CORVALLIS
ENHANCING COMMUNITY LIVABILITY

CITY OF CORVALLIS

Wastewater Utility Master Plan

REGISTERED PROFESSIONAL ENGINEER 10511 OREGON JULY 13, 1979 GORDON A. NICHOLSON

10/98

CH2MHILL

NOVEMBER 1998

Type	Rate
Per capita, wet season, dry weather	193 gallons per capita per day--peak rate
Per acre, wet season, wet weather	4,000 gallons per acre per day--peak for 5-year, 24-hour event

*2.14 population per EDU assumed for Master Planning purposes

ZONE	Dwelling Units per Acre
AG-OS	-
CB	4.4
CBF	4.4
CS	4.4
GI	7.4
GI	8.7
II	8.7
LC	4.4
LI	4.4
OSU	
P-AO	4.4
PD(12U)	16
PD(CS)	4.4
PD(GI)	8.7
PD(LC)	4.4
PD(LI)	8.4
PD(P-AO)	4.4
PD(RS-12)	12.0
PD(RS-12)	16.0
PD(RS-12U)	12.0
PD(RS-12U)	16.0
PD(RS-20)	25.0
PD(RS-3.5)	4.0
PD(RS-5)	4
PD(RS-6)	4.0
PD(RS-9)	9.0
PD(RTC)	4.4
PD(SA)	4.35
RS-12	12.0
RS-12	16.0
RS-12U	12.0
RS-20	25
RS-3.5	4
RS-5	4
RS-6	4.0
RS-9	9
RTC	4.4
SA	4.35
SAU	4.35
SSD	4.4

Land Use	Dwelling Units per Acre
Res-Low	4.0
Res-Medium	9.0
Res-MH	12-16
Commercial (SA)	4.4
Ind-Limited	8.7
Ind-Intensive	7.4

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Executive Summary

The City of Corvallis Wastewater Utility encompasses more than 200 miles of sewer pipe, six major pump stations, and one regional wastewater reclamation plant. The fiscal year 1997/98 operating and capital budget was $11.8 million. The 1998 5-year Capital Improvement Program for the utility exceeds $42 million, which includes the combined sewer overflow remediation project. Preparation of a wastewater utility master plan is necessary for the efficient planning and implementation of utility improvements that are protective of human health, the environment, and comply with all regulatory requirements.

The collection system and treatment plant are in good condition, competently operated, and meet Oregon Department of Environmental Quality (DEQ) and U.S. Environmental Protection Agency (EPA) discharge permit requirements. There are some collection system piping limitations and a few customer concerns. The Wastewater Utility is fiscally conservative, and offers reasonable rates to its customers.

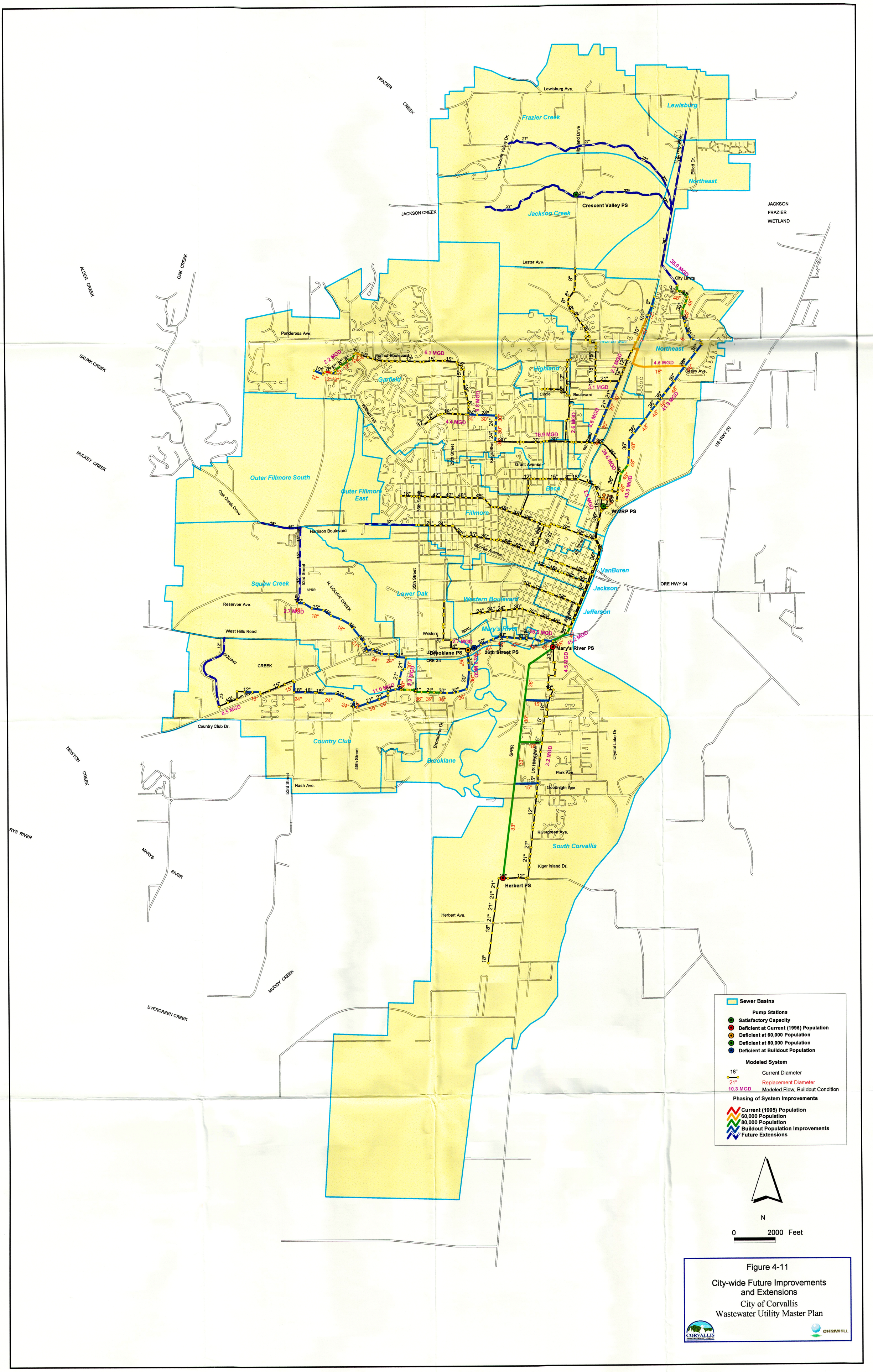
Since the late 1970s, the era when the last master planning effort was completed, the number of individuals and businesses served by the Wastewater Utility has increased by more than 25 percent, City land uses have changed, and environmental regulations have become more comprehensive. As a result, a new master plan was needed. The planning effort was initiated with the following objectives:

1. Develop plans to correct existing collection system and treatment system deficiencies
2. Plan for the orderly extension of utility services in the urban growth boundary (UGB) consistent with the City's Comprehensive Plan
3. Ensure wastewater collection and treatment is protective of public health and the environment
4. Maintain full compliance with all DEQ and EPA regulations
5. Develop plans to meet potentially more stringent future regulatory requirements
6. Obtain community input and guidance in developing the master plan

The desired outcome of the master planning effort is the preparation of a comprehensive plan protective of public health and the environment that is supported by the community and is the basis of the Wastewater Utility Capital Improvement Program.

Planning Criteria

Land use, population, and regulatory planning criteria were established to guide the development of the plan. Current and future land use and population criteria were obtained from the City's latest Comprehensive Plan. Population planning horizons of 60,000, 80,000, and probable buildout (120,000) were used for the master planning effort. As a result, the master plan is consistent with other infrastructure planning documents such as the water utility master plan and transportation plan.


Sewer Basins
Pump Stations
Satisfactory Capacity
Deficient at Current (1995) Population
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Deficient at 80,000 Population
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0 2000 Feet
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WWRP PS
Brooklane PS
26th Street PS
Mary's River PS
Herbert PS
Frazier Creek
Jackson Creek
Lewisburg
Northeast
Highland
Garfield
Outer Fillmore South
Outer Fillmore East
Fillmore
Beca
Squaw Creek
Lower Oak
Western Boulevard
Mary's River
VanBuren
Jackson
Jefferson
Country Club
Brooklane
South Corvallis
JACKSON FRAZIER WETLAND
FRAZIER CREEK
JACKSON CREEK
ALDER CREEK
OAK CREEK
SKUNK CREEK
MULKEY CREEK
NEWTON CREEK
MARYS RIVER
MUDDY CREEK
EVERGREEN CREEK
SQUAW CREEK
Lewisburg Ave.
Lester Ave.
Ponderosa Ave.
Walnut Boulevard
Harrison Boulevard
Monroe Avenue
Reservoir Ave.
West Hills Road
Country Club Dr.
Nash Ave.
Grant Avenue
Park Ave.
Goodnight Ave.
Rivergreen Ave.
Kiger Island Dr.
Herbert Ave.
Seavy Ave.
City Limits
Elliott Dr.
Crescent Valley Dr.
Highland Drive
Circle
Boulevard
Kings Blvd.
25th Street
35th Street
45th Street
53rd Street
Brooklane Dr.
Crystal Lake Dr.
US HWY 20
ORE HWY 34
SPRR
N. SQUAW CREEK
Figure 4-11
City-wide Future Improvements and Extensions
City of Corvallis
Wastewater Utility Master Plan
CORVALLIS
CH2MHILL

The Wastewater Utility existing service area is approximately 5,495 acres. Ultimately 15,463 acres will be served when the urban growth boundary area is fully developed. The Corvallis urban growth boundary, sewer basins, and limits of current wastewater service (city limits) are shown in Figure ES-1. The current population served by the Wastewater Utility is 49,275 persons. About 1,386 acres compose a combined sewer area, conveying both sanitary sewage and stormwater in a common pipe to the wastewater reclamation plant. Sanitary sewage collected from the balance of the service area is not combined with stormwater. From these separated areas, sanitary sewage is conveyed to the wastewater reclamation plant and treated before being discharged to the Willamette River. Stormwater from these areas is discharged directly to Corvallis urban streams and/or the Willamette River.

A portion of the stormwater, however, does enter the separated area sanitary sewage pipes because of defects in the sanitary sewage pipes or illegal cross connections. This extraneous stormwater that enters into the sanitary sewage pipes is termed "infiltration and inflow." The City has an ongoing program to control infiltration and inflow from entering the pipelines in separated sewered areas.

Community Outreach

During preparation of the master plan, community input was sought via many varied outreach efforts. Two community meetings, guidance from the Water Infrastructure Committee (WIC) during its existence, City Urban Services Committee briefings, and periodic City Council presentations were conducted. Public meetings were advertised in the Gazette Times, TCI community channel, various City publications, and extensive direct mailings were sent to potentially interested individuals or organizations. The goal of these outreach activities was to obtain community input and feedback to help guide the master planning effort and ensure it was responsive to community concerns and values. The feedback received has been incorporated into the plan. In addition, the City is implementing solutions to specific comments received from citizens on their individual level of wastewater service.

