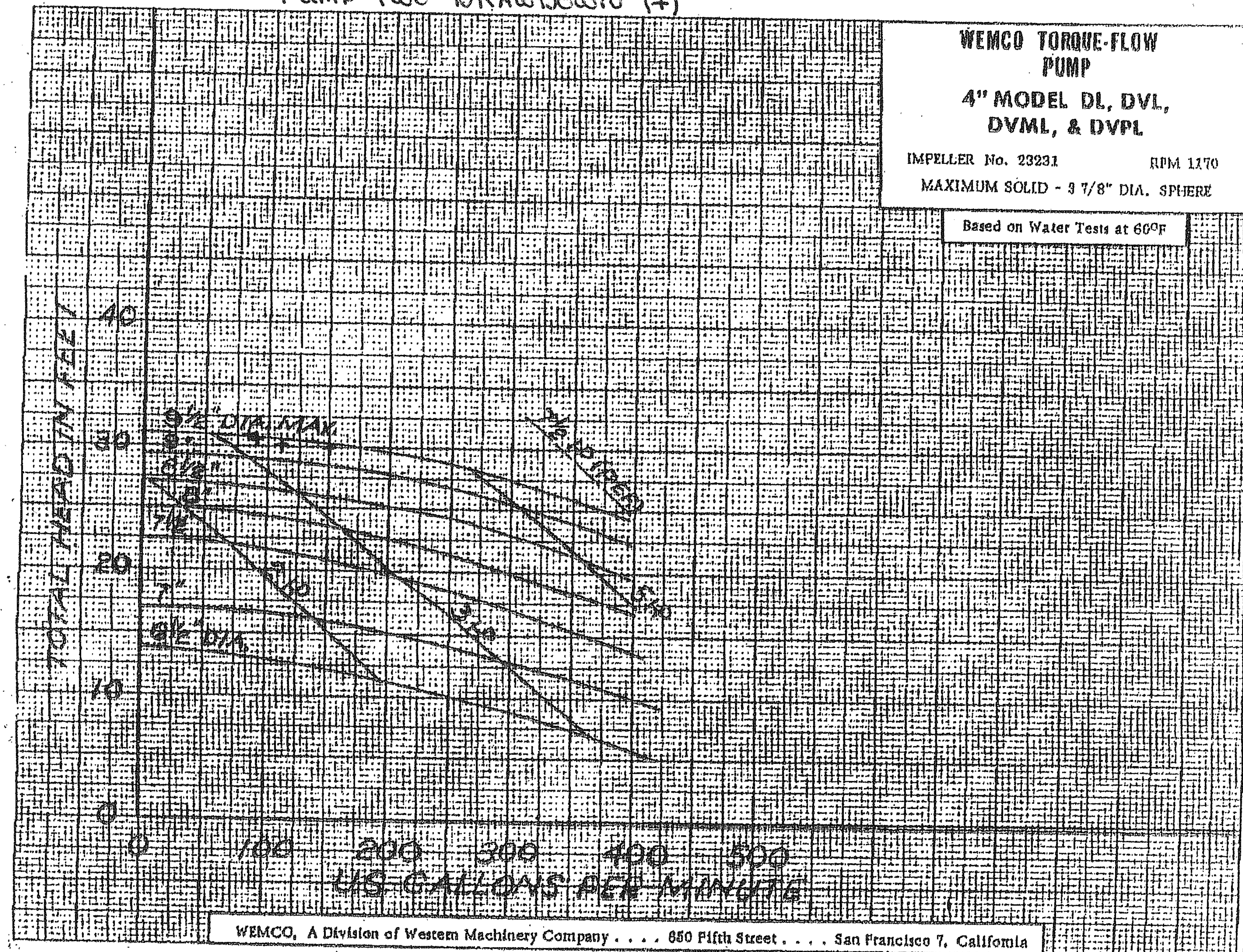
LAKESHORE DRIVE PUMP STATION						
DRAW DOWNS MAY-2000						
DESCRIPTION				FILL RATE		
LEVEL			ELEVATION	211G/FT	MIN	GPM
6 INCH FORCE MAI		INVERT	435.5	211	2.75	77
WET WELL		RIM	415	211	3.25	65
START			406	211	3.5	60
STOP			404	211	3.05	69
CENTER-LINE OF INTAKE			401.5			68
6' DIAMETER			TO 403.5			
PUMP #1						
START	STOP	DRAWD		DRAWD	P. RATE	TDH
		FT	MINUTES	GPM	GPM	FT
405	404.83	0.17	2	17.935	85.66902	30.585
404.5	404.2	0.3	2	31.65	99.38402	31.15
405	404	1	4.83	43.6853	111.4193	31
406	405.1	0.9	1.73	109.7688	177.5028	29.95
405.75	405	0.75	3.25	48.69231	116.4263	30.125
405	404.75	0.25	4.83	10.92133	78.65534	30.625
AVERAGES		0.561667	3.11	43.77545	111.5095	30.5725
PUMP #2		DRAWD		DRAWD	+FILL	TDH
AVE		FT	MINUTES	GPM	GPM	FT
405.5		0.69	1.71	86	153.734	30
405		0.41	4.13	21	88.73402	30.5
405.5		0.45	2.27	42	109.734	30
404.5		0.33	4	17.5	85.23402	31
AVERAGES			3.03	41.63	109.36	30.38

PUMP ONE DRAWDOWN CURVE (+)



FROM : CITY OF ESTACADA PHONE NO. : 5036302670 Apr. 25 2006 05:03PM P3

PUMP TWO DRAWDOWN (+)



SEWAGE PUMP STATION PLAN-REVIEW SUBMITTAL
State of Oregon / Department of Environmental Quality

Owner/Facility:	**City of Estacada, Oregon --- Timber Park Pump Station**
Address/Location:	**Timber Park** (West of Highway 224 and South of River Mill Road)
Area Served:	**Up to 300 acres of Light Industrial Development**
Pump Station Type:	**Wet-well, side-mounted, submersible, non-clog, duplex, lead-lag**

Design Summary: A duplex sewage pump station is needed to serve up to 300 acres of Light Industrial Development, with each pump handling a peak flow of approximately 1,200 gpm (in the future). The shorter-term service area is planned at 126 acres, so pumps will be initially installed with a capacity of 550 gpm each. Larger pumps can be installed in the future, which is expected to be ten years or more. The wetwell is sized to handle the design inflow for the future 300 acres.

The proposed pump station will use an 8-inch force main discharging at 3.5 fps minimum velocity directly into the city's wastewater treatment plant. This pipe size can effectively handle the minimum initial pumping rate of 550 gpm as well as a future rate of 1,200 gpm, or even more.

1. SEWAGE PUMP STATION DESIGN CALCULATIONS AND DOCUMENTS

System Sizing and Design Basis Calculations

The pump station has been designed to serve 126 and up to 300 acres of Light Industrial Development. Sewage flow calculations, shown on Tables LG-1 and LG-2, have been estimated using a range of both Low and High values for the following conditions: Low Minimum, High Minimum, Low Average, High Average, Low Peak (no I/I), High Peak (no I/I), Low Peak (with I/I), and High Peak (with I/I).

The pump station is designed for light industrial businesses which generate 1,000 to 1,500 gal/acre/day (gpapd), or an average of 1,250 gpapd. A peaking factor of 3.0 is then applied, and an I/I range of 1,000 to 3,000 gpapd (average of 2,000 gpapd) is added. Using these values, inflows are estimated to peak at about 1,200 gpm (range: 833 gpm to 1,563 gpm) at full buildout of 300 acres. Inflows at start-up can be expected to be fairly low during early morning hours when businesses are closed. Inflows for the 126-acre service area are expected to be 110 gpm (average) and 500 gpm (peak).

The wetwell will be sized with capacity to accommodate the inflow for the future 300 acres. When inflows increase beyond the initial service area of 126 acres (i.e. 550 gpm pump capacity each), larger pumps can be installed to provide double or even triple the flow capacity.

Hydraulics / Headloss Calcs / Pump Curve / System Head Curves

Force main and headloss calculations are shown on Table LG-3. A pumping velocity of 3.5 feet per second (fps) is used; this results in the selection of an 8-inch diameter force main. C-factors of C = 150 (new, smooth pipe) and C = 100 (old, rough pipe), have been used to estimate friction losses, and resulting Total Dynamic Head (TDH) values used to select the pump. The piping in the pumping station is expected to be 6-inch diameter. Hence, the overall TDH is the sum of the 6" and 8" values in the table.

Figure LG-1 depicts the two system curves (C = 100 & 150) plotted against the pump curves for a 4" Aurora/Hydromatic non-clog centrifugal pump (Model S4B/S4BX) operating at 1150 rpm. For this pump, a 10" diameter (or slightly smaller) impeller will pump the design flow. A pump motor of 7.5 Hp would be needed to preclude overloading in low-head or zero-head start-up conditions. A somewhat larger impeller of up to 10.5" diameter could also be used with the 7.5 Hp motor, without overloading. An alternate pump (Model S6A/S6AX) could be used if a 6" pump is desired, however this would require a 10 Hp motor to preclude overloading in low-head or zero-head start-up conditions.

Pump Starts Per Hour (@50% Pump Capacity)

Wetwell sizing calculations are shown on Tables LG-4 and LG-5. Though the station will use two pumps alternating on a lead-lag scenario, it is assumed only one pump may be operating at times, with the other out-of-service for replacement or maintenance. In this case, the one pump would cycle at a maximum of 9 starts per hour (@ inflow of 50% of the pump capacity). During normal operating conditions with two pumps operating, each pump would cycle at a maximum of 5 starts per hour. (According to the manufacturer, the proposed pump can cycle at up to 11 starts per hour.) During average flow conditions, one pump would cycle at 6 starts per hour; two pumps at 3 starts per hour each.

Wetwell Buoyancy Calculations

Wetwell buoyancy calculations are shown on Table LG-6. The Timber Park pump station will be a 96 - inch diameter wetwell, and it is 13.0 feet deep (ground elevation is at about 420.0 and the bottom of the wetwell is at 407.00). Conservatively, assume the wetwell is 14 feet deep, the water table is at ground level, the wetwell is empty, and exclude the weight of piping, pumps and equipment. The weight of the concrete is 74,150 pounds (including top and base slabs). The buoyant force is 70,175 pounds, which results in a 1.06 safety factor. Including the weight of the soil directly above the overhang on the base slab (an additional 36,279 pounds), the safety factor is 1.52. Including skin friction imposed by the granular backfill, against the side walls of the wetwell, the safety factor would be significantly greater. The wetwell (with base slab overhang) is of adequate weight to resist buoyancy.

Uncommon Equipment Design

The pump station has no uncommon equipment.

Plan and Profile of Force Main (including details)

The plan and profile drawings for the force main are included separately. Plan, section and detail drawings are also included for the pump station.

Wetwell and Force Main Detention Time Calculations

Detention times for the wetwell and force main vary, depending upon flow rates. Tables LG-4 and LG-5 list the respective detention times for various flows. The left column, under each service area scenario, lists detention times for the wetwell, assuming no provisions for force main backdrainage. The right column, under each scenario, lists detention times for the force main.

For average inflow (110 gpm), the detention time in the force main is about 38 minutes, and the detention time in the wetwell is about 8 minutes. For peak inflow (500 gpm), the detention times in both the wetwell and the force main are less than 10 minutes.

Sewage Overflow when Station or Power Fails

If the pump station should overflow, due to unforeseen mechanical or electrical failure, there is some potential for human or household pet contact since it is located in a city park. There are no adjacent creeks or drainageways, nor any nearby wells that could be contaminated. The pump station will have portable standby power, and will have two pumps installed, each with the capacity to handle the peak design inflow, should one pump be out of service at any time.

Standby Power and Alarm Considerations

Standby power will be provided via a new, mobile, trailer-mounted generator, located at the Timber Park pump station. Alarms will be transmitted directly to the City of Estacada's Wastewater Treatment Plant (WWTP), via a multiconductor cable buried during construction of the 8-inch force main to the WWTP.

Water Service at Pump Station

Water service will be provided at the pump station to maintain the wetwell and to clean equipment. New waterlines are being constructed as part of the overall project.

2. PUMP STATION DESIGN DATA

A design data sheet for the pump station, which follows the format provided by DEQ, is enclosed.

3. SCHEDULE OF ALARM ELEVATIONS AND CONDITIONS

A schedule of alarm elevations and conditions is shown on the drawings.

4. SITE PLAN AND ELECTRICAL/CONTROL DRAWINGS

Site plan and electrical/control drawings are provided.

5. CAPACITY OF DOWNSTREAM SEWERS

The pump station will pump directly to the city's wastewater treatment plant.

6. EXISTING PUMP STATION PROVISIONS

This is a new pump station, so there are no existing pump station provisions to address.

7. PROJECT INSPECTION PROVISIONS

The project will be inspected in accordance with OAR 340-52-015(1)(e).