Regulatory Requirements

Regulatory criteria that guided development of the plan were based on all current DEQ and EPA applicable regulations. A critical DEQ regulatory criterion was the establishment of a design storm with a 1-in-5-year return frequency. This identified the intensity and pattern of the type of storm that had a probability of occurring once every 5 years. The storm was used to assess collection system conveyance capacity, the potential for sanitary sewer system overflows (SSOs), and wastewater reclamation plant discharge permit compliance. The design storm adopted for the master planning effort is consistent with the methodology used for the approved *Combined Sewer Overflow Remediation Plan*. The methodology for developing the design storm is also consistent with the approach used for other Willamette Valley communities and approved by DEQ.

Collection System Modeling

An inventory of the existing wastewater collection system was compiled and used to expand the City's computer hydraulic model of the system. The City's hydraulic model was initially developed for the combined sewer overflow project. Collection system flow monitoring information collected in 1996 and wastewater reclamation plant flow records were used to calibrate the model. The first hand knowledge and experience of Utility personnel augmented the calibration efforts. Once calibrated, hydraulic model runs for the existing (1995) population of 47,485, 60,000, 80,000 and probable buildout (120,000) projections were performed to identify collection system deficiencies.

Figure ES-2 shows the portions of the collection system that were modeled and the system deficiencies that were identified for 80,000 population conditions. These deficiencies include those currently existing and those that will progressively develop as the area population increases. The hydraulic model was then used to evaluate various improvement alternatives to alleviate the deficiencies. The model was also used to identify the cost-effective level of infiltration and inflow (I/I) correction for the system. Maintaining I/I at the current level was found to be necessary and most cost-effective. This is commonly done by regular inspection, repair, and maintenance of the system and the removal of known inflow sources to the separated system.

Collection System Improvements

Figure ES-3 shows the recommended improvements to alleviate existing collection system deficiencies and those that will arise when the area population increases to 60,000, and then 80,000. In addition to ongoing annual wastewater collection system maintenance, rehabilitation, and replacement, collection system improvements having a total project cost of $26.8 million (1998 dollars) are necessary to serve a planning horizon population of 80,000. Projects will be phased as demand increases. Projects to serve existing and 60,000 population demands total $3.5 million and $5.3 million respectively. These amounts are included in the $31 million. Collection system improvements are needed for relief sewers and pump station upgrades or replacements. Extensions of sewers into currently unsewered areas are not included in the cost and are shown in Section 4 for each sewer basin. The timing and cost of sewer extensions will depend on the rate and location of population growth within the City's UGB. The short-term projects improvement plan for collection system improvements is $5.0 million (1998 dollars). This plan is analogous to the 5-year Capital Improvement Plan.

Treatment Plant Assessment

Concurrent with the collection system master planning work, a condition assessment of the wastewater reclamation plant (WWRP) was performed. The assessment consisted of analyzing 5 years (October 1991-96) of plant influent, effluent, and treatment performance data, and reviewing the physical condition and adequacy of treatment processes. Treatment plant performance has been outstanding over the time period. With one exception, the plant has not exceeded any National Pollutant Discharge Elimination System (NPDES) permitted effluent limit during this period. A permit exceedance occurred in 1991 as a result of a

process upset that occurred concurrent with construction activities, which compounded the upset effects.

Since mid-1992, the plant's performance has been perfect. The plant's outstanding performance recently garnered it national recognition from the Association of Metropolitan Sewerage Agencies. Using plant records, the capacity of each treatment process, either liquid, hydraulic, or solids handling processes, was estimated. Plant records also provided the basis for projecting future influent flow and wasteload projections based on a per capita basis. The analysis of historical flow values was augmented by hydraulic modeling results. The existing WWRP facilities along with the CSO wet weather treatment facilities currently under construction are shown in Figure ES-4.

Short-Term Treatment Improvements

The plant condition assessment provided the basis for identifying potential short-term improvements. It was determined that the most significant short-term improvement needed was additional solids handling capacity. The last major solids handling improvement occurred with the construction of a primary digester in the late 1960's and the facultative sludge lagoons in 1979. Additional solids handling capacity is needed to accommodate population growth, the combined sewer overflow project, redundancy, and maintenance of the existing solids handling facilities. Given the solids handling improvements short-term need and with the combined sewer overflow facilities scheduled for start-up in 2000, the City determined it would be desirable to begin implementing the improvements before completion of the Master Plan. Design of the solids handling improvements is underway. Careful planning is occurring to ensure the improvements continue to be consistent with the master plan recommendations. A seismic hazard evaluation conducted as part of this planning effort identified potential corrective actions that should be implemented in the short-term after further analysis, ranking by priority, and development of cost estimates. Further seismic analysis is recommended next fiscal year.

Long-Term Treatment Alternatives

Long-term WWRP improvement alternatives to serve a population of 60,000, 80,000, and probable buildout (120,000) were evaluated. In addition to serving increasing population levels, alternatives to provide a higher level of treatment performance were evaluated. Higher levels of treatment were evaluated in light of the probability of more stringent regulatory requirements.

Three planning scenarios were evaluated for the liquid and hydraulic processes, solids handling processes, and environmental adjacency issues. The first planning scenario represented existing regulatory requirements and environmental mitigation measures. The second and third planning scenarios represented increasingly more stringent conditions. The second scenario, termed the Most Probable Scenario, is reflective of the more stringent conditions anticipated to be required during the master planning period. The third scenario, termed Most Stringent Scenarios, was the most stringent planning condition and was assumed not likely to be required but was nonetheless prudent to plan for.

NPDES permit conditions associated with each planning scenario can be generalized as follows: existing secondary effluent permit conditions (Existing Conditions), advanced secondary treatment (Most Probable Scenario), and tertiary treatment (Most Stringent Scenario). Each scenario assumed a greater removal of conventional pollutants being required. Advanced secondary and tertiary scenarios also assumed the removal of nutrients that may be detrimental to the receiving water quality and aquatic environment under certain circumstances; specifically ammonia and nitrogen and phosphorus respectively. Planning for more stringent effluent quality requirements is prudent given increased awareness and concern over the health of the Willamette River.

Solids handling scenarios consisted of increasing levels of solids stabilization, pathogen reduction, and restrictions on the land application of biosolids. Environmental adjacency criteria addressed the areas of air quality (principally objectionable odors), noise, safety, visual (aesthetic), and traffic. Each of these items, if uncontrolled, can create objectionable impacts to adjacent neighbors and/or plant operating personnel. Therefore, increasingly more stringent mitigation measures to limit the impacts of the wastewater reclamation plant operations in each of these areas was evaluated.

Assuming the Most Probable planning scenario, the projected cost of all the wastewater reclamation plant improvements to serve a planning horizon scenario of 80,000 population is $43 million (1998$). Build-out of WWRP site, assuming the most probable scenario, is shown in Figure ES-5. The estimated cost of short-term WWRP improvements consisting of the solids handling improvements, associated odor control features, and non-process facilities is $4.3 million (1998$). Should the Existing Conditions planning scenario prevail, improvement costs will be lower. Conversely, should the Most Stringent planning scenario be required, project costs would increase by 100 to 200 percent.

Cost Estimates and Schedule

Table ES-1 presents the total capital cost of collection system and WWRP improvements for short-term, 60,000, and 80,000 population planning criteria. These are order-of-magnitude estimates. Final costs will vary depending on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other factors. Figure ES-6 is the recommended schedule for implementing the short-term improvement projects. The costs shown assume the Most Probable planning scenarios for the wastewater reclamation plant.

Policy Recommendations

In addition to identifying utility system improvements and funding requirements, the Master Plan also identified utility policy and management activities necessary to achieve the plan objectives. Many necessary policies and management practices are currently in place. In which case, recommendations to further enhance the items to better achieve the plan objectives have been made. Seventeen new policies and/or management activities have been recommended. Of particular importance is the need for the City to maintain and enhance the ongoing program to rehabilitate existing sewer pipes and control infiltration and inflow. Infiltration and inflow from existing sewered areas must be controlled to

current levels. Whereas infiltration and inflow from new developed areas must be limited through new construction and inspection practices. Given the importance of controlling infiltration and inflow, the establishment of a dedicated funding source is recommended.

TABLE ES-1
Capital Project Summary

	Total Project Costs[a]			
Projects	**Short-Term CIP**	**Population 60,000**	**Population 80,000**	**Total (All Phases)**
Collection System Projects				
Marys River Pump Station	$3,320,000	$340,000	$510,000	$4,170,000
Brooklane Pump Station		$3,320,000	$490,000	$3,810,000
South Corvallis Parallel Intercept & Diversions			$12,410,000	$12,410,000
Herbert Pump Station High Flow Bypass	30,000			$30,000
Marys River Interceptor			$1,000,000	$1,000,000
Country Club Trunk			$1,200,000	$1,200,000
North Ninth Street Interim Grinder Pump Station	40,000			
North Ninth Street Diversion		$1,570,000[b]		$1,570,000
Garfield Relief Sewers	$110,000	$90,000	$330,000	$530,000
Crescent Valley Interceptor			$2,300,000	$2,300,000
Subtotal Collection System Projects	***$3,500,000***	***$5,320,000***	***$18,240,000***	***$26,820,000***
WWRP Projects				
Influent Pumping/Headworks			$2,606,200	$2,606,200
Primary Treatment		$3,000,000	$1,687,500	$4,687,500
Biological Treatment		$406,200	$3,662,500	$4,068,700
Secondary Clarification		$4,237,500	$2,250,000	$6,487,500
Disinfection		$2,380,000		$2,380,000
Filtration			$10,856,300	$10,856,300
Effluent Pumping			$1,900,000	$1,900,000
Solids Handling	$4,150,000		$4,300,000	$8,450,000
Odor Control		$968,800	$150,000	$1,118,800
Non Process Facilities	$179,100	$221,000	$187,500	$587,600
Subtotal WWRP Projects	***$4,329,100***	***$11,213,500***	***$27,600,000***	***$43,142,600***
Total Project Cost	**$7,829,100**	**$16,533,500**	**$45,640,000**	**$69,962,600**

[a] Cost estimates include 25 percent construction contingency and 25 percent allowance for engineering, legal, and administrative costs. They do not include seismic improvements. Cost estimates are in 1998 dollars indexed to the *Engineering News Record* construction costs index of 6500 for the Seattle area.

[b] Postponement of diversion to potentially 60,000 population is dependent on the installation of an interim grinder pump station to serve a single business.

Conclusions

The major conclusions and recommendations of the master plan are as follows:

Collection System

- Through an increased commitment to the City's I/I control and sewer replacement program, control existing I/I to current levels. This will prevent further deterioration of the collection system and any resulting increase in I/I. Integral to controlling existing I/I should the City's evaluation of implementing a private lateral replacement program.
- Similarly, the City must ensure that I/I from collection system extensions and new private construction is minimized by continually improving building standards and inspections.
- Collection system improvements are necessary to eliminate sanitary sewer system overflows by 2010 as required by DEQ. Expansion of the Marys River pump station is a short-term priority project necessary to meet this regulatory requirement.
- Collection system improvements are necessary to eliminate collection system deficiencies that will cause system surcharging and flooding. The short-term priority collection deficiency project is a North Ninth Street Diversion or interim grinder pump station to avoid flooding of a selected business. Short-term improvements projects are also required at the Herbert pump station and selected locations in the Garfield basin.

Wastewater Reclamation Facility

- Continue implementing the short-term solids handling improvements to ensure the new facilities are operational together with the new CSO remediation facilities at the end of 2000.
- Periodically assess the possible application of new emerging technologies, i.e., disinfection alternatives and digester gas utilization.
- Incorporate environmental adjacency improvements in WWRP projects to continually enhance the facility's "good neighbor" image.
- When it is necessary to expand the WWRP secondary process, critically examine current and emerging regulatory requirements as they may significantly influence the preferred expansion alternative

Regulatory

- Monitor and participate in the development of new State and Federal regulatory requirements affecting the utility. Follow in particular, temperature, disinfection and Willamette River specific water quality issues for the WWRP.

General

- Continue the wastewater utility's proactive public outreach program to ensure the services provided are responsive to and align with the community values of being protective of human health and the environment.

- Periodically review and update the master plan to account for changes in population growth patterns or distribution and regulatory requirements. Also, periodically update to ensure the master plan remains consistent with other city planning documents. Expertise developed by utility staff in the use of the collection system hydraulic model should help facilitate the periodic updates.

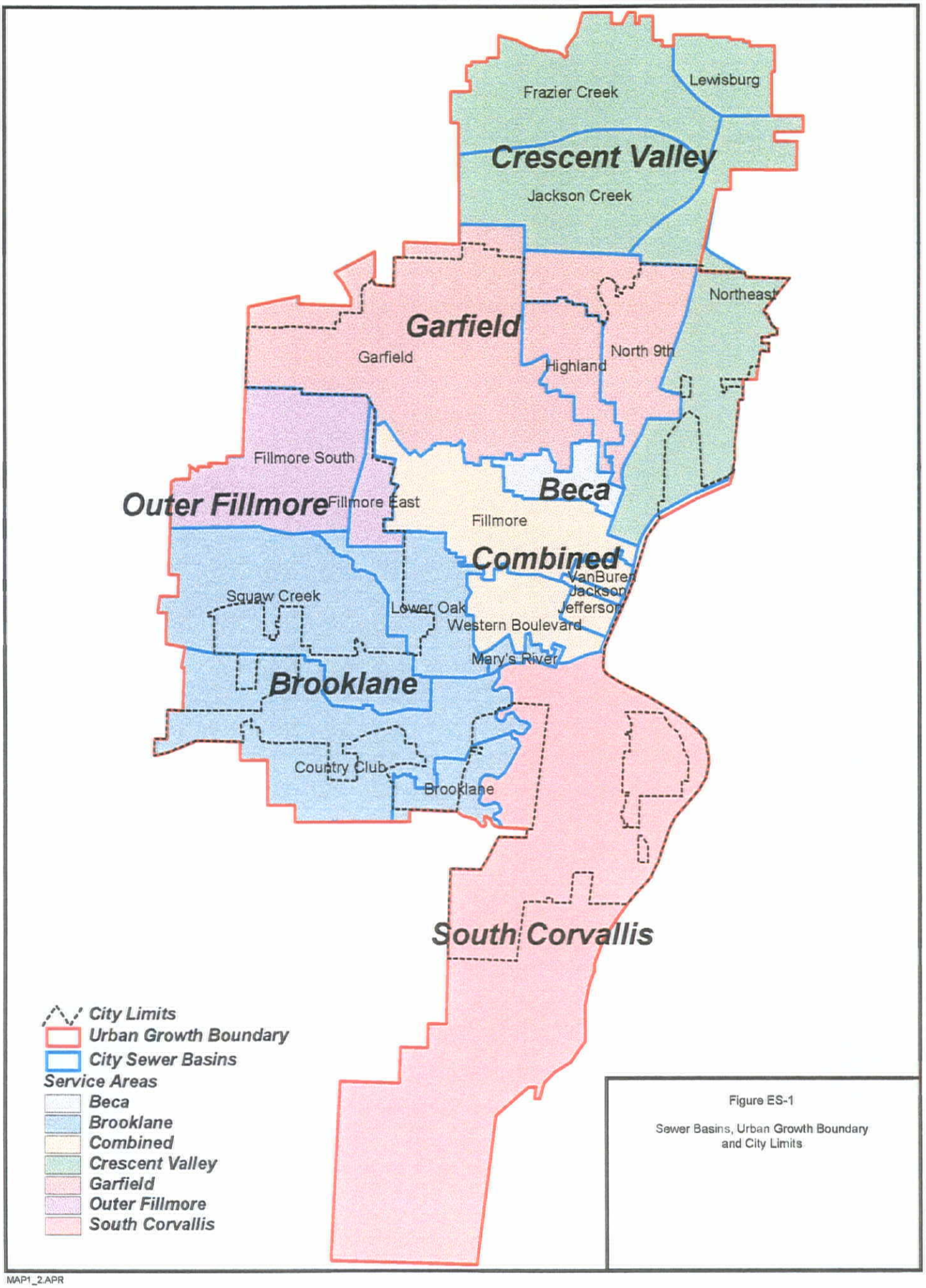

Figure ES-1

Sewer Basins, Urban Growth Boundary and City Limits

MAP1_2.APR

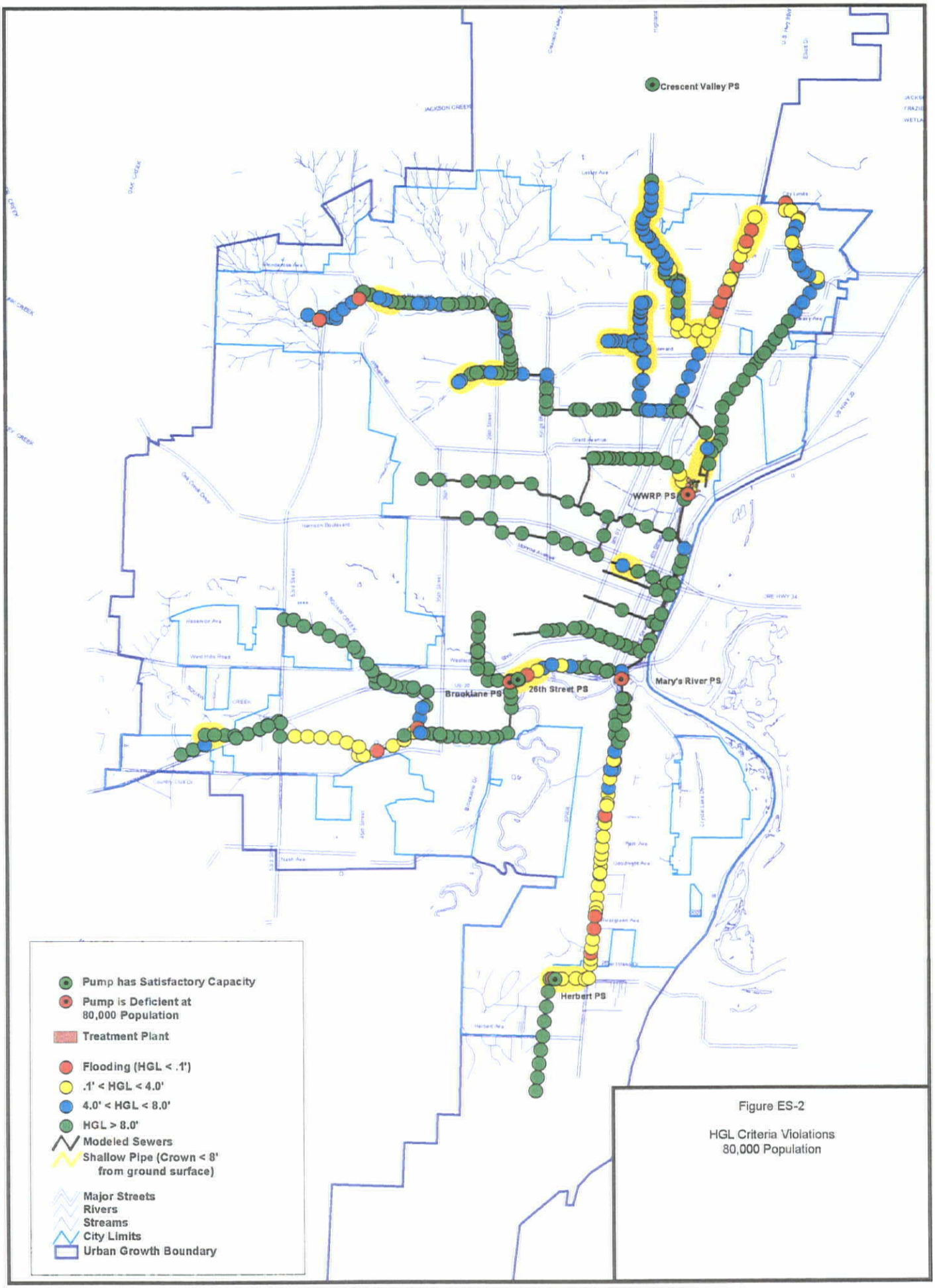

Figure ES-2

HGL Criteria Violations
80,000 Population

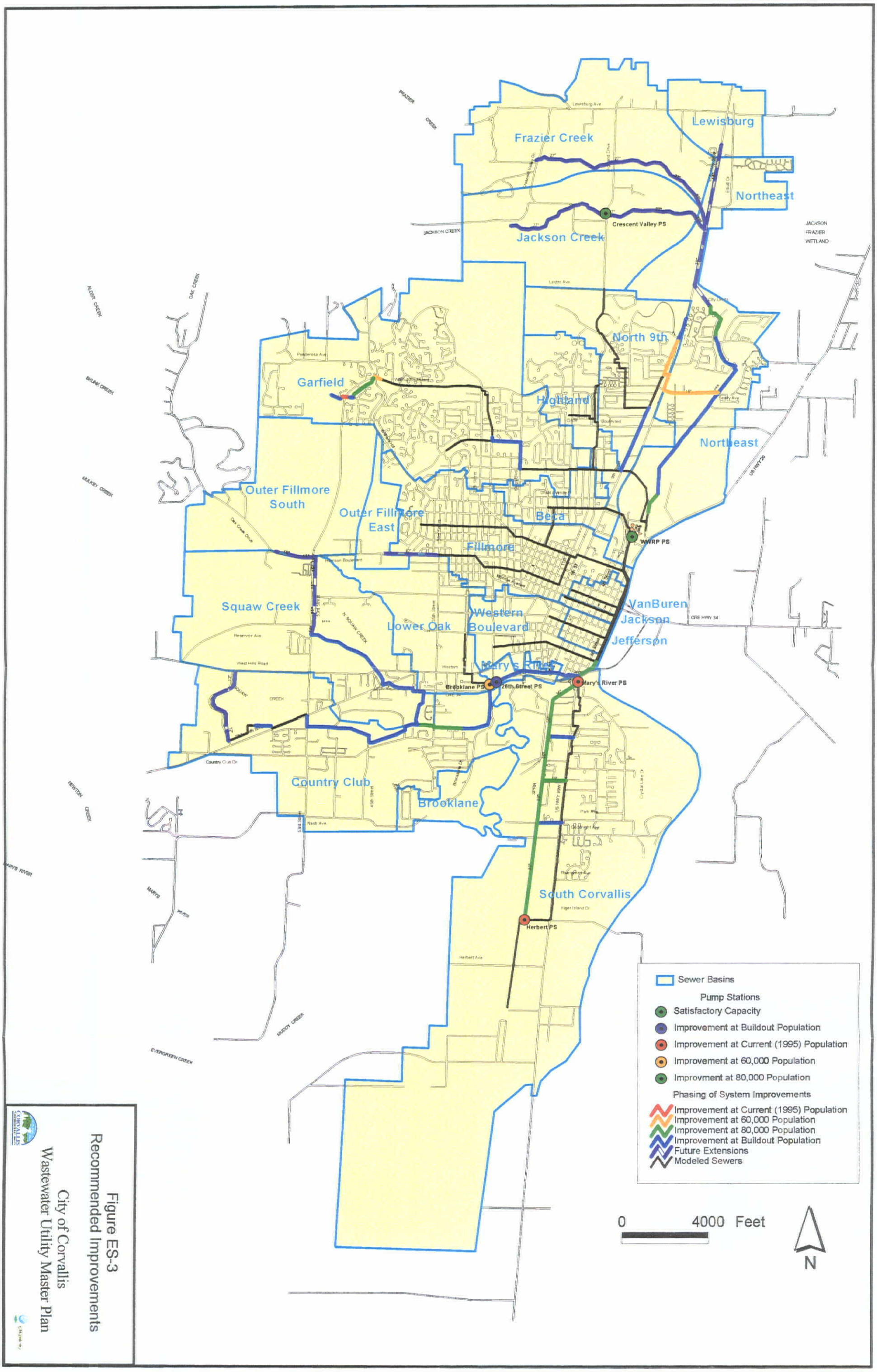

Figure ES-3
Recommended Improvements
City of Corvallis
Wastewater Utility Master Plan