8. O&M MANUAL PROVISIONS

Operation and maintenance requirements and manuals will be provided by the pumping system supplier.

9. CITY'S REVIEW AND APPROVAL OF PLANS

The proposed pump station has been developed in direct coordination with the City of Estacada, to meet their design, operational, and maintenance requirements.

10. TECHNICAL ACTIVITIES FEE

Please directly invoice the City of Estacada for the technical activities fee.

SEWAGE PUMP STATION DESIGN DATA

Owner/Facility:	**City of Estacada, Oregon --- Timber Park Pump Station**
Address/Location:	**Timber Park** (West of Highway 224 and South of River Mill Road)
Area Served:	**Up to 300 acres of Light Industrial Development**

PUMP STATION

Type:	Wet-well submersible, side-mounted, duplex, lead-lag
Pump Type:	Constant speed, 1150 rpm, non-clog
Capacity:	550 gpm at 32 ft Total Dynamic Head
Pump Hp (each):	7.5 Hp chk electr
Level Control Type:	Ultrasonic level transmitter; PLC-controlled; backed up by intrinsically safe mercury level sensors
Overflow Point:	Elev. 420.0 (approx.)
Overflow Discharge:	To adjacent ground around station
Avg. Time to Overflow:	45 minutes at 110 gpm design average Q
Auxiliary Power Type:	Portable diesel generator
Location:	On-site at the Timber Park P.S.
Output:	35 KW (240 volt, 3-phase)
Fuel Tank Capacity:	24 hours
Transfer Switch:	Manual; mounted at pump station
Alarm Telemetry Type:	Direct burial multiconductor cable to City's WWTP
EPA Reliability Class:	I

FORCE MAIN

Length, Type:	1,500 feet of 8-inch PVC
Profile:	Continuously ascending at approx. 0.5% to 1% slope
Discharge Manhole:	Directly into City's wastewater treatment plant
Air Release Valves:	None (no localized high points) plans show A/V release?
Vacuum Release Valves:	None
Average Detention:	150 min at 26 gpm (startup); 15 min at 260 gpm (average ultimate)
Sulfide Control System:	None

Land Use	Sewage (gal/acre/day) Low Avg.	High Avg.
Commercial	800	1,500
Light Industrial	1,000	1,500
Medium Industrial	1,500	3,000

Flow Case	Factor (for Minimum and Peak)	
Minimum	30%	[range: 30% to 50%]
Peak (Hrly)	300%	[< 2 mgd Avg. Flow]
Peak (Hrly)	150%	[> 2 mgd Avg. Flow]

Infiltration / Inflow (gal/acre/day)	
I/I - Low	1,000
I/I - High	3,000

126	<- Site (acreage)	Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow [incl I/I] (gal/day)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	100,800	189,000	30,240	56,700	302,400	567,000	428,400	945,000
P	Light Industrial	126,000	189,000	37,800	56,700	378,000	567,000	504,000	945,000
D	Medium Industrial	189,000	378,000	56,700	113,400	567,000	1,134,000	693,000	1,512,000

172	<- Site (acreage)	Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow [incl I/I] (gal/day)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	137,600	258,000	41,280	77,400	412,800	774,000	584,800	1,290,000
P	Light Industrial	172,000	258,000	51,600	77,400	516,000	774,000	688,000	1,290,000
D	Medium Industrial	258,000	516,000	77,400	154,800	774,000	1,548,000	946,000	2,064,000

126	<- Site (acreage)	Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow [incl I/I] (gal/hr)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	4,200	7,875	1,260	2,363	12,600	23,625	17,850	39,375
P	Light Industrial	5,250	7,875	1,575	2,363	15,750	23,625	21,000	39,375
H	Medium Industrial	7,875	15,750	2,363	4,725	23,625	47,250	28,875	63,000

172	<- Site (acreage)	Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow [incl I/I] (gal/hr)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	5,733	10,750	1,720	3,225	17,200	32,250	24,367	53,750
P	Light Industrial	7,167	10,750	2,150	3,225	21,500	32,250	28,667	53,750
H	Medium Industrial	10,750	21,500	3,225	6,450	32,250	64,500	39,417	86,000

126	<- Site (acreage)	Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow [incl I/I] (gal/min)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	70	131	21	39	210	394	298	656
P	Light Industrial	88	131	26	39	263	394	350	656
M	Medium Industrial	131	263	39	79	394	788	481	1,050

172	<- Site (acreage)	Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow [incl I/I] (gal/min)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	96	179	29	54	287	538	406	896
P	Light Industrial	119	179	36	54	358	538	478	896
M	Medium Industrial	179	358	54	108	538	1,075	657	1,433

Table LG-1

Land Use	Sewage (gal/acre/day) Low Avg.	High Avg.
Commercial	800	1,500
Light Industrial	1,000	1,500
Medium Industrial	1,500	3,000

Flow Case	Factor (for Minimum and Peak)	
Minimum	30%	[range: 30% to 50%]
Peak (Hrly)	300%	[< 2 mgd Avg. Flow]
Peak (Hrly)	150%	[> 2 mgd Avg. Flow]

Infiltration / Inflow	(gal/acre/day)
I/I - Low	1,000
I/I - High	3,000

210 <- Site (acreage)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow [incl I/I] (gal/day)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	168,000	315,000	50,400	94,500	504,000	945,000	714,000	1,575,000
P	Light Industrial	210,000	315,000	63,000	94,500	630,000	945,000	840,000	1,575,000
D	Medium Industrial	315,000	630,000	94,500	189,000	945,000	1,890,000	1,155,000	2,520,000

300 <- Site (acreage)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow (gal/day)		Sewage Flow [incl I/I] (gal/day)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	240,000	450,000	72,000	135,000	720,000	1,350,000	1,020,000	2,250,000
P	Light Industrial	300,000	450,000	90,000	135,000	900,000	1,350,000	1,200,000	2,250,000
D	Medium Industrial	450,000	900,000	135,000	270,000	1,350,000	2,700,000	1,650,000	3,600,000

210 <- Site (acreage)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow [incl I/I] (gal/hr)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	7,000	13,125	2,100	3,938	21,000	39,375	29,750	65,625
P	Light Industrial	8,750	13,125	2,625	3,938	26,250	39,375	35,000	65,625
H	Medium Industrial	13,125	26,250	3,938	7,875	39,375	78,750	48,125	105,000

300 <- Site (acreage)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow (gal/hour)		Sewage Flow [incl I/I] (gal/hr)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	10,000	18,750	3,000	5,625	30,000	56,250	42,500	93,750
P	Light Industrial	12,500	18,750	3,750	5,625	37,500	56,250	50,000	93,750
H	Medium Industrial	18,750	37,500	5,625	11,250	56,250	112,500	68,750	150,000

210 <- Site (acreage)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow [incl I/I] (gal/min)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	117	219	35	66	350	656	496	1,094
P	Light Industrial	146	219	44	66	438	656	583	1,094
M	Medium Industrial	219	438	66	131	656	1,313	802	1,750

300 <- Site (acreage)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow (gal/min)		Sewage Flow [incl I/I] (gal/min)	
	Land Use	Low Avg.	High Avg.	Low Min.	High Min.	Low Peak	High Peak	Low Peak	High Peak
G	Commercial	167	313	50	94	500	938	708	1,563
P	Light Industrial	208	313	63	94	625	938	833	1,563
M	Medium Industrial	313	625	94	188	938	1,875	1,146	2,500

Table LG-2

6" PIPE AND FITTINGS IN PUMP STATION?

--------> **OK, headloss is fair; use 6" pipe**

Pipe D (in) =	6
GPM @ 3.5fps =	308
Set Flow Increment (gpm) --->	100
Pipe Vol. (gal) =	37

6" Fittings	Eq. Length (ft)	No. Fittings
45-bend	8	2
90-bend	16	5
side tee	33	1
gate valve	4	2
BF valve		
swing chk	40	1
other?		

Total Equiv. Length (ft): 202

Length (ft): 25

Static (ft) = 0

Flow (gpm)	Flow (cfs)	Velocity (fps)	V*V/2g (ft)	Friction Headloss (ft/ft) C = 100	Friction Headloss (ft/ft) C = 150	Friction Headloss (feet) C = 100	Friction Headloss (feet) C = 150	Total Dynamic Head (ft) C = 100	Total Dynamic Head (ft) C = 150
0	0.00000	0.00000	0.00000	0.00000	0.00000	0	0	0	0
100	0.22282	1.13479	0.02000	0.00171	0.00081	0	0	0	0
200	0.44563	2.26959	0.07999	0.00615	0.00291	1	1	1	1
300	0.66845	3.40438	0.17997	0.01303	0.00615	3	1	3	1
400	0.89127	4.53918	0.31994	0.02219	0.01048	4	2	5	2
500	1.11408	5.67397	0.49991	0.03353	0.01584	7	3	7	4
600	1.33690	6.80877	0.71987	0.04698	0.02219	9	4	10	5
700	1.55971	7.94356	0.97982	0.06248	0.02951	13	6	14	7
800	1.78253	9.07836	1.27976	0.07999	0.03778	16	8	17	9
900	2.00535	10.21315	1.61970	0.09946	0.04698	20	9	22	11
1,000	2.22816	11.34795	1.99963	0.12087	0.05709	24	12	26	14
1,100	2.45098	12.48274	2.41955	0.14417	0.06809	29	14	32	16
1,200	2.67380	13.61754	2.87946	0.16935	0.07999	34	16	37	19

Note: > add 6" & 8" for TDH <

Table LG-3

Pipe D (in) =	8
GPM @ 3.5fps =	548
Set Flow Increment (gpm) --->	100
Pipe Vol. (gal) =	3,917

8" Fittings	Eq. Length (ft)	No. Fittings
45-bend	10	1
90-bend	21	3
side tee	43	
gate valve	5	
BF valve		
swing chk	53	
other?		

Total Equiv. Length (ft): 1,573

Length (ft): 1,500

Static (ft) = 19

Flow (gpm)	Flow (cfs)	Velocity (fps)	V*V/2g (ft)	Friction Headloss (ft/ft) C = 100	Friction Headloss (ft/ft) C = 150	Friction Headloss (feet) C = 100	Friction Headloss (feet) C = 150	Total Dynamic Head (ft) C = 100	Total Dynamic Head (ft) C = 150
0	0.00000	0.00000	0.00000	0.00000	0.00000	0	0	19	19
100	0.22282	0.63832	0.00633	0.00042	0.00020	1	0	20	19
200	0.44563	1.27664	0.02531	0.00152	0.00072	2	1	21	20
300	0.66845	1.91497	0.05694	0.00321	0.00152	5	2	24	21
400	0.89127	2.55329	0.10123	0.00547	0.00258	9	4	28	23
500	1.11408	3.19161	0.15817	0.00827	0.00391	13	6	32	25
600	1.33690	3.82993	0.22777	0.01159	0.00547	18	9	37	28
700	1.55971	4.46825	0.31002	0.01541	0.00728	24	11	44	31
800	1.78253	5.10658	0.40492	0.01973	0.00932	31	15	50	34
900	2.00535	5.74490	0.51248	0.02453	0.01159	39	18	58	38
1,000	2.22816	6.38322	0.63269	0.02981	0.01408	47	22	67	42
1,100	2.45098	7.02154	0.76556	0.03556	0.01680	56	26	76	46
1,200	2.67380	7.65986	0.91108	0.04177	0.01973	66	31	86	51