30-JUL-1998

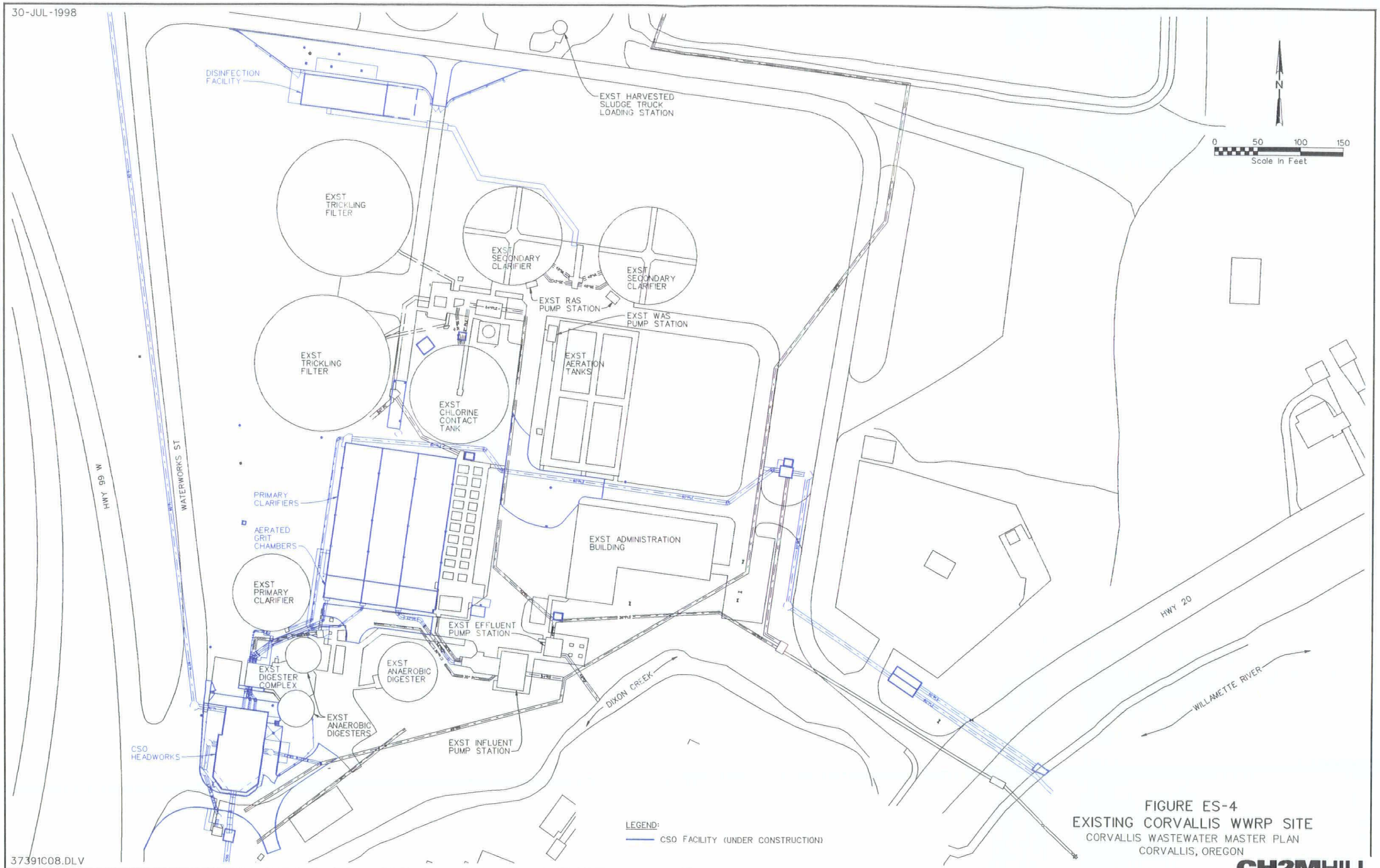

FIGURE ES-4
EXISTING CORVALLIS WWRP SITE
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

37391C08.DLV

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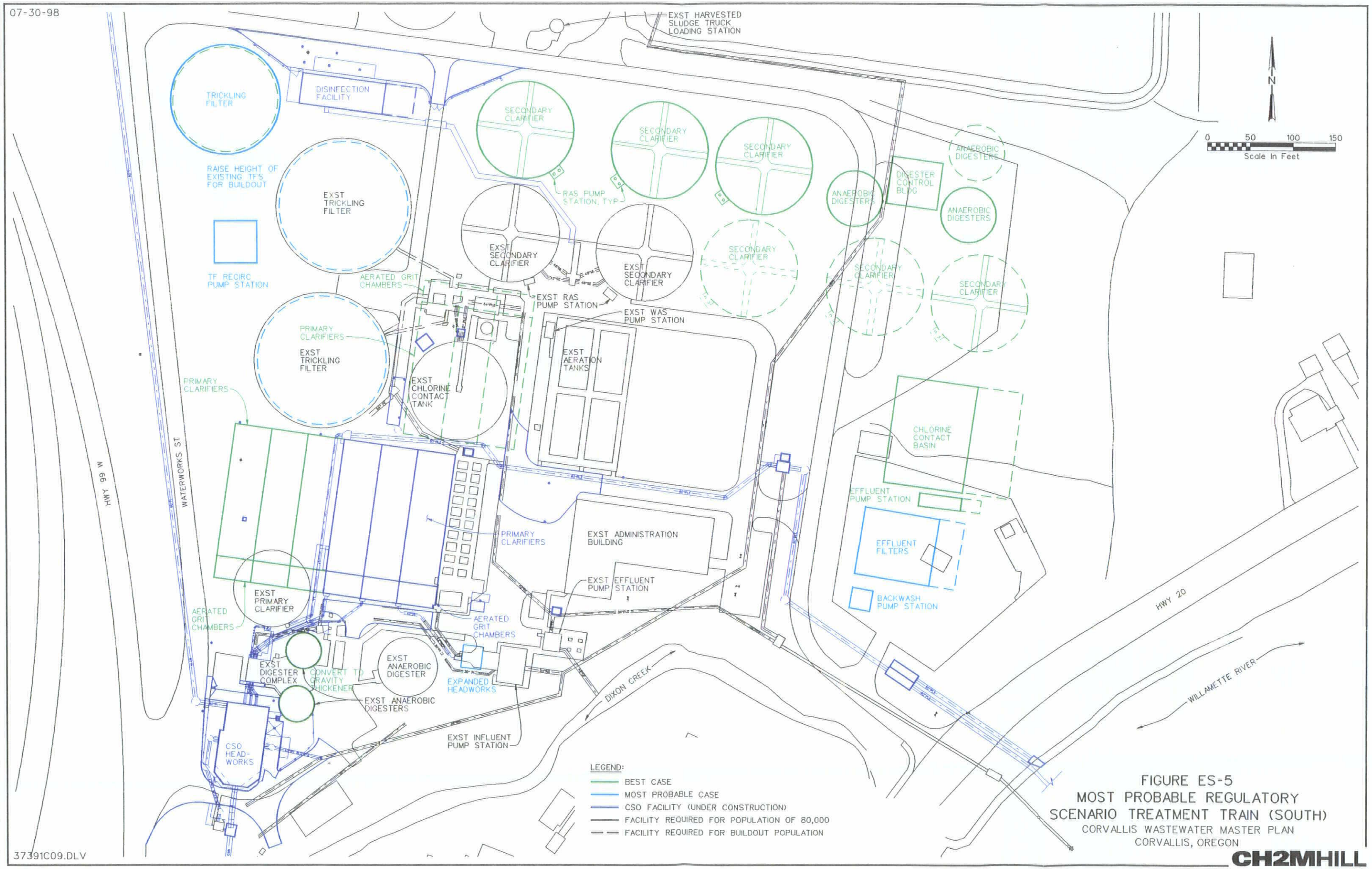

FIGURE ES-5
MOST PROBABLE REGULATORY SCENARIO TREATMENT TRAIN (SOUTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

37391C09.DLV

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07-30-98

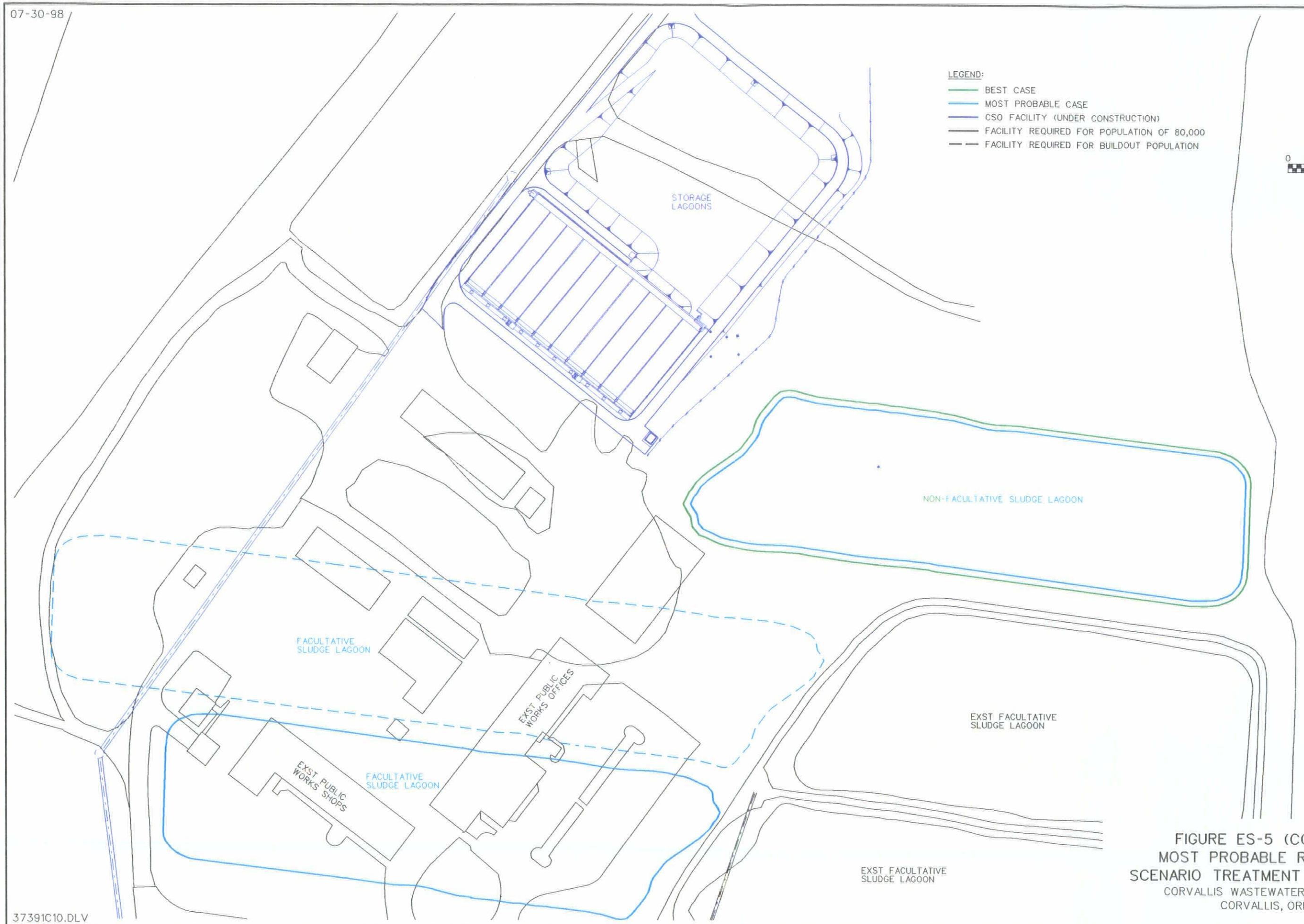


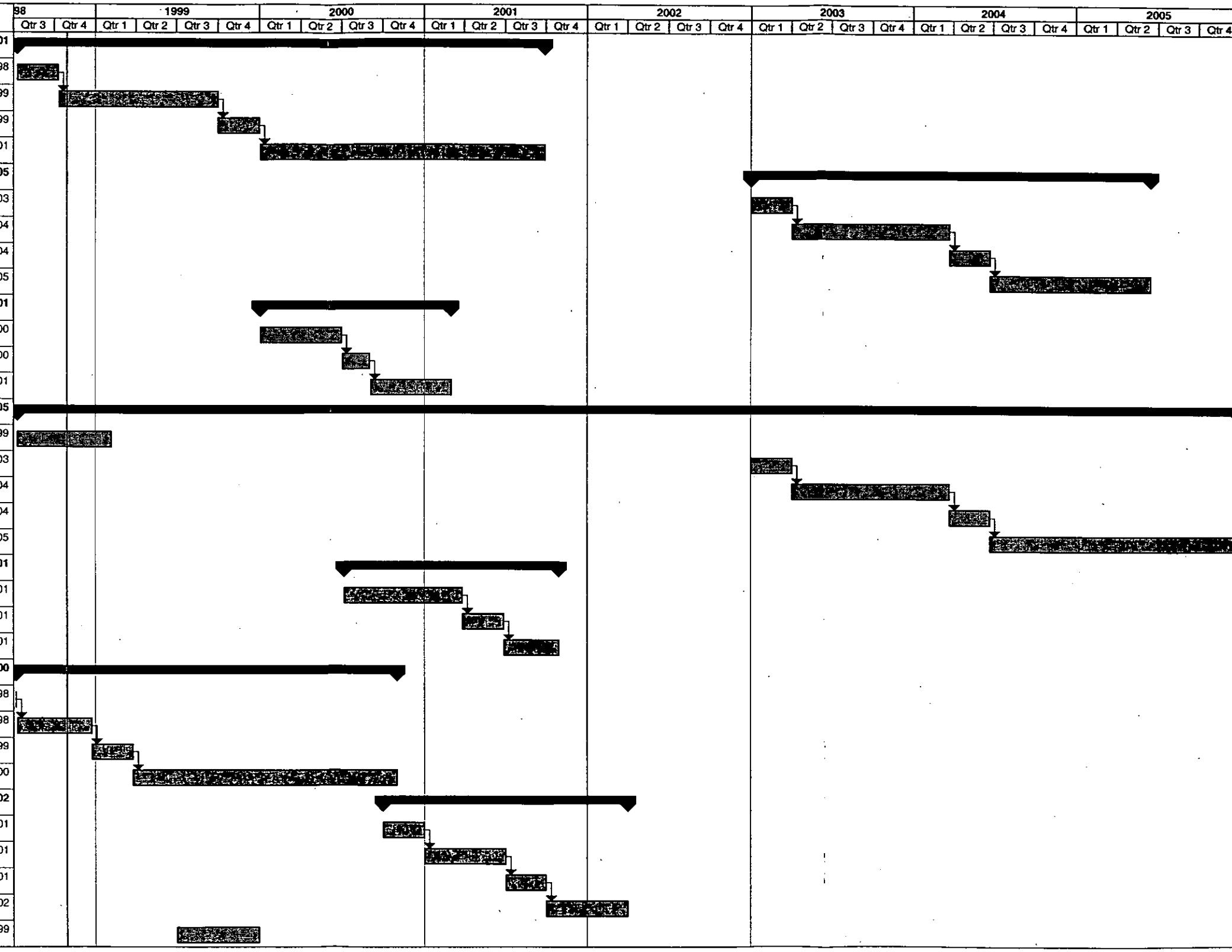
FIGURE ES-5 (CONTINUED)
MOST PROBABLE REGULATORY
SCENARIO TREATMENT TRAIN (NORTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

37391C10.DLV

CH2MHILL

Figure ES-6
Short-term Capital Improvement Projects
Corvallis Wastewater Utility Master Plan

ID	Task Name	Duration	Start	Finish
1	**Marys River Pump Station**	**841 days**	**Fri 7/10/98**	**Fri 9/28/01**
2	Professional Services Procurement	66 days	Fri 7/10/98	Fri 10/9/98
3	Design	254 days	Mon 10/12/98	Thu 9/30/99
4	Advertisement/Award	66 days	Fri 10/1/99	Fri 12/31/99
5	Construction	455 days	Mon 1/3/00	Fri 9/28/01
6	**Brooklane Pump Station**	**640 days**	**Wed 1/1/03**	**Tue 6/14/05**
7	Professional Services Procurement	66 days	Wed 1/1/03	Wed 4/2/03
8	Design	254 days	Thu 4/3/03	Tue 3/23/04
9	Advertise/Award	66 days	Wed 3/24/04	Wed 6/23/04
10	Construction	254 days	Thu 6/24/04	Tue 6/14/05
11	**Herbert Pump Station High Flow Bypass**	**305 days**	**Mon 1/3/00**	**Fri 3/2/01**
12	Design (Assumed by City Staff)	130 days	Mon 1/3/00	Fri 6/30/00
13	Advertise/Award	45 days	Mon 7/3/00	Fri 9/1/00
14	Construction	130 days	Mon 9/4/00	Fri 3/2/01
15	**North Ninth Street Diversion**	**1950 days**	**Fri 7/10/98**	**Thu 12/29/05**
16	Interim Grinder Pump Station(design/const.)	150 days	Fri 7/10/98	Thu 2/4/99
17	Professional Services Procurement	66 days	Wed 1/1/03	Wed 4/2/03
18	Design	254 days	Thu 4/3/03	Tue 3/23/04
19	Advertisement/Award	66 days	Wed 3/24/04	Wed 6/23/04
20	Construction	396 days	Thu 6/24/04	Thu 12/29/05
21	**Garfield Relief Sewer**	**342 days**	**Fri 7/7/00**	**Mon 10/29/01**
22	Design (assumed by City staff)	188 days	Fri 7/7/00	Tue 3/27/01
23	Advertisement/Award	66 days	Wed 3/28/01	Wed 6/27/01
24	Construction	88 days	Thu 6/28/01	Mon 10/29/01
25	**Solids Handling/Odor Control**	**607 days**	**Tue 7/7/98**	**Wed 11/1/00**
26	Professional Services Procurement (completed)	1 day	Tue 7/7/98	Tue 7/7/98
27	Design	118 days	Fri 7/10/98	Tue 12/22/98
28	Advertisement/Award	66 days	Wed 12/23/98	Wed 3/24/99
29	Construction	420 days	Thu 3/25/99	Wed 11/1/00
30	**Non-Process Facilities**	**391 days**	**Mon 10/2/00**	**Mon 4/1/02**
31	Professional Services Procurement	66 days	Mon 10/2/00	Mon 1/1/01
32	Design	130 days	Tue 1/2/01	Mon 7/2/01
33	Advertise/Award	65 days	Tue 7/3/01	Mon 10/1/01
34	Construction	130 days	Tue 10/2/01	Mon 4/1/02
35	**Seismic Analysis, Phase 2**	130 days	Thu 7/1/99	Wed 12/29/99



Project: SCH0707D
Date: Wed 10/28/98

Task | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Milestone | Rolled Up Progress | External Tasks | Project Summary | Split | Rolled Up Split

SECTION 1

Planning Criteria

Introduction

The City of Corvallis Wastewater Utility encompasses more than 200 miles of sewer pipe, six major lift stations, and one regional wastewater reclamation plant (WWRP). The fiscal year 1997/98 annual operating budget is $2.1 million. The 1997 capital improvement program for the utility exceeds $42 million. Given the large size and complexity of the utility infrastructure, preparation of a wastewater utility master plan is necessary for the efficient planning and implementation of utility improvements that are required to protect human health and the environment, and comply with all regulatory requirements.

In 1978, a Corvallis master plan was prepared for the utility collection system, i.e., sewer collection pipes and lift stations. A facilities plan (analogous to a master plan) was prepared in 1979 for the wastewater reclamation plant. Subsequently, these planning documents have successfully guided wastewater utility improvements for many years.

Since the late 1970s, the number of individuals and businesses served by the wastewater utility has increased by more than 25 percent. City land uses have changed, and environmental regulations have become much more stringent. As a result of these changes, the utility determined in 1996 that a new master plan was needed—a master plan integrating collection system and WWRP requirements as well as the results of the recently completed combined sewer overflow remediation plan. Preparation of this wastewater utility master plan was initiated with the following objectives:

1. Develop plans to correct existing collection system and treatment system deficiencies
2. Plan for the orderly extension of utility services within the urban growth boundary consistent with the City's Comprehensive Plan
3. Ensure wastewater collection and treatment is protective of the environment
4. Maintain full compliance with all Oregon Department of Environmental Quality (DEQ) and U.S. Environmental Protection Agency (EPA) regulations
5. Develop plans to meet potentially more stringent future regulatory requirements
6. Obtain community input and guidance in developing the master plan

The desired outcome of the master planning effort is the preparation of a comprehensive plan that protects public health and the environment, that is supported by the community, and that serves as a basis for the wastewater utility Capital Improvement Program.