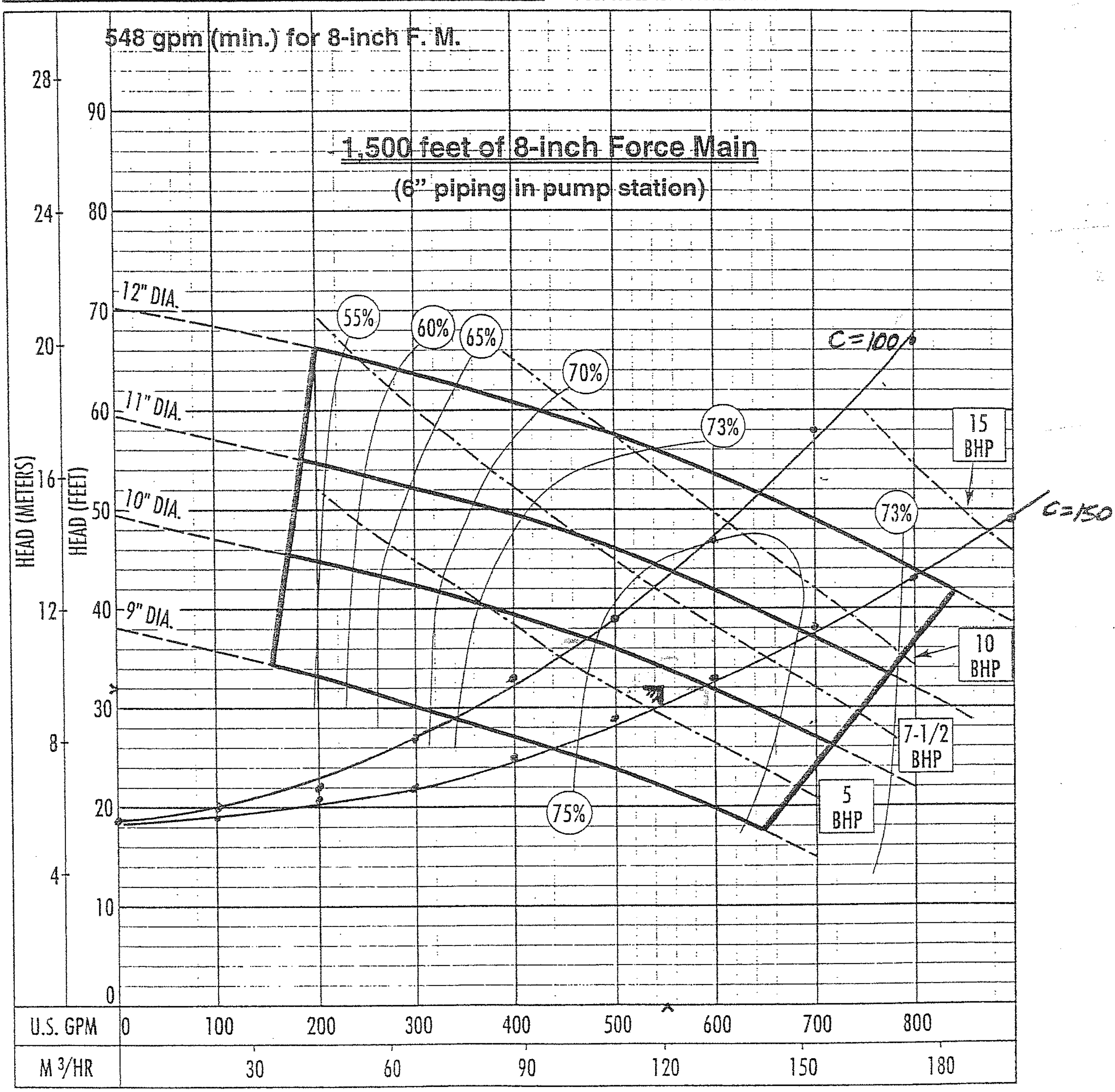
Figure LG-1
Timber Park Pump Station

Performance Curve S4B/S4BX

126 Acres

RPM: **1150** Discharge: **4"** Solids: **3"**

NOTE: FOR USE WITH M-T-M AND PULTRUDED RAIL SYSTEM ONLY.



The curves reflect maximum performance characteristics without exceeding full load (Nameplate) horsepower. All pumps have a service factor of 1.2. Operation is recommended in the bounded area with operational point within the curve limit. Performance curves are based on actual tests with clear water at 70° F. and 1280 feet site elevation.

Conditions of Service:
GPM: 550 TDH: 32'

Table LG-4
Wet Well Calculations

Case # 1 --- Light Industrial				Case # 2 --- Light Industrial			
Area	126 acres			Area	172 acres		
Min. flow	26 to	39 gpm		Min. flow	36 to	54 gpm	
Avg. flow	88 to	131 gpm		Avg. flow	119 to	179 gpm	
Peak flow	350 to	656 gpm		Peak flow	478 to	896 gpm	
FM diam.	8 inches			FM diam.	8 inches		
FM length	1,500 feet			FM length	1,500 feet		
FM vol.	3,917 gal			FM vol.	3,917 gal		
FM min. Q	548 gpm @ 3.5fps			FM min. Q	548 gpm @ 3.5fps		

WET WELL VOLUME

V = (60/N/t)(q/Q)(Q-q)

	Case # 1		Case # 2
N = No. of pumps	1	N =	1
t = Cycle times (@ avg.)	6 per hour	t =	6 per hour
q = Avg. influent flow	109 gpm	q =	149 gpm
Q = Pump discharge	548 gpm	Q =	687 gpm
Wet Well Volume, V =	876 gallons	**V =**	1,168 gallons
Depth @ 48" dia. =	9.3 feet	Depth @ 48" dia. =	12.4 feet
Depth @ 60" dia. =	6.0 feet	Depth @ 60" dia. =	7.9 feet
Depth @ 72" dia. =	4.1 feet	Depth @ 72" dia. =	5.5 feet
Depth @ 96" dia. =	2.3 feet	Depth @ 96" dia. =	3.1 feet

	Case # 1		Case # 2
Force Main Volume =	3,917 gallons	**FM Vol. =**	3,917 gallons
Depth @ 48" dia. =	41.7 feet	Depth @ 48" dia. =	41.7 feet
Depth @ 60" dia. =	26.6 feet	Depth @ 60" dia. =	26.6 feet
Depth @ 72" dia. =	18.5 feet	Depth @ 72" dia. =	18.5 feet
Depth @ 96" dia. =	10.4 feet	Depth @ 96" dia. =	10.4 feet

t = Pump Starts/Hr (@50% capacity)

t = (60/N/V)(q/Q)(Q-q)

q = Q/2 (inflow @ 50% of pump capacity)

	Case # 1		Case # 2
t = (60/N/V)(Q/4) =	9 per hour	t =	9 per hour

Detention Time (mins)	**Wetwell**	**FM**	**Det. Time** (mins)	**Wetwell**	**FM**
Low minimum flow	33	149	Low minimum flow	33	109
High minimum flow	22	99	High minimum flow	22	73
Low average flow	10	45	Low average flow	10	33
High average flow	7	30	High average flow	7	22
Low peak flow	3	11	Low peak flow	2	8
High peak flow	1	6	High peak flow	1	4

Case # 3 --- Light Industrial			Case # 4 --- Light Industrial		
Area	210 acres		Area	300 acres	
Min. flow	44 to	66 gpm	Min. flow	63 to	94 gpm
Avg. flow	146 to	219 gpm	Avg. flow	208 to	313 gpm
Peak flow	583 to	1,094 gpm	Peak flow	833 to	1,563 gpm
FM diam.	8 inches		FM diam.	8 inches	
FM length	1,500 feet		FM length	1,500 feet	
FM vol.	3,917 gal		FM vol.	3,917 gal	
FM min. Q	548 gpm @ 3.5fps		FM min. Q	548 gpm @ 3.5fps	

WET WELL VOLUME

V = (60/N/t)(q/Q)(Q-q)

Case # 3		Case # 4	
N = No. of pumps	1	N =	1
t = Cycle times (@ avg.)	6 per hour	t =	6 per hour
q = Avg. influent flow	182 gpm	q =	260 gpm
Q = Pump discharge	839 gpm	Q =	1,198 gpm
Wet Well Volume, V =	1,427 gallons	V =	2,038 gallons
Depth @ 48" dia. =	15.2 feet	Depth @ 48" dia. =	21.7 feet
Depth @ 60" dia. =	9.7 feet	Depth @ 60" dia. =	13.9 feet
Depth @ 72" dia. =	6.7 feet	Depth @ 72" dia. =	9.6 feet
Depth @ 96" dia. =	3.8 feet	Depth @ 96" dia. =	5.4 feet

Case # 3		Case # 4	
Force Main Volume =	3,917 gallons	**FM Vol. =**	3,917 gallons
Depth @ 48" dia. =	41.7 feet	Depth @ 48" dia. =	41.7 feet
Depth @ 60" dia. =	26.6 feet	Depth @ 60" dia. =	26.6 feet
Depth @ 72" dia. =	18.5 feet	Depth @ 72" dia. =	18.5 feet
Depth @ 96" dia. =	10.4 feet	Depth @ 96" dia. =	10.4 feet

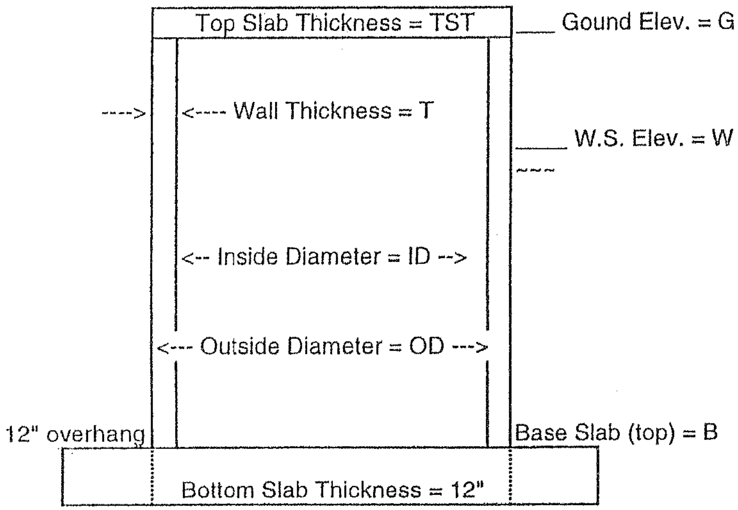
t = Pump Starts/Hr (@50% capacity)

t = (60/N/V)(q/Q)(Q-q)

q = Q/2 (inflow @ 50% of pump capacity)

Case # 3		Case # 4	
t = (60/N/V)(Q/4) =	9 per hour	t =	9 per hour

Detention Time (mins)	**Wetwell**	**FM**	**Det. Time** (mins)	**Wetwell**	**FM**
Low minimum flow	33	90	Low minimum flow	33	63
High minimum flow	22	60	High minimum flow	22	42
Low average flow	10	27	Low average flow	10	19
High average flow	7	18	High average flow	7	13
Low peak flow	2	7	Low peak flow	2	5
High peak flow	1	4	High peak flow	1	3



ID (in)	OD (in)	T (in)	TST (in)	Weight (lbs/ft) MH	Weight (lbs/ft) Buoyancy	Slab Weight (lbs) Top	Slab Weight (lbs) Base
48	58	5	8	936	1,145	1,835	7,004
60	72	6	10	1,427	1,764	3,534	9,600
72	87.5	7.75	10	2,140	2,606	5,220	12,950
96	114	9	12	3,120	4,423	10,632	19,838

<u>MH Weight, Buoyancy, and Slab Weight Assumptions</u>:

MH Shaft: weights listed for typical concrete pipe sections

Buoyancy

Bouyancy: volume displaced by MH shaft @ 62.4 lbs per cu. ft.

[for worst case, assume water table is at ground level]

Top Slab: round; overhang = 0 inches; thickness = TST

Base Slab: square; overhang = 12 inches; thickness = 12 inches

Table LG-6

<u>Pounds of Force for Various Manhole Diameters and Depths [NOT INCLUDING weight of soil above base slab]</u>

ID (in)	G - B = 8 Uplift	G - B = 8 Weight	G - B = 10 Uplift	G - B = 10 Weight	G - B = 12 Uplift	G - B = 12 Weight	G - B = 14 Uplift	G - B = 14 Weight	G - B = 16 Uplift	G - B = 16 Weight	G - B = 18 Uplift	G - B = 18 Weight	G - B = 20 Uplift	G - B = 20 Weight
48	12,073	16,327	14,363	18,199	16,653	20,071	18,942	21,943	21,232	23,815	23,522	25,687	25,812	27,559
60	18,108	24,550	21,637	27,404	25,165	30,258	28,694	33,112	32,223	35,966	35,751	38,820	39,280	41,674
72	26,233	35,290	31,445	39,570	36,656	43,850	41,867	48,130	47,079	52,410	52,290	56,690	57,502	60,970
96	43,637	55,430	52,483	61,670	61,329	67,910	70,175	74,150	79,021	80,390	87,867	86,630	96,713	92,870

<u>Pounds of Force for Various Manhole Diameters and Depths [INCLUDING weight of soil above base slab]</u>

ID (in)	G - B = 8 Uplift	G - B = 8 Weight	G - B = 10 Uplift	G - B = 10 Weight	G - B = 12 Uplift	G - B = 12 Weight	G - B = 14 Uplift	G - B = 14 Weight	G - B = 16 Uplift	G - B = 16 Weight	G - B = 18 Uplift	G - B = 18 Weight	G - B = 20 Uplift	G - B = 20 Weight
48	12,073	24,854	14,363	28,857	16,653	32,861	18,942	36,865	21,232	40,868	23,522	44,872	25,812	48,876
60	18,108	35,297	21,637	40,837	25,165	46,378	28,694	51,918	32,223	57,459	35,751	62,999	39,280	68,540
72	26,233	48,699	31,445	56,331	36,656	63,963	41,867	71,595	47,079	79,227	52,290	86,860	57,502	94,492
96	43,637	73,889	52,483	84,744	61,329	95,599	70,175	106454	79,021	117309	87,867	128164	96,713	139018

SEWAGE PUMP STATION PLAN-REVIEW SUBMITTAL
State of Oregon / Department of Environmental Quality

Owner/Facility:	**City of Estacada, Oregon --- Campus Pump Station**
Address/Location:	**Park Industrial Campus** (West of Highway 224 and North of River Mill Road)
Area Served:	**30 acres of Light Industrial Development**
Pump Station Type:	**Wet-well, side-mounted, submersible, non-clog, duplex, lead-lag**

Design Summary:

A duplex sewage pump station is needed to serve 30 acres of Light Industrial Development, with each pump handling a peak flow of approximately 140 gpm. The pump station is not expected to serve any additional area. If industrial property to the west should be developed in the future, an additional pump station would be required, since that area is generally lower in elevation.