To achieve these objectives and desired outcome, preparation of the master plan followed a prescribed series of major activities. The activities formed the basis for the master plan document and its organization. The activities, which are addressed more thoroughly in this and subsequent sections, were as follows:

1. Preparation of land use and population planning criteria to guide the development of the plan. Current and future land use and population criteria were obtained from the City's latest Comprehensive Plan.
2. Establishment of regulatory criteria to guide development of the plan based on all current DEQ and EPA applicable regulations, and identification of possible future regulatory changes that might impact the utility.
3. Inventory of the existing sewer collection system and expansion of the City's XP-SWMM hydraulic model of the system. The hydraulic model was then used to evaluate various improvement alternatives to alleviate the deficiencies and assess the cost-effective level of infiltration and inflow correction for the system.
4. Concurrent with the collection system work, a condition assessment of the WWRP was performed. The plant condition assessment coupled with short-term population projections provided the basis for identifying potential short-term improvements. Long-term WWRP improvement alternatives to serve higher population projections were then evaluated.
5. Collection system and WWRP improvements were then integrated into a comprehensive plan for implementation and fiscal planning. Short-term improvements consistent with a Capital Improvement Program, and long-term improvements required over the duration of the planning horizon were developed.
6. In addition to identifying utility system improvements and funding requirements, identification of utility policy and management activities necessary to support achieving the master plan objectives was performed.
7. Perform seismic vulnerability evaluation of the existing WWRP facilities.
8. Throughout the master planning effort, community input was continually sought through a variety of outreach efforts to ensure the plan is responsive to and consistent with community values.

Existing Conditions

Study Area Boundaries and Topography

The City of Corvallis is located on the western side of the Willamette River Valley on an alluvial plain broken on the north and west by the foothills of the Coast Range. Figure 1-1 delineates the land use designations within the study area which is the Corvallis urban growth boundary. The study area fully encompasses the separated and combined sewer systems within the corporate limits of the City. Figure 1-2 shows the sewer basins and the existing service area which matches the current City limits. The City of Corvallis provides community wastewater services to areas within the city limits and additional areas when annexed to the City.

The topography is divided into two distinct areas: the Willamette Valley floor and the foothills at the north and western periphery of the City. The valley floor is relatively flat. Most of the City is at an elevation between 220 feet and 250 feet mean sea level, and the

slopes are generally less than three percent. Most of the urban development has occurred in the valley floor area. In recent years, however, considerable development has extended into the foothill areas. These are the principal areas being developed at this time. The more hilly districts are seen at IV Hill, Cemetery Hill, Witham Hill, Timberhill Ridge, and portions of the Garfield and Crescent Valley basins, with elevations rising to 550 feet.

Drainage Basins

The significant surface waters within the City's general service area are Squaw Creek, Mill Race, Sequoia Creek, Oak Creek, Dixon Creek, Marys River, and the Willamette River. Squaw Creek and Oak Creek are tributaries of Marys River, while Mill Race and Dixon Creek discharge directly to the Willamette River. Seven of the City's 12 service basins have separated sanitary and storm sewers: Crescent Valley, Garfield, Highland, North 9th Street, Beca, Brooklane, and South Corvallis. The Fillmore, Van Buren, Jackson, Jefferson, Oregon State University (OSU), and Western basins contain combined sewers.

Crescent Valley

The current Crescent Valley service area provides for collection of wastewater discharged from the light industrial facilities of Hewlett-Packard, Crescent Valley High School, and the residential areas of this basin which includes the Lewisburg, Frazier Creek, and Northeast basins. The service area is currently 525 acres. The service area has an additional developable area of 2,893 acres. Fully developed, the basin will encompass 3,418 acres.

Garfield

The current Garfield service area, which is comprised of the Garfield, Highland, and North 9th Street sub-basins, totals 1,984 acres. It is the largest current service area in the City, and contains the largest population. The area is primarily residential, but includes the retail shopping center at Kings and Circle Boulevards, commercial development along 9th Street, and the Good Samaritan Hospital. The service area has an additional developable area of 1,814 acres. Fully developed, the basin will encompass 3,798 acres.

Beca

The Beca basin is currently 245 acres, and is completely surrounded by other basins. As a result, minimal developable area increase of 107 acres through infill is projected. Most of the service area is residential, with commercial developments along 9th Street and Kings Boulevard. Fully developed, the basin will encompass 352 acres.

Brooklane

The Brooklane service area includes the Squaw Creek, Country Club, Lower Oak Creek, Mary's River and Brooklane basins which are predominately developed with single-family homes and open space. Commercial and industrial development is limited to the Sunset Research Park. The total area served is currently 755 acres. The service area has an additional developable area of 2,065 acres. Parts of the Brooklane basin are within the 100-year floodplain of Marys River and are therefore considered less desirable for urban development. Fully developed, the basin will encompass 2,820 acres.

South Corvallis

The south Corvallis service area is currently 600 acres of mixed residential development with a section of linear commercial development along Highway 99W, and some industrial development. This area includes the Corvallis Municipal Airport. The service area has an additional developable area of 2,221 acres. Fully developed, the basin will encompass 2,821 acres.

Combined Sewer Area

The combined sewer area which includes the Fillmore, Van Buren, Jackson, Jefferson, OSU, and Western basins, is currently 1,386 acres. The area consists of a combination of intense commercial development and older homes that have been converted to office and multi-family units, as well as large tracts of single-family residential housing. It includes the downtown core area and portions of Oregon State University. It is also served by the oldest sewers in the City. Growth will occur predominately in the westernmost regions. Ultimately, the basin area will increase to 2,254 acres. Of this increase, 206 acres will drain through the existing basin. The balance, 262 acres, will flow through the Brooklane service area.

When it is not raining, all the wastewater and any surface drainage or infiltration from this combined area is conveyed to the Corvallis WWRP. During periods of heavy rainfall, the combined sewers become overloaded from stormwater and overflow at five locations into the Willamette River before they reach the treatment plant. When this happens, the wastewater component of the overflow becomes a source of pollution in the river, particularly bacterial contamination. Such an event is referred to as a combined sewer overflow (CSO).

Although the Corvallis piping system transports wastewater to the treatment plant effectively during the dry weather season, it at times cannot do so during the wet weather season, when the capacities of the combined sewers are exceeded because of high stormwater flows. Corvallis is one of about 1,100 cities in the United States and three cities in Oregon that have combined sewer systems. During 1994, an estimated 425 million gallons of CSOs discharged into the Willamette River from the Corvallis combined system. The discharge consisted mostly of stormwater runoff from Corvallis's streets, sidewalks, parking lots, rooftops, and other impervious surfaces. The City has selected a plan to remediate this condition which consists of construction of wet weather treatment processes, a storage lagoon and a relief interceptor. This project is currently under construction.

Previous Studies

The City of Corvallis has had a long history of responding to water quality challenges. Accordingly, reducing the frequency of CSOs has been a concern of the City of Corvallis for many years. Long before there was either a federal or state policy on CSOs, the City sought solutions to the water quality problems caused by CSOs. The *1973 Wastewater Treatment Program* report (Brown and Caldwell) acknowledged the issue, and when the Corvallis WWRP was upgraded in 1977, the existence of combined sewers was taken into consideration. As a result of this recognition, the problem was studied and documented in the 1977 *Wet Weather Overflow Management Program* report (Brown and Caldwell), which recommended constructing holding basins for overflow that would be treated later at the WWRP.

The 1979 *City of Corvallis Sanitary Sewer Plan* (Regional Consultants, Inc.) addressed growth management criteria established in the City's Comprehensive Plan (December 1978). In 1992, the holding basin recommendation was reiterated in a memorandum report entitled "Update of the 1977 Combined Sewer Overflow Report" (Brown and Caldwell). In January of 1996, CH2M HILL completed the *City of Corvallis CSO Remediation Plan*, which is incorporated by reference into this Master Plan. The scope of the project included characterization of CSOs, analysis of infiltration and inflow, development of remediation alternatives, conducting a public information and involvement program, selection of a preferred remediation alternative, and development of an implementation plan.

Soils

Surface soils within the study area are mainly silt-loams having high agricultural and silvicultural capabilities. These soils have a moderate shrink-swell potential with a slow to moderate permeability of less than 2 inches per hour. Isolated areas of clay soil exist primarily in the foothills and along natural drainageways. Earthflow and slump topography occur in hilly terrain and are associated with poor drainage, shallow subsurface flow of groundwater and springs, and the erosion susceptibility of these soils. A large portion of the land within the Corvallis urban growth boundary consists of hydric soils and may be classified as wetlands, which will affect development of these lands. Bedrock is not seen at normal excavation depths in the valley floor areas.

Climatological Effects on Groundwater and Surface Water

The climate of the Corvallis area is of a marine type, with warm, dry summers and mild, wet winters. The Oregon Climate Service gives an average annual rainfall for Corvallis of 42 inches, approximately 70 percent of which occurs November through March. Groundwater levels vary with the season. The wet weather high water table is roughly 20 inches below the ground surface, but during periods of heavy rainfall the ground becomes saturated to the surface. Ponding is seen in poorly drained areas, and marshy or bog-like conditions are observed in the lowlands much of the rainy season. During the summer months, the groundwater table drops 10 to 20 feet or more, so that most sewerlines within the study area are above the groundwater table.

Because of Pacific marine influences, temperatures are moderate and freezing temperatures rarely affect groundwater conditions. Surface runoff temperatures are often affected during periods of extreme cold. The mean monthly temperature varies from 38.3°F in January to 66.4°F in July. Average annual snowfall is 8 inches. However, accumulations on the valley floor seldom exceed 4 inches. Temperatures below freezing occur 60 to 70 days a year.

Water Quality

Untreated point sources, principally CSOs, and nonpoint sources, primarily agricultural and forest runoff, pose a potential for seasonal water quality problems in the local Willamette River vicinity and area streams now and in the future. According to recent DEQ studies, during wet weather periods the Willamette River basin experiences elevated pollutant concentrations, particularly bacterial and suspended solids, which are mainly caused by nonpoint sources. The presence of high coliform bacteria counts is the primary surface water quality problem in the area. CSOs, when they occur, contribute locally to these high bacteria counts.

Land Use

The study area is devoted primarily to urban land uses. However, outside the Corvallis city limits and within the urban growth boundary, the land use is 90 percent rural. The available open space within the city limits includes sparse woodlands of natural oak, Douglas fir, and maple trees, parks, green spaces, gardens, and lawns. Green spaces, lawns, and grassy areas are the main forms of open space in the study area. The City's vegetation serves a variety of uses, including erosion control, noise attenuation, atmospheric purification, reduction of glare, wind control, and aesthetics.

In general, the study area is at a high level of development. The types of land use present include residential, commercial, industrial, public, streets, agricultural and undeveloped lands. Land use is primarily single family, multiple family, and commercial in the combined sewer area, with a large percentage of impermeable surface area. Public and semi-public land is composed primarily of schools and parks.