If a future pump station is constructed to serve the additional area, the currently proposed pump station could continue to serve the initial 30 acres, or it could be abandoned and replaced with a gravity sewer to the future pump station to the west. The future pump station would need to be appropriately sized.

The currently proposed pump station will use a 4-inch force main discharging at 3.5 fps minimum velocity into a new gravity sewer installed as part of the overall project. In the event of significantly low flows (i.e. long force main detention), a timeclock-controlled pinch valve will empty the pipe contents back to the wet well. The future pump station, if any, is expected to be able to use all or a portion of the initial 4-inch force main, and pump to the same manhole.

1. SEWAGE PUMP STATION DESIGN CALCULATIONS AND DOCUMENTS

System Sizing and Design Basis Calculations

The pump station has been designed to serve 30 acres of Light Industrial Development. Sewage flow calculations, shown on Table SM-1, have been estimated using a range of both Low and High values for the following conditions: Low Minimum, High Minimum, Low Average, High Average, Low Peak (no I/I), High Peak (no I/I), Low Peak (with I/I), and High Peak (with I/I).

The pump station is designed for light industrial businesses which generate 1,000 to 1,500 gal/acre/day (gpapd), or an average of 1,250 gpapd. A peaking factor of 3.0 is then applied, and an I/I range of 1,000 to 3,000 gpapd (average of 2,000 gpapd) is added. Using these values, inflows are estimated to peak at about 120 gpm (range: 83 gpm to 156 gpm) at full buildout. Inflows at start-up can be expected to be nearly zero gallons per min (gpm) during early morning hours when businesses are closed.

The wetwell will be sized with extra capacity to accommodate backdrainage of the force main during low inflow conditions. If inflows should increase beyond design values, and backdrainage is no longer required, larger pumps could be installed to provide double, triple, or even greater flow capacity.

Hydraulics / Headloss Calcs / Pump Curve / System Head Curves

Force main and headloss calculations are shown on Table SM-2. A pumping velocity of 3.5 feet per second (fps) is used; this results in the selection of a 4-inch diameter force main, which is typically a minimum diameter for sewage flows. C-factors of C = 150 (new, smooth pipe) and C = 100 (old, rough pipe), have been used to estimate friction losses, and resulting Total Dynamic Head (TDH) values used to select the pump.

Figure SM-1 depicts the two system curves (C = 100 & 150) plotted against the pump curves for a 4" Aurora/Hydromatic non-clog centrifugal pump (Model S4N/S4NX) operating at 1750 rpm. For this pump, a 6.90" diameter impeller will pump the design flow. A pump motor of 5 Hp would be needed to preclude overloading in low-head or zero-head start-up conditions.

Pump Starts Per Hour (@50% Pump Capacity)

Wetwell sizing calculations are shown on Table SM-3. Though the station will use two pumps alternating on a lead-lag scenario, it is assumed only one pump may be operating at times, with the other out-of-service for replacement or maintenance. In this case, the one pump would cycle at a maximum of 10 starts per hour (@ inflow of 50% of the pump capacity). During normal operating conditions with two pumps operating, each pump would cycle at a maximum of 5 starts per hour. (According to the manufacturer, the proposed pump can cycle at up to 11 starts per hour.) During average flow conditions, one pump would cycle at 6 starts per hour; two pumps at 3 starts per hour each.

Wetwell Buoyancy Calculations

Wetwell buoyancy calculations are shown on Table SM-4. The Campus pump station will be a 72 -inch diameter wetwell, and it is 18 feet deep (ground elevation is at about 422.0 and the bottom of the wetwell is at 404.00). Conservatively, assume the water table is at ground level, the wetwell is empty, and exclude the weight of piping, pumps and equipment. The weight of the concrete is 56,690 pounds (including top and base slabs). The buoyant force is 52,290 pounds, which results in a 1.08 safety factor. Including the weight of the soil directly above the overhang on the base slab (an additional 30,170 pounds), the safety factor is 1.66. Including skin friction imposed by the granular backfill, against the side walls of the wetwell, the safety factor would be significantly greater. The wetwell (with base slab overhang) is of adequate weight to resist buoyancy.

Uncommon Equipment Design

A backdrainage system will utilize a timeclock-controlled pinch valve. Otherwise, the pump station has no uncommon equipment.

Plan and Profile of Force Main (including details)

The plan and profile drawings for the force main are included separately. Plan, section and detail drawings are also included for the pump station.

Wetwell and Force Main Detention Time Calculations

Detention times for the wetwell and force main vary, depending upon flow rates. Table SM-3 lists the respective detention times for various flows. The left column lists detention times for the wetwell, assuming no provisions for force main backdrainage. The middle column lists times for the force main. The right column lists detention times for the wetwell, assuming force main backdrainage is provided.

Due to the relatively long detention times in the force main at minimum flows (i.e. more than one hour), backdrainage of the force main will be provided. For 1,170 feet of 4-inch force main (i.e. 764 gallons), 3.6 feet of depth will be provided in a 6-foot diameter wetwell.

For average inflow (26 gpm), the detention time in the force main is about 30 minutes, and the detention time in the wetwell is about 40 minutes. For peak inflow (120 gpm), the detention times in both the wetwell and the force main are less than 30 minutes.

Sewage Overflow when Station or Power Fails

If the pump station should overflow, due to unforeseen mechanical or electrical failure, there is very low potential for human or household pet contact since it is located in an industrial area. There are no adjacent creeks or drainageways, nor any nearby wells that could be contaminated. The pump station will have portable standby power, and will have two pumps installed, each with the capacity to handle the peak design inflow, should one pump be out of service at any time.

Standby Power and Alarm Considerations

Standby power will be provided via a new, mobile, trailer-mounted generator, located at the nearby Timber Park pump station, near the City of Estacada's Wastewater Treatment Plant (WWTP). Alarms will be telemetered to the WWTP via an autodialer.

Water Service at Pump Station

Water service will be provided at the pump station to maintain the wetwell and to clean equipment. New waterlines are being constructed as part of the overall development.

2. PUMP STATION DESIGN DATA

A design data sheet for the pump station, which follows the format provided by DEQ, is enclosed.

3. SCHEDULE OF ALARM ELEVATIONS AND CONDITIONS

A schedule of alarm elevations and conditions is shown on the drawings.

4. SITE PLAN AND ELECTRICAL/CONTROL DRAWINGS

Site plan and electrical/control drawings are provided.

5. CAPACITY OF DOWNSTREAM SEWERS

All downstream sewers, including a larger downstream pump station which will pump directly to the city's wastewater treatment plant, are being designed as a part of this project and will accommodate the flow from this pump station.

6. EXISTING PUMP STATION PROVISIONS

This is a new pump station, so there are no existing pump station provisions to address.

7. PROJECT INSPECTION PROVISIONS

The project will be inspected in accordance with OAR 340-52-015(1)(e).

8. O&M MANUAL PROVISIONS

Operation and maintenance requirements and manuals will be provided by the pumping system supplier.

9. CITY'S REVIEW AND APPROVAL OF PLANS

The proposed pump station has been developed in direct coordination with the City of Estacada, to meet their design, operational, and maintenance requirements.

10. TECHNICAL ACTIVITIES FEE

Please directly invoice the City of Estacada for the technical activities fee.

SEWAGE PUMP STATION DESIGN DATA

Owner/Facility:	**City of Estacada, Oregon --- Campus Pump Station**
Address/Location:	**Park Industrial Campus** (West of Highway 224 and North of River Mill Road)
Area Served:	**30 acres of Light Industrial Development**

PUMP STATION

Type:	Wet-well submersible, side-mounted, duplex, lead-lag
Pump Type:	Constant speed, 1750 rpm, non-clog
Capacity:	140 gpm at 35 ft Total Dynamic Head
Pump Hp (each):	5 Hp
Level Control Type:	Ultrasonic level transmitter; PLC-controlled; backed up by intrinsically safe mercury level sensors
Overflow Point:	Elev. 422.0 (approx.)
Overflow Discharge:	To adjacent ground around station
Avg. Time to Overflow:	2+ hours at 26 gpm design average Q
Auxiliary Power Type:	Portable diesel generator
Location:	Timber Park P.S. (nearby; near City WWTP)
Output:	35 KW (240 volt, 3-phase)
Fuel Tank Capacity:	24 hours
Transfer Switch:	Manual; mounted at the pump station
Alarm Telemetry Type:	Autodialer; using local telephone line
EPA Reliability Class:	I

FORCE MAIN

Length, Type:	1,170 feet of 4-inch PVC
Profile:	Continuously ascending at approx. 0.5% to 1% slope
Discharge Manhole:	Terminus manhole for Schedule 1 gravity sewer (unnamed street)
Air Release Valves:	None (no localized high points)
Vacuum Release Valves:	None
Average Detention:	122 min at 6gpm (startup); 30 min at 26gpm (average ultimate)
Sulfide Control System:	Backdrainage

BACKDRAINAGE SYSTEM

Control Valve Type:	Pinch Valve
Valve Size:	4-inch

Land Use	Sewage (gal/acre/day) Low Avg.	High Avg.
Commercial	800	1,500
Light Industrial	1,000	1,500
Medium Industrial	1,500	3,000

Flow Case	Factor (for Minimum and Peak)	
Minimum	30%	[range: 30% to 50%]
Peak (Hrly)	300%	[< 2 mgd Avg. Flow]
Peak (Hrly)	150%	[> 2 mgd Avg. Flow]

Infiltration / Inflow (gal/acre/day)	
I/I - Low	1,000
I/I - High	3,000

	30 <- Site (acreage) Land Use	Sewage Flow (gal/day) Low Avg.	High Avg.	Sewage Flow (gal/day) Low Min.	High Min.	Sewage Flow (gal/day) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/day) Low Peak	High Peak
G	Commercial	24,000	45,000	7,200	13,500	72,000	135,000	102,000	225,000
P	Light Industrial	30,000	45,000	9,000	13,500	90,000	135,000	120,000	225,000
D	Medium Industrial	45,000	90,000	13,500	27,000	135,000	270,000	165,000	360,000

	<- Site (acreage) Land Use	Sewage Flow (gal/day) Low Avg.	High Avg.	Sewage Flow (gal/day) Low Min.	High Min.	Sewage Flow (gal/day) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/day) Low Peak	High Peak
G	Commercial	0	0	0	0	0	0	0	0
P	Light Industrial	0	0	0	0	0	0	0	0
D	Medium Industrial	0	0	0	0	0	0	0	0

	30 <- Site (acreage) Land Use	Sewage Flow (gal/hour) Low Avg.	High Avg.	Sewage Flow (gal/hour) Low Min.	High Min.	Sewage Flow (gal/hour) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/hr) Low Peak	High Peak
G	Commercial	1,000	1,875	300	563	3,000	5,625	4,250	9,375
P	Light Industrial	1,250	1,875	375	563	3,750	5,625	5,000	9,375
H	Medium Industrial	1,875	3,750	563	1,125	5,625	11,250	6,875	15,000

	0 <- Site (acreage) Land Use	Sewage Flow (gal/hour) Low Avg.	High Avg.	Sewage Flow (gal/hour) Low Min.	High Min.	Sewage Flow (gal/hour) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/hr) Low Peak	High Peak
G	Commercial	0	0	0	0	0	0	0	0
P	Light Industrial	0	0	0	0	0	0	0	0
H	Medium Industrial	0	0	0	0	0	0	0	0

	30 <- Site (acreage) Land Use	Sewage Flow (gal/min) Low Avg.	High Avg.	Sewage Flow (gal/min) Low Min.	High Min.	Sewage Flow (gal/min) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/min) Low Peak	High Peak
G	Commercial	17	31	5	9	50	94	71	156
P	Light Industrial	21	31	6	9	63	94	83	156
M	Medium Industrial	31	63	9	19	94	188	115	250

	0 <- Site (acreage) Land Use	Sewage Flow (gal/min) Low Avg.	High Avg.	Sewage Flow (gal/min) Low Min.	High Min.	Sewage Flow (gal/min) Low Peak	High Peak	Sewage Flow [incl I/I] (gal/min) Low Peak	High Peak
G	Commercial	0	0	0	0	0	0	0	0
P	Light Industrial	0	0	0	0	0	0	0	0
M	Medium Industrial	0	0	0	0	0	0	0	0

Table SM-1

Table SM-2

Pipe D (in) =	4
GPM @ 3.5fps =	137
Set Flow Increment (gpm) --->	25
Pipe Vol. (gal) =	764

4" Fittings	Eq. Length (ft)
45-bend	5
90-bend	11
side tee	22
gate valve	2
BF valve	
swing chk	27
other?	

Static (ft) = 16

Flow (gpm)	Flow (cfs)	Velocity (fps)	V*V/2g (ft)	Friction Headloss (ft/ft) C = 100	Friction Headloss (ft/ft) C = 150	Total Equiv. Length (ft)	Friction Headloss (feet) C = 100	Friction Headloss (feet) C = 150	Total Dynamic Head (ft) C = 100	Total Dynamic Head (ft) C = 150
0	0.00000	0.00000	0.00000	0.00000	0.00000	1,331	0	0	16	16
25	0.05570	0.63832	0.00633	0.00094	0.00045		1	1	17	17
50	0.11141	1.27664	0.02531	0.00341	0.00161	Length (ft)	5	2	21	18
75	0.16711	1.91497	0.05694	0.00721	0.00341	1,170	10	5	26	21
100	0.22282	2.55329	0.10123	0.01228	0.00580		16	8	32	24
125	0.27852	3.19161	0.15817	0.01855	0.00876	No. Fittings	25	12	41	28
150	0.33422	3.82993	0.22777	0.02599	0.01228	4	35	16	51	33
175	0.38993	4.46825	0.31002	0.03457	0.01633	8	46	22	62	38
200	0.44563	5.10658	0.40492	0.04426	0.02090	1	59	28	75	44
225	0.50134	5.74490	0.51248	0.05503	0.02599	2	73	35	90	51
250	0.55704	6.38322	0.63269	0.06688	0.03159		89	42	106	59
275	0.61275	7.02154	0.76556	0.07977	0.03768	1	106	50	123	67
300	0.66845	7.65986	0.91108	0.09370	0.04426		125	59	142	76

Pipe D (in) =	6
GPM @ 3.5fps =	308
Set Flow Increment (gpm) --->	50
Pipe Vol. (gal) =	1,718

6" Fittings	Eq. Length (ft)
45-bend	8
90-bend	16
side tee	33
gate valve	4
BF valve	
swing chk	40
other?	

Static (ft) = 16

Flow (gpm)	Flow (cfs)	Velocity (fps)	V*V/2g (ft)	Friction Headloss (ft/ft) C = 100	Friction Headloss (ft/ft) C = 150	Total Equiv. Length (ft)	Friction Headloss (feet) C = 100	Friction Headloss (feet) C = 150	Total Dynamic Head (ft) C = 100	Total Dynamic Head (ft) C = 150
0	0.00000	0.00000	0.00000	0.00000	0.00000	1,411	0	0	16	16
50	0.11141	0.56740	0.00500	0.00047	0.00022		1	0	17	16
100	0.22282	1.13479	0.02000	0.00171	0.00081	Length (ft)	2	1	18	17
150	0.33422	1.70219	0.04499	0.00361	0.00171	1,170	5	2	21	18
200	0.44563	2.26959	0.07999	0.00615	0.00291		9	4	25	20
250	0.55704	2.83699	0.12498	0.00930	0.00439	No. Fittings	13	6	29	22
300	0.66845	3.40438	0.17997	0.01303	0.00615	4	18	9	35	25
350	0.77986	3.97178	0.24495	0.01733	0.00819	8	24	12	41	28
400	0.89127	4.53918	0.31994	0.02219	0.01048	1	31	15	48	31
450	1.00267	5.10658	0.40492	0.02759	0.01303	2	39	18	55	35
500	1.11408	5.67397	0.49991	0.03353	0.01584		47	22	64	39
550	1.22549	6.24137	0.60489	0.03999	0.01889	1	56	27	73	43
600	1.33690	6.80877	0.71987	0.04698	0.02219		66	31	83	48

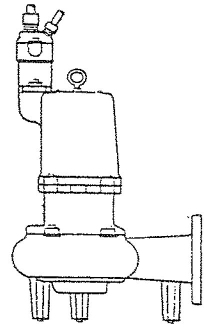
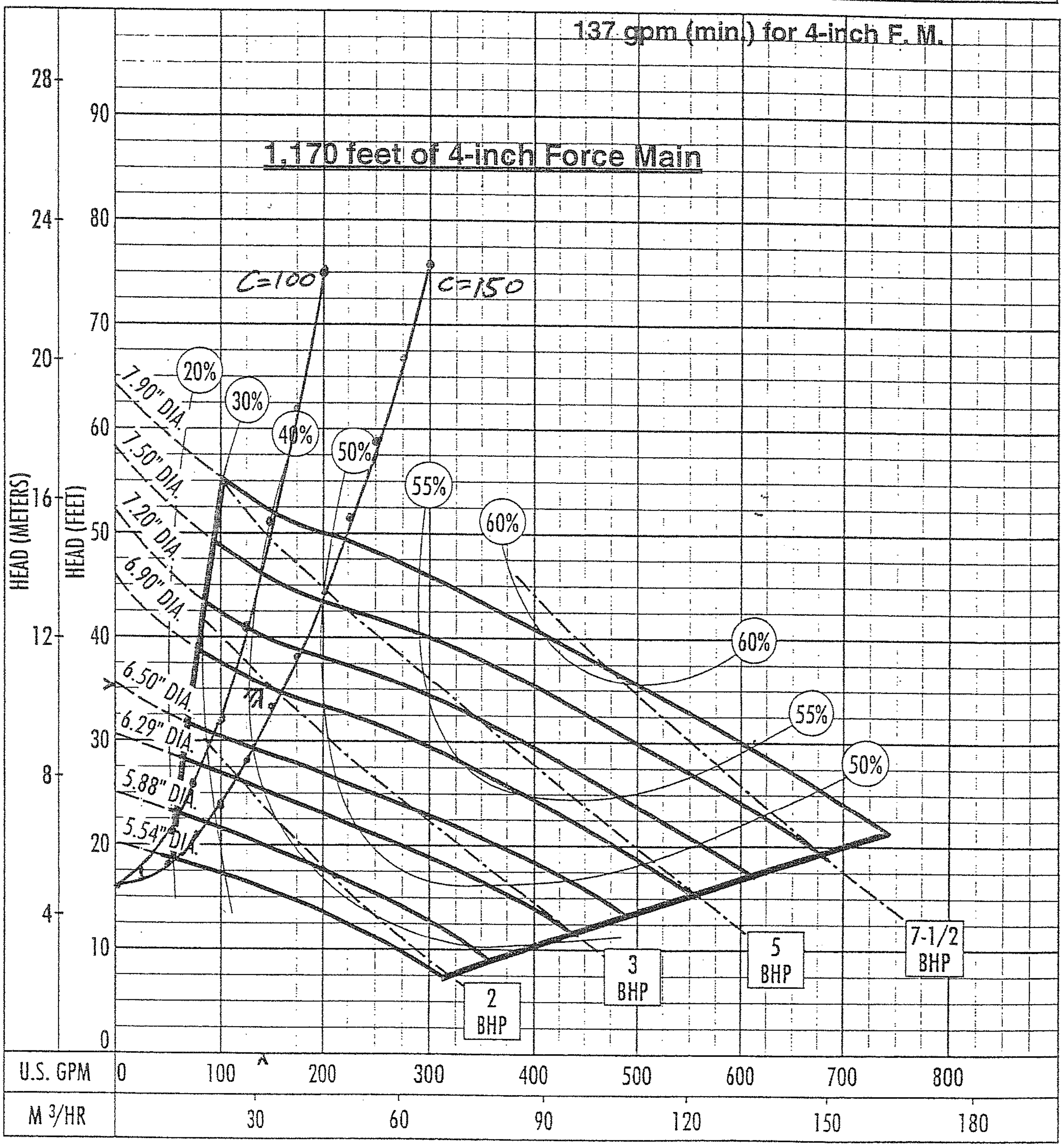


Figure SM-1
Campus Pump Station

30 Acres

Performance Curve **S4N/S4NX**

RPM: **1750** Discharge: **4"** Solids: **3"**



The curves reflect maximum performance characteristics without exceeding full load (Nameplate) horsepower. All pumps have a service factor of 1.2. Operation is recommended in the bounded area with operational point within the curve limit. Performance curves are based on actual tests with clear water at 70° F. and 1280 feet site elevation.

Conditions of Service:
GPM: 140 TDH: 35'

Table SM-3
Wet Well Calculations

Case # 1 --- Light Industrial			
Area	30 acres		
Min. flow	6 to	9	gpm
Avg. flow	21 to	31	gpm
Peak flow	83 to	156	gpm
FM diam.	4 inches		
FM length	1,170 feet		
FM vol.	764 gal		
FM min. Q	137 gpm @ 3.5fps		

WET WELL VOLUME

V = (60/N/t)(q/Q)(Q-q)

N = No. of pumps	1
t = Cycle times (@ avg.)	6 per hour
q = Avg. influent flow	26 gpm
Q = Pump discharge	137 gpm

Wet Well Volume, V = 211 gallons

Depth @ 48" dia. =	2.2 feet
Depth @ 60" dia. =	1.4 feet
Depth @ 72" dia. =	1.0 feet
Depth @ 96" dia. =	0.6 feet

Force Main Volume = 764 gallons

Depth @ 48" dia. =	8.1 feet
Depth @ 60" dia. =	5.2 feet
Depth @ 72" dia. =	3.6 feet
Depth @ 96" dia. =	2.0 feet

---

t = Pump Starts/Hr (@50% capacity)

t = (60/N/V)(q/Q)(Q-q)

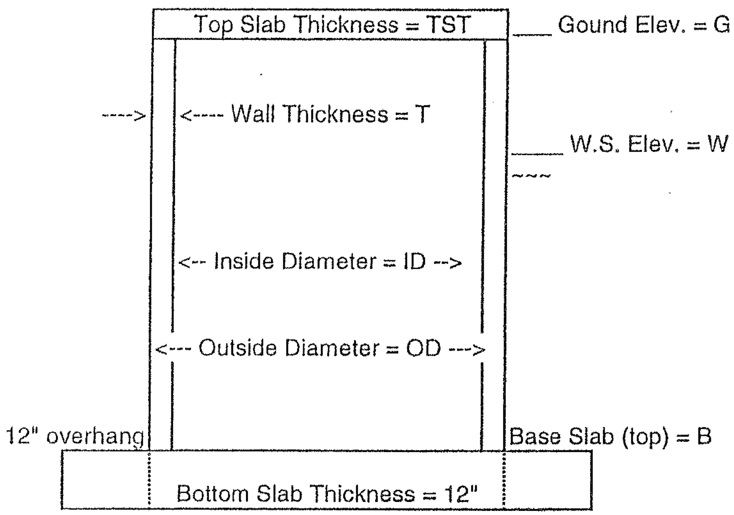
q = Q/2 (inflow @ 50% of pump capacity)

t = (60/N/V)(Q/4) = 10 per hour

Detention Time (mins)	**Wetwell** (no B.D.)	**FM**	**Wetwell** (with B.D.)
Low minimum flow	34	122	156
High minimum flow	23	81	104
Low average flow	10	37	47
High average flow	7	24	31
Low peak flow	3	9	12
High peak flow	1	5	6

B.D. = BackDrainage

Table SM-4



				Weight (lbs/ft)		Slab Weight (lbs)	
ID (in)	OD (in)	T (in)	TST (in)	MH	Buoyancy	Top	Base
48	58	5	8	936	1,145	1,835	7,004
60	72	6	10	1,427	1,764	3,534	9,600
72	87.5	7.75	10	2,140	2,606	5,220	12,950
96	114	9	12	3,120	4,423	10,632	19,838

MH Weight, Buoyancy, and Slab Weight Assumptions:
MH Shaft: weights listed for typical concrete pipe sections
Bouyancy: volume displaced by MH shaft @ 62.4 lbs per cu. ft.
[for worst case, assume water table is at ground level]
Top Slab: round; overhang = 0 inches; thickness = TST
Base Slab: square; overhang = 12 inches; thickness = 12 inches

Pounds of Force for Various Manhole Diameters and Depths [NOT INCLUDING weight of soil above base slab]

	G - B = 8		G - B = 10		G - B = 12		G - B = 14		G - B = 16		G - B = 18		G - B = 20	
ID (in)	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight
48	12,073	16,327	14,363	18,199	16,653	20,071	18,942	21,943	21,232	23,815	23,522	25,687	25,812	27,559
60	18,108	24,550	21,637	27,404	25,165	30,258	28,694	33,112	32,223	35,966	35,751	38,820	39,280	41,674
72	26,233	35,290	31,445	39,570	36,656	43,850	41,867	48,130	47,079	52,410	52,290	56,690	57,502	60,970
96	43,637	55,430	52,483	61,670	61,329	67,910	70,175	74,150	79,021	80,390	87,867	86,630	96,713	92,870

Pounds of Force for Various Manhole Diameters and Depths [INCLUDING weight of soil above base slab]

	G - B = 8		G - B = 10		G - B = 12		G - B = 14		G - B = 16		G - B = 18		G - B = 20	
ID (in)	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight	Uplift	Weight
48	12,073	24,854	14,363	28,857	16,653	32,861	18,942	36,865	21,232	40,868	23,522	44,872	25,812	48,876
60	18,108	35,297	21,637	40,837	25,165	46,378	28,694	51,918	32,223	57,459	35,751	62,999	39,280	68,540
72	26,233	48,699	31,445	56,331	36,656	63,963	41,867	71,595	47,079	79,227	52,290	86,860	57,502	94,492
96	43,637	73,889	52,483	84,744	61,329	95,599	70,175	106454	79,021	117309	87,867	128164	96,713	139018

APPENDIX IV

CURRENT METHODOLOGY FOR CALCULATING SYSTEM DEVELOPMENT CHARGES

SANITARY SEWER UTILITY SDC

Reimbursement Fee

Using the 1992 audit[3] and data from other city records on investments at the wastewater treatment plant, we determined the current value of the sewer system that is assessable as a reimbursement fee. The amount assessable is based on two factors. The first factor is the current depreciated value of the utility's recent capital improvements less contributed capital, plus the cash balance. Table 3 and Note 1 of Table 3 show the two values. The second factor is the percentage of excess capacity in the sewer system. The product of the two factors equals the dollar amount assessable as a reimbursement fee. It equals the value of plant and cash reserves rate payers have invested in the system for future development. Table 4 shows the calculations.