The Oregon State University campus is within the combined sewer area, and is the geographic center of the community. The university is also the largest single landowner within the urban growth boundary. The areas outside of the combined sewer area are mainly residential, multiple family, commercial and light industrial.

Residential uses occupy more developed land than any other land use in the City, and are in most cases classified as relatively new and in good repair. A 1979 study classified 15 to 20 percent of the City's dwelling units as substandard. A 1989 study indicated that 58 percent of the existing housing consists of single-family dwellings and 42 percent consists of multiple-family dwelling units. Historically, Corvallis has maintained an average of 20 percent undeveloped land.

The City's 1995 population was estimated to be 47,485 increasing to 49,275 by 1996. An unserved population outside the City limits but within the UGB was estimated to be 4,513. As with most communities, Corvallis' population is closely tied to employment. Currently, the stability of the City's economy and population depends largely on a few major employers in a few economic sectors. Primarily, these consist of Oregon State University; local, state, and federal government employers; and firms engaged in electronics, forest products, consulting, and medical services. Oregon State University, Hewlett-Packard, and health service employers provide jobs for more than one-third of the work force.

The college student population comprises approximately one-third of the total Corvallis population. The population growth rate for Corvallis from 1970 to 1996 was 1.22 percent. Figure 1-3 shows the City's population growth during this time period. Based on the 1990 periodic review of the Corvallis Comprehensive Plan, the population was projected to grow to 62,500 in 20 years. The City's 1997 estimates in *The Corvallis 2020 Vision Statement* were that Corvallis would grow to between 57,500 and 63,500people by the year 2020.Although Oregon State University will continue to grow, its growth will be a smaller component of the overall growth of the city when compared to past decades. Table 1-1 shows population and land area estimates for the sewered basins of the study area under several growth scenarios. The growth projections are for population milestones and are not tied to specific time frames, as growth rates can fluctuate considerably. The Beca basin is entirely enclosed by other basins and cannot increase its areal extent, although its population will continue to increase because of infill. The downtown combined sewer area and Crescent Valley basin

Figure 1-3
City of Corvallis Historical Population Growth

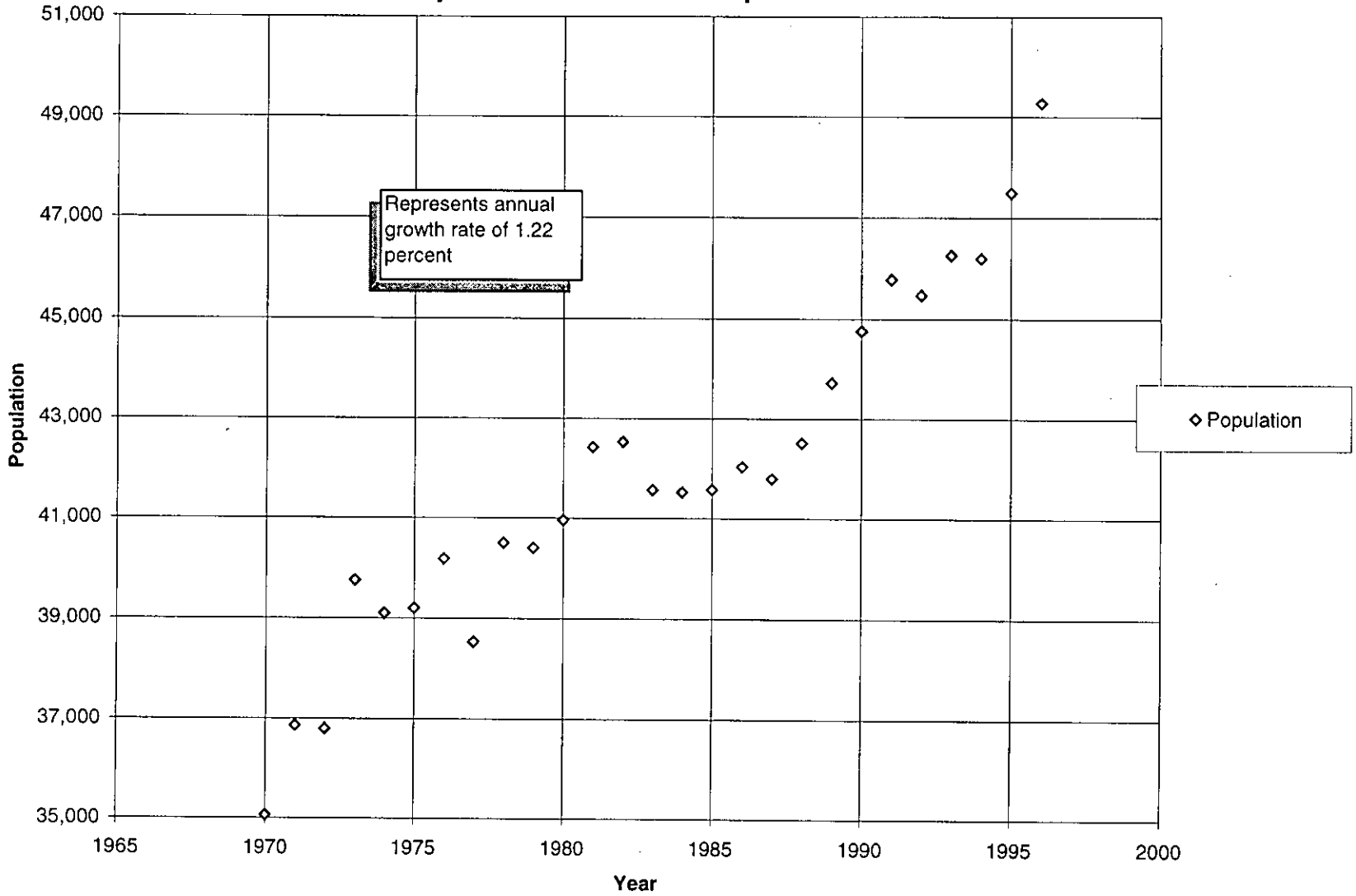


Table 1-1
Population Distribution for System Modeling

Basin	Existing Service Area Estimate (acres)	Existing (1995) Population within City Limits	Growth Population Increment for 60,000	Incremental Developed Acres at 60,000	Growth Population Increment for 80,000	Incremental Developed Acres at 80,000	Growth Population Increment for 120,000 (Probable Buildout)	Incremental Developed Acres at 120,000 (Probable Buildout)	Growth Population Increment for Maximum Buildout	Incremental Developed Acres at Maximum Buildout	Total Developed Acres at Maximum Buildout
Crescent Valley Service Area											
Lewisburg			202	15	685	52	1677	127	3,124	236	
Frazier Creek			701	66	2382	225	5831	550	10,865	1025	
Northeast			889	51	3018	172	7390	421	13,770	784	
Jackson Creek			531	55	1805	186	4419	455	8,235	848	
Subtotal	**525**	**2,009**	**2,323**	**187**	**7,890**	**634**	**19,317**	**1,553**	**35,994**	**2,893**	**3,418**
Garfield Service Area											
North Ninth			278	32	944	108	2310	264	4,305	492	
Highland			30	5	103	16	253	40	471	74	
Garfield			608	81	2066	273	5058	670	9,425	1248	
Subtotal	**1,984**	**16,776**	**916**	**117**	**3,113**	**398**	**7,621**	**973**	**14,201**	**1,814**	**3,798**
Beca	**245**	**1,685**	**38**	**7**	**128**	**24**	**312**	**58**	**582**	**107**	**352**
Brooklane Service Area											
Lower Oak Creek-BR2A			110	7	372	24	911	59	1,697	111	
Squaw Creek-BR3			754	56	2561	190	6269	464	11,681	865	
Country Club-BR4			775	64	2632	219	6444	536	12,007	998	
Brooklane			45	5	153	16	373	40	696	74	
Marys River			42	1	143	4	350	9	653	18	
Subtotal	**755**	**8,270**	**1,725**	**133**	**5,860**	**453**	**14,348**	**1,108**	**26,735**	**2,065**	**2,820**
Combined Service Area											
Western Blvd.			-		0		0		-	0	
Van Buren			-		0		0		-	0	
Jackson											
Jefferson			-		0		0		-	0	
Fillmore			504	25	1712	83	4191	204	7,810	380	
Outer Fillmore East			66	13	222	45	545	111	1,015	206	
Outer Fillmore South			82	18	279	62	682	151	1,271	282	
Subtotal	**1,386**	**14,360**	**651**	**56**	**2,213**	**190**	**5,418**	**466**	**10,096**	**868**	**2,254**
South Corvallis	**600**	**4,385**	**2,607**	**143**	**8,857**	**487**	**21,684**	**1,192**	**40,404**	**2,221**	**2,821**
Total Acres and Incremental Growth Acres	**5,495**			**643**		**2,185**		**5,350**		**9,968**	**15,463**
Total Incremental Population Growth			**8,261**		**28,060**		**68,700**		**128,011**		
1995 Population within City Limits		**47,485**	**47,485**		**47,485**		**47,485**		**47,485**		
1995 Unserved Population			**4,513**		**4,513**		**4,513**		**4,513**		
Total Population			**60,259**		**80,058**		**120,698**		**180,009**		

Note: All incremental populations and acreages are relative to the existing population and developed acres.

are not expected to expand unless the Corvallis population exceeds the 80,000 milestone. Most of the near-term population growth is expected in the Brooklane, West Corvallis and South Corvallis areas.

Future Conditions

Planning Area

Figure 1-1 is a map of the area within the City's urban growth boundary with land use designations for buildout conditions, including boundaries of the City's sewer basins.

Population and Land Use Projections

Urban development patterns in the Corvallis area will continue to be heavily influenced by geographic constraints. The City is surrounded on three sides by land that is poorly suited for urbanization. To the east and south there are the Willamette and Marys Rivers and their lowland floodplains; and abutting the western periphery of the City there are substantial holdings of state-owned property under the control of Oregon State University. The City's growth is also hindered by its distance from the interstate freeway system, its location on a railroad spur line instead of a main line, a minimum of scheduled airline flights, and a lack of significant tourist attractions. The direction of urban expansion in Corvallis has been and continues to be to the south, southwest, and northwest. The largest land use category will continue to be residential.

The timing and location of public water and sewer system expansions are also important factors affecting future growth. Provision of adequate sewerage facilities is a prerequisite to the urbanization of land. Generally, without a central wastewater collection system, properties cannot be constructed more intensively than one dwelling unit per acre. Current UGB planning rules allow no more than one unit per acre.

The Corvallis urban growth boundary, adopted as an element of the Corvallis Comprehensive Plan, was originally intended to provide for a population of 80,000. Subsequent planning efforts, such as the development of master plans for water or sanitary sewer systems, used greater development densities, and projected a residential buildout population of 120,000 within the urban growth boundary.

If, however, it is assumed that each basin develops to the maximum density allowable under the land use and zoning regulations, then the aggregate projected population for the area within the urban growth boundary equals approximately 180,000. Since this estimate was produced, protection of resource lands has received greater attention. This could significantly constrain development to less than the maximum density allowed. Such circumstances support selection of the 120,000 estimate as the probable buildout population to use for planning purposes.

The 120,000 population was therefore used as the buildout condition for treatment plant improvements. It was recognized, however, that some basins may reach the maximum density allowed and that collection system improvements must be sized to accommodate that demand. Planning periods were developed to reflect these population milestones. A population milestone of 60,000 was selected for purposes of near-term planning and timing of improvements.