The excess capacity is based on the treatment plant. Sewer lines are built to serve all of the expected demand in a particular drainage basin, hence excess capacity exists in most of the sewer lines. Also, most of the small collector and some of the larger sewer lines were built by private developers (or land owners through local improvement districts) and contributed to the city. All of the non-contributed investment by the city are therefore in the treatment plant.

In 1992, the treatment plant had a maximum capacity of 4.5 million gallons per day (mgd), and the current maximum flow was 1.983 mgd. Our assumption about maximum capacity actually overstates the capacity of some components of the plant. For example the gritworks is the first major component of the treatment plant and it has a design capacity of only 2.5 mgd and the comminutor at 3.5 mgd. However, the majority of the investments in the plant (the primary and secondary treatment components) have a capacity of 4.5 mgd. The plant was last upgraded in fiscal 1990.

To design the reimbursement fee, we based it on likely maximum flow by size of water meter installed. Since the city has no economical means of determining the flow of sewage from a proposed development (particularly for non-residential), water meter size for the proposed development will provide as unambiguous a measure of potential sewage flow as can be economically determined.

The flows are based on the current water usage for a 3/4-inch meter. The reimbursement fee for a meter equals the number of 3/4-inch meter equivalents multiplied by 383 gallons per day.

---

[3] *City of Estacada, Oregon Report on Examination of Financial Statements and Supplementary Data for the fiscal year ending June 30, 1992* by Henton & Company, P.C.

as much flow as a 3/4-inch meter.

Table 3
City of Estacada, Sewer Utility
Cost Basis for Reimbursement Fee

	1993
Total Current Assets	$316,712
Fixed Assets net of accumulated depreciation (Note 1)	
Treatment Plant	685,653
Lime Stabilization	50,000
Net Value of Plant	$1,052,365
Treatment Capacity	0.54 mgd
Current Maximum Flow	0.275 mgd
Excess Capacity	0.265 mgd
Percent Excess Capacity	49.1%
Value Attributable to Future Development	$516,438
Reimbursement Fee/Gallon	$1.95 / gallon
Average household flow	252.77 gallons
Reimbursement Fee Average Household Flow, 3/4" Meter	$493

Source: City of Estacada, Clackamas County, Oregon, Annual Financial Report for the year ending June 30, 1993. The value of plant in service is derived in Note 1.

Note 1: The original City cost, date in service, depreciation schedule and net depreicated value in 1994 are:

Plant Improvements	Original Cost	Year in Srvc.	Life	Depreciated Value
Treatment Plant	$857,066	1990	20	$685,653
I/I Repairs	50,000	1988	10	20,000
I/I Repairs	40,000	1993	10	36,000
Lime Stabilization	50,000	1994	20	50,000
	$997,066			

The I/I project benefits only existing development.
The City actually issued a $1 million bond issue in 1987 for these projects.

Improvement Fee

The improvement fee is based on the *City of Estacada Sanitary Sewer Distribution System Capital Improvement Program*, (February 1994, by Curran-McLeod, Inc.). Table 4 shows the 10 projects in the CIP. Each project either replaces with a larger diameter sewer pipes in already developed areas, or add new main sewage collection lines to currently undeveloped land. All of these lines are interceptor lines designed primarily to convey sewage from lateral collection systems to the sewage treatment plant. The total construction costs in 1994 dollars sum to $4,278,800. These costs do not include the costs of lateral collection lines that are customarily installed by developers.

Three of the proposed interceptors will rehabilitate subsystems 1, 2, and 4 of the existing collection system. These portions of the collection system will be replaced with larger diameter pipes to convey sewage to be collected in the proposed Regan Hill and Cemetery Road interceptors. The cost of the rehabilitation projects is divided equally between the Regan Hill and Cemetery Road sewer line projects.

The remaining interceptor projects will serve areas of the city that are not now sewered. These areas are undeveloped. The CIP identifies the number of acres to be served by each line. The pipe diameters and design standards are based on an average flow generated by 8 single-family housing units per acre. Table 5 shows the number of equivalent residential units each line will serve.

The sum of the costs for the 10 collection lines that are attributable to future development ($4,278,710) divided by the number of ERUs (8,820) equals the improvement fee for a single-family house on a 3/4" water meter, $485 per ERU.

Table 4
City of Estacada, Sewer Utility
Capital Improvements List and Cost Basis for Improvement Fee

	Cost (1994 $'s)	Total Acres	Zoning R1	R2	C1	M1	ERUs
Regan Hill Sewers	$714,005	120	120				480
Highway 224 East Interceptor	367,400	30	30				120
Coupland Road Interceptor	469,800	200	200				800
Duus Road Interceptor	399,400	1,000	600			300	4,200
Cemetery Road Pump Station	485,455	10	10				40
Timber Park Interceptor	297,250	1,300				300	1,800
Hinman Road Interceptor	446,900	200	100		100		600
Rivermill Road Interceptor	133,300	50				50	300
River Lake Road Sewers	965,200	100	60	40			480
Total	$4,278,710	3,010	1,120	40	100	650	
ERUs per Acre			4	6	2	6	
Total ERUs			4,480	240	200	3,900	8,820
Cost per ERU	$485						

Source: City of Estacada, Sanitary Sewer Distribution System Capital Improvement Program, prepared by Curran-McLeod, Inc., February 1994.

Table 5
City of Estacada, Sewer Utility
Summary, Systems Development Charge for Sewer Service

Meter Size	Equiv. Ratio	Reimbursement Fee	Improvement Fee	Total SDC
3/4"	1.00	$493	$485	$978
1"	1.78	876	862	1,738
1-1/4"	2.78	1,368	1,348	2,716
1-1/2"	4.00	1,970	1,940	3,911
1-3/4"	5.44	2,682	2,641	5,323
2"	7.11	3,503	3,450	6,953
3"	16.00	7,882	7,762	15,643
4"	28.44	14,012	13,799	27,811
6"	64.00	31,526	31,047	62,574
8"	113.78	56,047	55,195	111,242

Source: Raymond J. Bartlett, Economic & Financial Analysis.

As explained above, the number of ERUs will be determined by water meter size. The improvement fee will be allocated like the reimbursement fee by meter size. Table 5 shows the reimbursement fee, the improvement fee, and the combined SDC for sewer service by meter size.

Credits

The city financed the costs recovered by the reimbursement fee with a general obligation bond. Those undeveloped properties that have been within the city have been paying for the excess sewage treatment capacity through property taxes. The statutes authorizing the city to assess SDCs, requires the city to credit any proposed development for tax revenues used to provide the excess capacity. For this reason, the city will waive the reimbursement fee for any development proposed on land that has been within the city since 1990. For property annexed since 1990, the city will assess the reimbursement fee proportionate to the years inside the city. The bond term is 7 years, therefore each year since 1990 equals one-seven of the SDC. For example, if a property annexes in 1995, then 71 percent (i.e., debt service for 5 of the 7 years) of the reimbursement fee (and all of the improvement fee) would be owing. Once annexed the property would pay property taxes toward retirement of the bond that was used to purchase the excess capacity.

RESOLUTION 1994-6

A RESOLUTION ESTABLISHING A SYSTEM DEVELOPMENT CHARGE FOR THE WATER AND SANITARY SEWER SYSTEM AND ESTABLISHING AN APPEAL FEE.

WHEREAS, the Council of the City of Estacada has duly adopted Ordinance Series of 1994, No. 9 declaring their intent to comply with the provisions of ORS 223, an ordinance regarding systems development charges, and

WHEREAS, a methodology for the calculation of the system development charge for water and sanitary sewer has been developed as specifically described in Resolution 1994-5, and

WHEREAS, the Council has deemed it desirable to charge the legally allowable charges developed in the methodology, and

NOW, THEREFORE, BE IT RESOLVED that an appeal fee is hereby established as described therein.

SYSTEM DEVELOPMENT CHARGES

WATER UTILITY

Meter Size	Reimbursement Fee	Improvement Fee	Total
3/4"	NA	$1,461	$1,461
1"	NA	2,597	2,597
1-1/4"	NA	4,057	4,057
1-1/2"	NA	5,843	5,843
1-3/4"	NA	7,953	7,953
2"	NA	10,387	10,387
3"	NA	23,371	23,371
4"	NA	41,548	41,548
6"	NA	93,483	93,483
8"	NA	166,192	166,192

SEWER UTILITY

Meter Size	Reimbursement Fee	Improvement Fee	Total SDC
3/4"	$ 493	$ 485	$ 978
1"	876	862	1,738
1-1/4"	1,368	1,348	2,716

1-1/2"	1,970	1,940	3,911
1-3/4"	2,682	2,641	5,323
2"	3,503	3,450	6,953
3"	7,882	7,762	15,643
4"	14,012	13,799	27,811
6"	31,526	31,047	62,574
8"	56,047	55,195	111,242

APPEAL FEE

Pursuant to Section 2.600 of the City of Estacada Municipal Code, an appeal fee of $50.00 per appeal is hereby established.

Dave Vail

Dave Vail, Mayor

ATTEST:

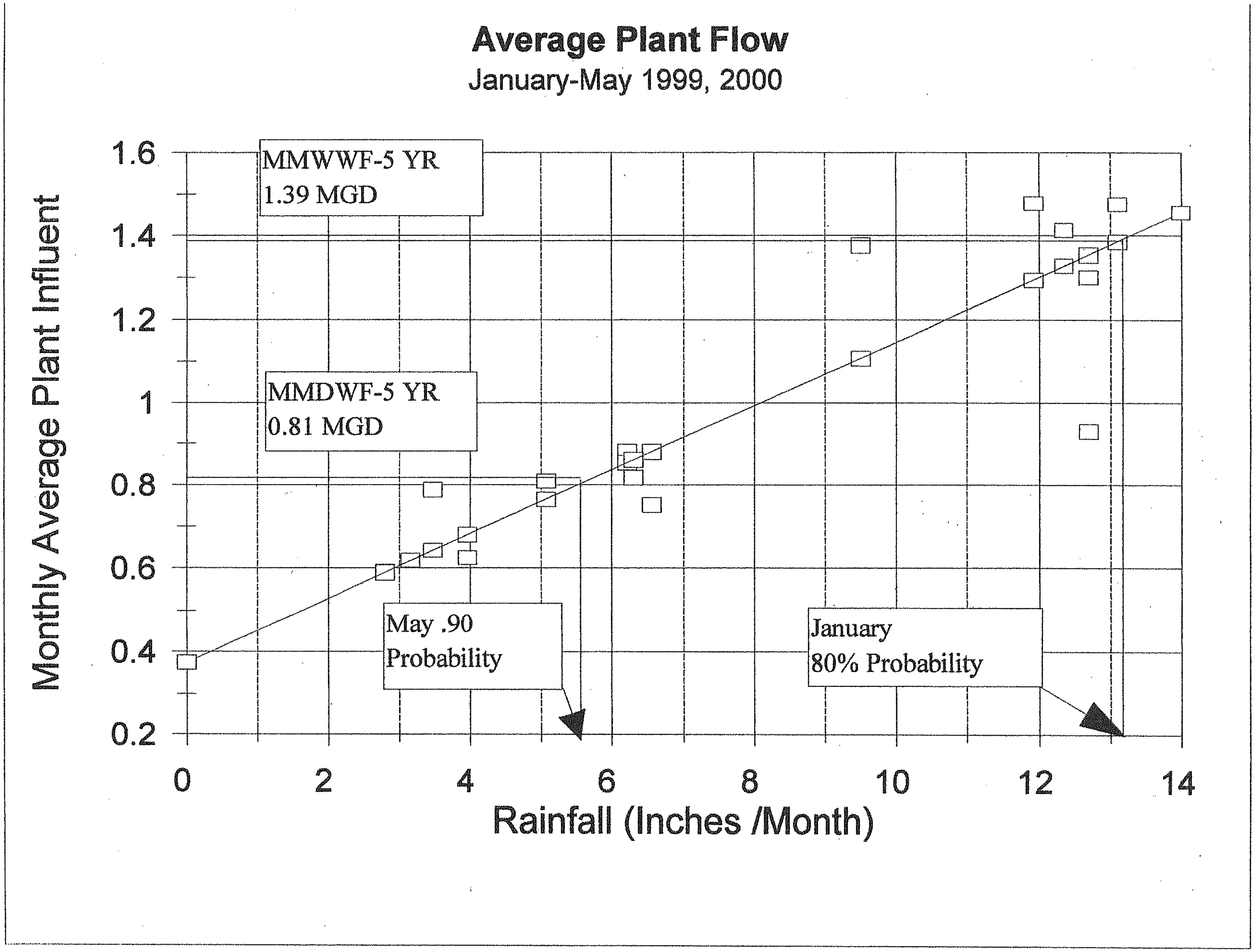
Denise Carey

Denise Carey, City Recorder

Date: 5/19/94

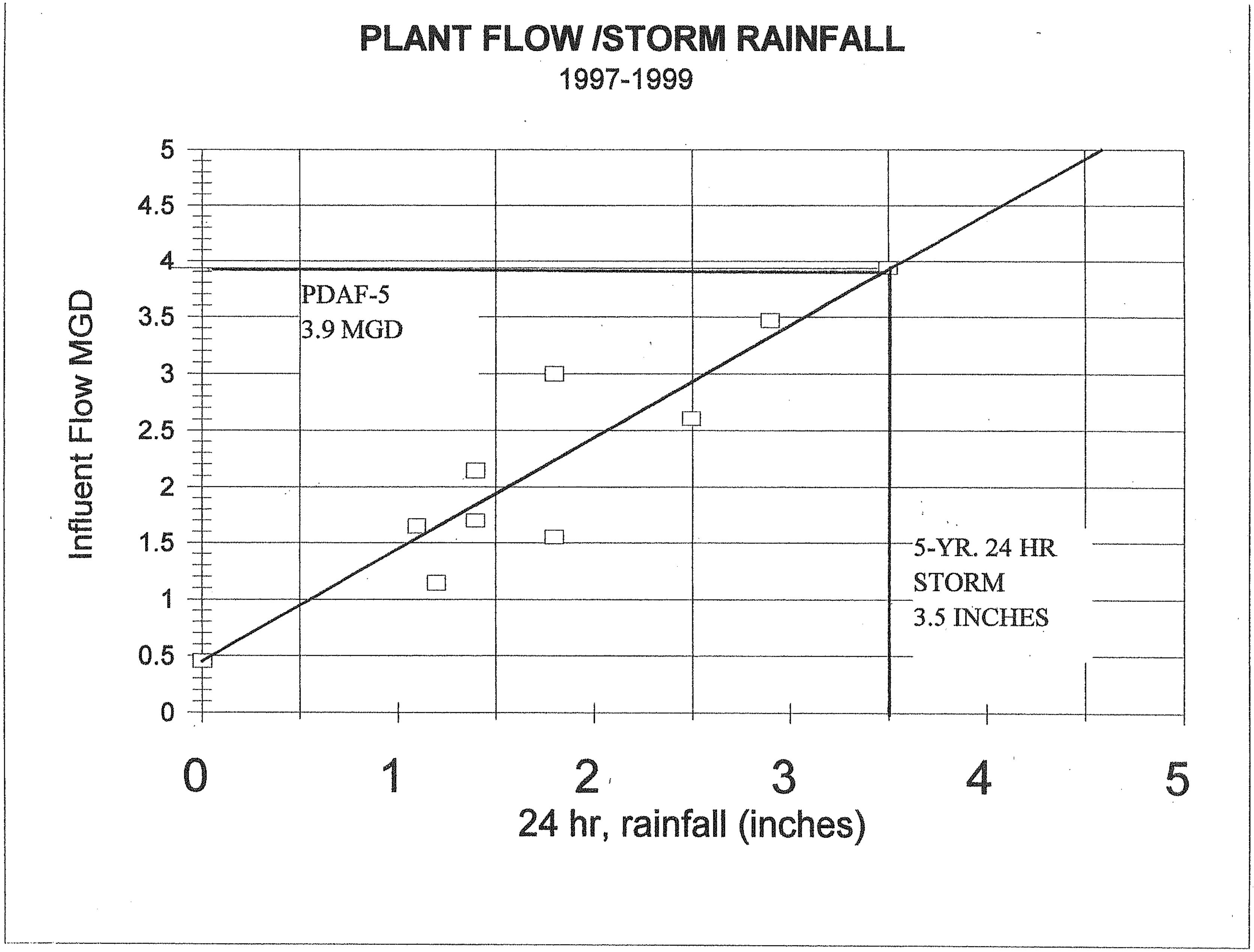
APPENDIX - V

TREATMENT PLANT FLOW ESTIMATES

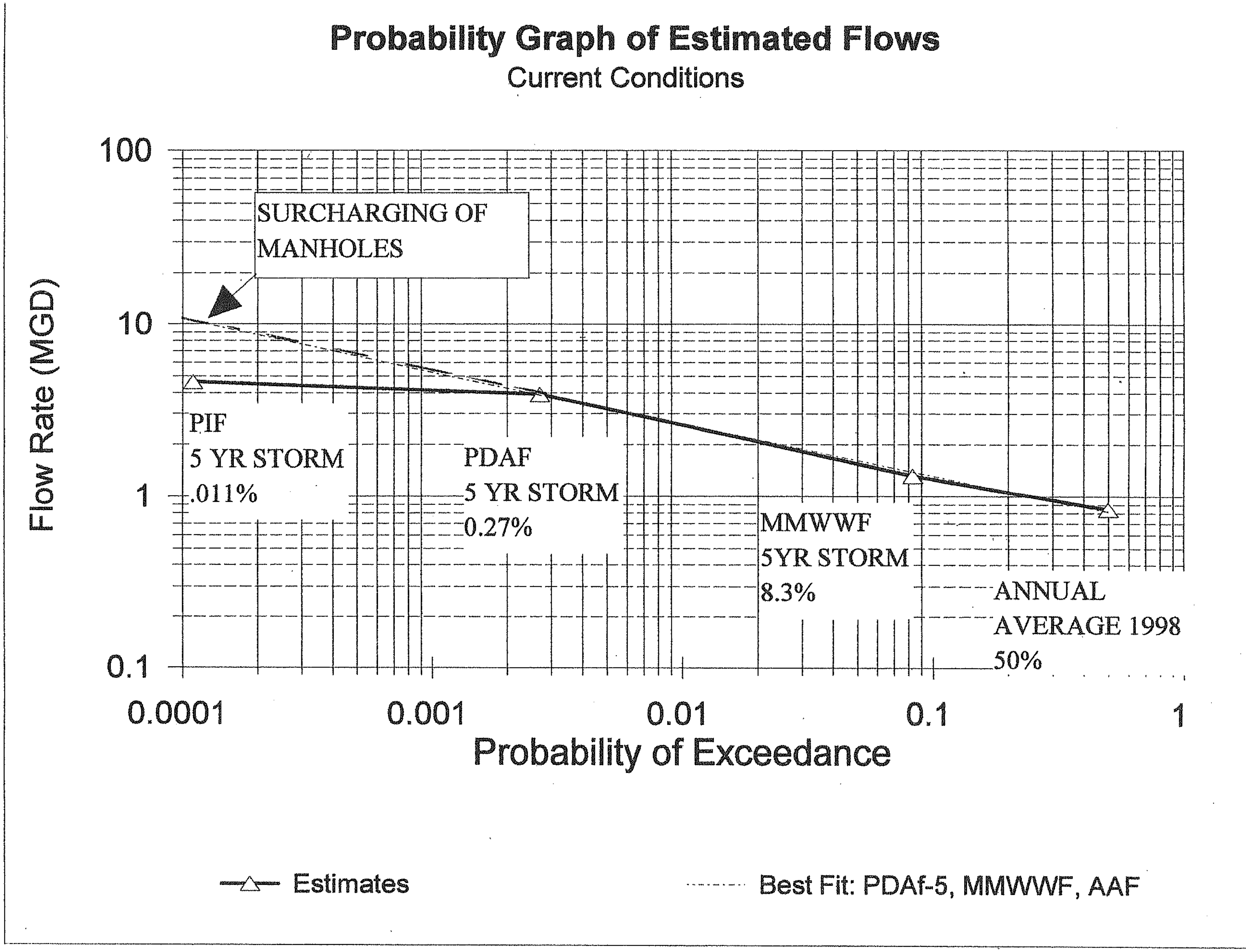


Average Plant Flow
January-May 1999, 2000
MMWWF-5 YR
1.39 MGD
MMDWF-5 YR
0.81 MGD
May .90
Probability
January
80% Probability
Monthly Average Plant Influent
Rainfall (Inches /Month)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
2
4
6
8
10
12
14

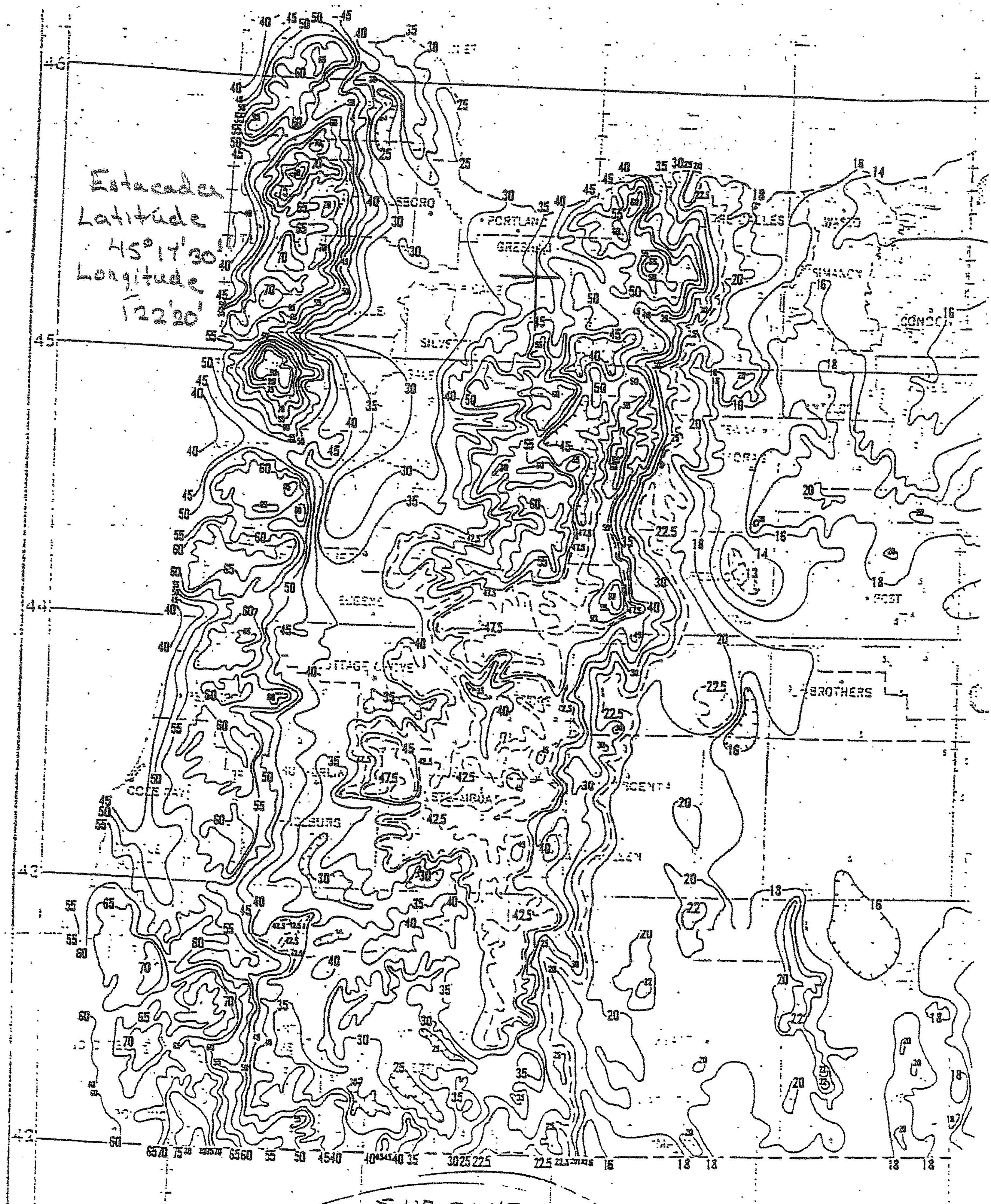
Month	Flow (MG)	MGD	Rainfall (Inches)			***Regression Statistics***						
January	43.796	1.41	12.36	1997								
Febuary	24.489	0.79	3.47			Multiple R	0.879202					
March	45.766	1.48	13.1			R Square	0.772995					
April	23.336	0.75	6.56			Adjusted R Square	0.754078					
May	14.912	0.48	3.16			Standard Error	0.175259					
January	45.827	1.48	11.93	1998		Observations	14					
Febuary	25.125	0.81	5.07									
March	27.307	0.88	6.22			***Analysis of Variance***						
April	19.368	0.62	3.96									
May	25.403	0.82	6.31				***df***	***Sum of S***	***Mean Squ***	***F***	***Significance F***	
January	42.727	1.38	9.5	1999		***Regression***	***1***	***1.255119***	***1.255119***	***40.86238***	***3.44E-05***	
Febuary	40.341	1.30	12.7			Residual	12	0.368589	0.030716			
March	28.876	0.93	12.7			Total	13	1.623708				
April	18.361	0.59	2.8									
		0.3749	0				***Coefficie***	***Standard***	***t Statistic***	***P-value***	***Lower 95.00***	
		1.4529	14			***Intercept***	***0.374972***	***0.105706***	***3.547304***	***0.003575***	***0.144658***	***0.605285***
							0.077208	0.012078	6.392369	2.37E-05	0.050892	0.103524


PLANT FLOW /STORM RAINFALL
1997-1999
Influent Flow MGD
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
PDAF-5
3.9 MGD
5-YR. 24 HR
STORM
3.5 INCHES
0
1
2
3
4
5
24 hr, rainfall (inches)

PDAF-5 Estimation				
	Rainfall	Flow	***Predicted Y***	
	inches /day	MGD	MGD	
01-Jan	2.9	3.47	3.41	
17-Jan	1.4	2.146	1.80	
30-Jan	2.5	2.608	2.98	
16-Jan	1.1	1.652	1.47	
27-Dec	3.5	3.938	4.06	
19-Feb	1.4	2.146	1.80	
23-Feb	1.4	1.7	1.80	
28-Feb	1.8	3	2.23	
07-March	1.2	1.15	1.58	
28-March	1.8	1.55	2.23	
	0	0.45		
	5	5.41		
		Regression Output:		
	Constant			0.456241
	Std Err of Y Est			0.452703
	R Squared			0.776515
	No. of Observations			10
	Degrees of Freedom			8
	X Coefficient(s)		0.989347	
	Std Err of Coef.		0.187652	


Probability Graph of Estimated Flows
Current Conditions
100
10
1
0.1
Flow Rate (MGD)
SURCHARGING OF MANHOLES
PIF
5 YR STORM
.011%
PDAF
5 YR STORM
0.27%
MMWWF
5YR STORM
8.3%
ANNUAL
AVERAGE 1998
50%
0.0001
0.001
0.01
0.1
1
Probability of Exceedance
Estimates
Best Fit: PDAf-5, MMWWF, AAF

									INCLUDING			
									Observations	***Predicted Y***		
-		log	Projected			Log			***Observations***	***Predicted Y***		
AAF	0.84	-0.07572	1.048	0.020361	0.5	-0.30103			1	-0.05107	0.889051	
MMWWF	1.32	0.120574	1.4	0.146128	0.083	-1.08092			2	0.122776	1.32671	
PDAF	3.9	0.591065	4.27	0.630428	0.0027	-2.56864			3	0.454409	2.847142	
PIF	4.6	0.662758	4.81	0.682145	0.00011	-3.95861			4	0.764254	5.811042	
									3-Point			
Regression Statistics									***Observations***	***Predicted Y***		
									1	-0.08971	0.813367	
Multiple R	0.962775								2	0.141902	1.386443	
R Square	0.926935								3	0.583729	3.834682	
Adjusted R Square	0.890402								PIF		10.42	
Standard Error	0.124295											
Observations	4											
Analysis of Variance												
									3-Point			
	df	***Sum of Squares***			***Mean Squ***	***F***	***Significance F***		***Regression Statistics***			
Regression	1	0.391993			0.391993	25.37286	0.037225					
Residual	2	0.030899			0.015449				Multiple R	0.998499		
Total	3	0.422892							R Square	0.997		
									Adjusted R Square	***0.994***		
	Coefficien	***Standard Error***			***t Statistic***	***P-value***	***Lower 95.***	***Upper 95.***	Standard Error	0.026542		
Intercept	-0.11818	0.107328			-1.10109	0.351277	-0.57997	0.343616	Observations	3		
x1	-0.22291	0.044254			-5.03715	0.015083	-0.41332	-0.0325				
									Analysis of Variance			
										df	***Sum of S***	***Mean Squ***
									Regression	1	0.234127	0.234127
									Residual	1	0.000704	0.000704
									Total	2	0.234832	
										Coefficien	***Standard***	***t Statistic***
									Intercept	-0.17911	0.026364	-6.79394
									x1	-0.29698	0.016291	-18.2301



OREGON

5-YR, 24-HR
STORMS IN
TENTHS OF
AN INCH

NOAA ATLAS 2, Volume X
Prepared by U.S. Department of Commerce
National Oceanic and Atmospheric Administration
National Weather Service, Office of Hydrology
Prepared for U.S. Department of Agriculture,
Soil Conservation Service, Engineering Division

CLIMATOGRAPHY OF THE UNITED STATES NO. 20

ESTACADA, OR

PERIOD: 1951-80
ELEVATION: 410 FT

CLIMATOLOGICAL SUMMARY

	TEMPERATURE (F)																PRECIPITATION TOTALS (INCHES)											
	MEANS			EXTREMES						MEAN NUMBER OF DAYS				DEGREE DAYS								SNOW			MEAN NUMBER OF DAYS		
										MAX		MIN															
	* DAILY MAXIMUM	* DAILY MINIMUM	* MONTHLY	RECORD HIGHEST	YEAR	DAY	RECORD LOWEST	YEAR	DAY	90 AND ABOVE	32 AND BELOW	32 AND BELOW	0 AND BELOW	* HEATING BASE 65	* COOLING BASE 65	* MEAN	* GREATEST MONTHLY	YEAR	GREATEST DAILY	YEAR	DAY	MEAN	MAXIMUM MONTHLY	YEAR	.10 OR MORE	.50 OR MORE	1.00 OR MORE
JAN	44.6	33.1	38.9	64+	65	15	7	57	27	0	2	14	0	809	0	9.37	16.69	53	3.80	72	20	2.8	13.1	54	15	6	3
FEB	49.6	35.6	42.6	71+	68	28	13+	79	2	0	0	8	0	627	0	6.10	16.04	61	3.22	61	09	.6	8.0	71	12	4	1
MAR	53.6	36.2	44.9	77+	66	25	22+	71	1	0	0	7	0	623	0	6.68	11.78	57	2.40	66	09	1.4	25.0	51	14	5	1
APR	60.0	39.1	49.6	90	57	29	26+	51	21	0	0	2	0	462	0	4.59	8.93	55	2.18	58	20	.0	.0		11	3	1
MAY	67.0	43.5	55.3	98	73	13	31+	64	2	0	0	0	0	301	0	3.54	6.51	60	1.47	63	05	.0	.0		9	2	0
JUN	72.7	48.2	60.5	101+	51	29	35+	76	3	1	0	0	0	154	19	2.45	5.85	54	2.26	69	23	.0	.0		6	1	0
JUL	79.9	50.9	65.5	107+	56	19	38+	62	2	4	0	0	0	61	76	.74	2.44	74	.99	66	02	.0	.0		2	0	0
AUG	78.6	50.9	64.8	105+	67	9	40+	65	30	3	0	0	0	75	68	1.56	6.28	68	1.82	54	19	.0	.0		4	1	0
SEP	73.5	47.8	60.7	103+	55	4	32+	65	17	2	0	0	0	153	24	2.53	5.75	59	1.84	72	21	.0	.0		5	2	0
OCT	61.3	43.0	52.2	88+	70	2	15	69	13	0	0	1	0	397	0	5.06	11.20	55	3.63	55	09	.0	.0		9	4	1
NOV	51.3	38.0	44.6	69+	70	1	8+	55	15	0	0	6	0	612	0	7.95	15.87	73	3.31	60	24	.3	5.0	77	14	6	2
DEC	46.4	34.9	40.6	64	58	2	8+	72	8	0	1	10	0	756	0	9.38	17.50	64	3.04	64	21	1.5	16.8	68	16	6	2
YEAR	61.5	41.8	51.7	107	JUL 56	19	7	JAN 57	27	10	3	48	0	5030	187	59.95	17.50	DEC 64	3.80	JAN 72	20	6.6	25.0	MAR 51	117	40	11

*FROM 1951-80 NORMALS

ESTIMATED VALUE BASED ON DATA FROM SURROUNDING STATIONS

\+ ALSO ON EARLIER DATES.

DEGREE DAYS TO SELECTED BASE TEMPERATURES (F)

BASE	HEATING DEGREE DAYS												
BELOW	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
65	809	627	623	462	301	154	61	75	153	397	612	756	5030
60	654	487	468	312	160	62	12	17	64	242	462	601	3541
57	561	403	375	228	95	26	0	6	28	157	372	508	2759
55	499	351	313	174	62	13	0	0	15	106	315	446	2294
50	358	223	175	73	12	0	0	0	0	25	179	297	1342

BASE	COOLING DEGREE DAYS												
ABOVE	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
55	0	0	0	12	71	178	326	304	186	19	0	0	1096
57	0	0	0	6	42	131	267	248	139	8	0	0	841
60	0	0	0	0	15	77	182	166	85	0	0	0	525
65	0	0	0	0	0	19	76	68	24	0	0	0	187
70	0	0	0	0	0	0	17	15	0	0	0	0	32

DERIVED FROM THE 1951-80 MONTHLY NORMALS

PROBABILITY THAT THE MONTHLY PRECIPITATION WILL BE EQUAL TO OR LESS THAN THE INDICATED PRECIPITATION AMOUNT

MONTHLY PRECIPITATION (INCHES)

PROBABILITY LEVELS	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
.05	2.72	2.44	2.79	1.84	1.40	.57	.00	.00	.34	1.36	2.31	3.85
.10	3.61	3.00	3.39	2.26	1.72	.80	.00	.05	.67	1.84	3.06	4.71
.20	4.95	3.80	4.25	2.86	2.19	1.15	.14	.23	1.10	2.57	4.20	5.92
.30	6.11	4.46	4.95	3.35	2.57	1.48	.27	.44	1.45	3.22	5.18	6.92
.40	7.24	5.08	5.61	3.82	2.94	1.80	.40	.69	1.81	3.85	6.15	7.85
.50	8.42	5.71	6.28	4.29	3.31	2.14	.54	1.00	2.19	4.51	7.15	8.81
.60	9.72	6.39	7.00	4.80	3.71	2.52	.71	1.38	2.61	5.24	8.25	9.83
.70	11.25	7.18	7.83	5.40	4.17	2.97	.93	1.87	3.12	6.11	9.56	11.02
.80	13.24	8.18	8.88	6.15	4.76	3.57	1.22	2.58	3.78	7.24	11.24	12.51
.90	16.35	9.71	10.48	7.29	5.65	4.52	1.71	3.79	4.83	9.01	13.89	14.80
.95	19.24	11.10	11.93	8.34	6.47	5.41	2.19	5.01	5.83	10.67	16.35	16.88

THESE VALUES WERE DETERMINED FROM THE INCOMPLETE GAMMA DISTRIBUTION.

DEPARTMENT OF ENVIRONMENTAL QUALITY (DEQ)

Oregon
John A. Kitzhaber, M.D., Governor

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Northwest Region Portland Office
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Portland, OR 97201-4987
(503) 229-5263
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June 6, 2000

Bill Strawn, Superintendent
Estacada Public Works
City of Estacada
PO Box 958
Estacada, Or 97023

RE: Estacada WQ
Acceptance of Facility Plan Update
City of Estacada
File No. 27866
Clackamas County

Dear Bill Strawn:

Thank you for sending revisions to the facility plan update, which we received last week from Curran-McLeod, Inc. Minor additional revisions are being completed this week, prior to final printing. Based on these revisions, we are accepting the update as final. Please send one copy of the final report to my attention.

The update includes flow projections to 2020, a comprehensive description of the existing wastewater system, and evaluates various options for the future. The flow projections and evaluation of existing facilities are outstanding, and the overall report is excellent. We would like to thank the city for sponsoring a report of this quality.

We greatly appreciate the city's initiative in undertaking this update.

Respectfully,

David S. Mann, P.E.
Senior Environmental Engineer
Northwest Regional Water Quality

cc:
Susan Foreman EIT, Curran-McLeod, Inc., 6655 SW Hampton St. #210,
Portland 97223

Lyle Christensen, NWR
Robert Baumgartner, Manager
ec:
DEQ regional sanitary engineers