Population estimates within individual sewer basins were obtained for the existing and maximum buildout land use conditions from the City's planning department. Estimates of sewer basin populations for 60,000, 80,000, and 120,000 planning milestones were calculated from this population information. Table 1-1 summarizes population projections by sewer basin for each of the four planning milestones.

Basis of Costing

Table 1-2 summarizes assumptions used as the basis for evaluating alternatives. Table 1-3 gives cost information used in the analysis of collection system alternatives.

TABLE 1-2
Basis of Costing

Item	Assumption
Useful Life	
Equipment	20 years
Structures	50 years
Pipelines	80 years
Present Worth Analysis	
Discount Rate	5.5 percent
Evaluation Period	20 years
Depreciation	Straight line method
Operation and Maintenance Costs	
Salary Cost	$28.00/hour (includes fringes)
Electricity	$0.04/kWh
Polymer	$2.00/lb
Equipment maintenance	5% of capital cost/yr
Land Cost	$3,500/acre
Truck/Hauling O&M	$7.25/wet ton (assumes 16 mile round-trip haul)
Dredge O&M	2 percent of equipment cost/year

All cost estimates presented in this master plan and appendices have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. The final costs of the projects will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors. As a result, the final project costs will vary from the estimates presented herein. Because of this, project feasibility and funding needs must be carefully reviewed before making specific financial decisions to help ensure proper project evaluation and adequate funding.

Because the cost estimates are based on planning level information, an allowance of 25 percent has been added to account for unknown elements that may be required but cannot be identified at this level of project development. To the extent possible, the construction cost estimates have been compared with similar, recently constructed facilities in the Pacific Northwest.

TABLE 1-3
Unit Cost for Pipeline Replacement and Lift Station Expansion/Replacement

Pipe Diameter (inches)	$/foot[a]	Pump Station Capacity (mgd)	Estimated Construction Cost
8	67	0.10	$270,000 [b]
10	84	0.30	$340,000 [b]
12	101	0.50	$420,000 [b]
15	126	1.00	$580,000 [b]
18	151	2.00	$810,000 [b]
21	177	3.00	$1,020,000 [b]
24	202	4.00	$1,300,000 [c]
27	227	5.00	$1,400,000 [c]
30	252	7.00	$1,800,000 [c]
36	303	10.00	$2,200,000 [c]
42	353	12.00	$2,400,000 [c]
48	404	15.00	$2,700,000 [c]
54	454	20.00	$3,200,000 [c]
60	505	40.00	$4,900,000 [c]

Notes:

[a] Cost shown are from EPA construction cost tables updated to 1998 basis. (ENR CCI Seattle = 6500). Costs do not include contingency or E,L,A

[b] Costs assume package pump stations with standby emergency generator. Costs do not include forcemain replacement.

[c] Costs assume permanent pump stations with emergency generator or secondary power supply. Costs do not include forcemain replacement.

An engineering, legal, and administrative cost allowance of 25 percent was added to the total construction cost in order to estimate the total program cost of the projects. The allowance would typically provide for services that are required on major projects such as special studies, engineering reports and contract documents, construction management, startup, legal services, and liaison with regulatory and funding agencies.

Total program costs (construction plus engineering, legal, and administrative) are in June 1998 dollars and indexed to an *Engineering News Record* Construction Cost Index (CCI) of 6500 for the Seattle area. The costs presented are order-of-magnitude estimates as defined by the American Association of Cost Engineers and the American National Standards Institute. Order-of-magnitude estimates are prepared without the benefit of detailed engineering data. It is normally expected that a cost estimate of this kind will be accurate within +50 percent or -30 percent. The range is based on statistical information covering a wide range of projects.

Basis of Facilities Improvements

Table 1-4 summarizes the criteria used for the system deficiency and improvement analysis.

Table 1-4
System Deficiency and Improvement Analysis Criteria

Population	Used for Deficiency Identification in the Collection System	Used for Sizing Improvements Where Deficiencies have been Identified	Peak Flow Rate at Plant Estimated
47,485(1995/existing)	Yes	No	Yes
60,000	Yes	No	Yes
80,000	Yes	No	Yes
120,000 (maximum total population) [a]	No	No[c]	Yes
Buildout within each subbasin [b]	Yes	Yes	No

[a] 120,000 has been estimated by City staff as the most probable maximum total population. It is less than full buildout in all basins within the urban growth boundary; however, the City does not anticipate that all basins will reach full buildout.
[b] This analysis is performed to reflect that some basins will reach a buildout condition and the sizing improvements must accommodate that demand. However, not all basins are expected to reach buildout, which reflects the City's prediction that there will be a growth differential among basins. Because it is not possible to predict the exact distribution of the 120,000 total population among basins, the analysis was performed for each basin at buildout. This approach provides sizing information for future improvements for the broadest range and combinations of growth conditions.
[c] Marys River and Crescent Valley Interceptors were sized based on 120,000 population because they serve multiple upstream subbasins, which are not all expected to reach buildout population levels.

Current Projects Assumed to be Implemented

As part of the City of Corvallis CSO Remediation Plan and Preliminary Design studies, ten potential alternatives for CSO mitigation were modeled and evaluated. The selected alternative, which is assumed to be implemented for purposes of the current study, consists of intercepting CSOs at the Fillmore, Polk, Van Buren, Jackson, and Western overflow locations, conveying them in a 42-inch-diameter interceptor along 1st Street to Fillmore Avenue, and then in a 72-inch-diameter pipe from Fillmore to a new wet weather treatment facility (WWTF). Construction of the CSO remediation facilities is currently underway. The facilities should be operational by 2001.

In order to address the long-standing problem of basement flooding in the Beca basin, the City recently constructed a relief sewer to divert rainfall-derived infiltration and inflow flows in excess of the capacity of the existing Beca sewer line to the Fillmore basin. This new line begins at Beca Avenue at NW 16th Street and end at NW 16th Street and Taylor Avenue. For purposes of this study, this project was considered to be in place.

SECTION 2

Regulatory Requirements

The environmental regulatory requirements that pertain to the City of Corvallis wastewater collection and treatment facilities were reviewed and evaluated to identify planning criteria for future expansions and improvements.

This entailed review of the existing regulatory requirements and emerging regulatory changes. On the basis of this review, an effluent quality evaluation was performed to identify potential discrepancies between current capabilities and future requirements. The analysis focused on the effects of a population of 60,000.

The results of the regulatory review and effluent quality evaluation were then used to develop three effluent planning scenarios. In addition, three environmental-adjacency planning scenarios were developed.

Existing Regulatory Framework

The existing regulatory framework for the collection system relates primarily to combined sewer overflows (CSOs). For the Corvallis WWRP, it includes instream water quality standards, treatment requirements, and key DEQ policies.

Combined Sewer Overflows

The U.S. Environmental Protection Agency (EPA) requires states to develop CSO control strategies. In the spring of 1992, The City of Corvallis and the Oregon Department of Environmental Quality (DEQ) began negotiations on how to address CSOs. In November 1992, the City and DEQ agreed to a stipulation and final order (SFO) regarding the control of CSOs. Subsequently in April 1994, EPA adopted a final policy for controlling CSOs. EPA also developed five guidance documents to assist municipalities and regulators in implementing and adhering to the federal policy. DEQ has developed a state-wide CSO control strategy that is significantly more stringent than the federal policy.

In accordance with the requirements of the SFO, the City formulated and adopted a CSO Remediation Plan in 1996 and began preliminary design of the selected alternative contained therein. Final design of the new CSO Remediation facilities was completed in 1998 and construction has begun on the CSO treatment and storage portions of the project. The CSO interceptor and outfall work will be bid and constructed in 1999. It is the City's goal to complete all construction activities associated with the CSO Remediation Project by the fall of 2000, approximately 1 year ahead of the SFO mandated completion date.

As defined by the SFO, the City is required to capture all of the CSOs that result from a 5-year return frequency winter storm event and a 10-year return frequency summer storm event. The definition of a design storm is provided by historical National Weather Service rainfall data. A 5-year return storm would be expected to be of an intensity and duration that, on average, occurs once every 5 years. Similarly, a 10-year return storm would be

expected to be of an intensity and duration that, on average, occurs only once every 10 years. The summer standard is more stringent because more recreational river use occurs during summer months.

The SFO outlined the time frame for conducting studies and completing construction of the facilities needed to correct the pollution problem caused by CSOs. The City agreed to the following schedule and milestone requirements:

- By January 1, 1997, submit to the DEQ 10 percent complete engineering plans and specifications for the necessary construction work
- By July 1, 1997, submit to the DEQ 30 percent complete engineering plans and specifications
- By July 1, 1998, submit to DEQ final engineering plans and specifications for construction work
- By May 1, 1999, begin construction
- By December 31, 2001, eliminate discharges that violate applicable water quality standards subject to the specified return storm frequencies

Water Quality Standards

The standards for river basins in the State of Oregon are established by DEQ through Oregon Administrative Rule (OAR) 340-41-445. This section is specific to discharges to the Willamette River at approximately River Mile 130, where the WWRP outfall is located. Standards for general water quality parameters, toxic substances, and mixing zones are addressed separately.

General Water Quality Parameters

DEQ's water quality regulations are reviewed every 3 years for setting new or modifying existing standards. Proposed amendments to OAR 340-41-445 resulting from the most recent triennial review were adopted January 11, 1996, although implementation did not occur until July 1, 1996, to allow time for development of implementation guidance. The new standards include dissolved oxygen, temperature, pH, and bacteria (Table 2-1).

The dissolved oxygen criteria are dependent on the fisheries habitat present in the affected river segment. DEQ has indicated that the Willamette River near Corvallis is cold water fisheries habitat capable of supporting salmonid rearing, primarily from March to August. The values listed in Table 2-1 reflect the criteria for these projected conditions.

The new temperature standard is of particular concern to dischargers to the Willamette River from the Calapooia River to Long Tom River because temperature has been identified as a problem in this river segment. The Corvallis WWRP discharge is located in this segment. According to the new temperature standards, exceptions are allowed if:

- It can be shown that the beneficial uses are not impaired, or
- A source is implementing all reasonable management practices or measures, the source's activity will not significantly affect the beneficial use of the water body, <u>and</u> the cost of

treating to the level necessary to assure full protection would outweigh the risk to the resource

TABLE 2-1
Applicable Willamette River Instream Water Quality Criteria
City of Corvallis WWRP

Parameter	Instream Water Quality Criteria
Dissolved Oxygen	Cold Water Criteria: > 8.0 m/L (absolute minimum for surface samples); or > 90 percent saturation, when barometric pressure, altitude, or temperature precludes attainment of 8.0 mg/L
Temperature	No measurable surface water temperature increase resulting from anthropogenic activities is allowed in a basin for which salmonid fish rearing is a designated beneficial use, and in which surface water temperatures exceed 64°F (17.8°C)
pH	> 6.5 and < 8.5
Bacteria	< 126 Escherichia coli (E. coli)/ 100 mL (30-day log mean); < 406 E. coli/ 100 mL (for any single test sample)

The temperature standard includes a provision for the development and implementation of a basin temperature management plan to control anthropogenic sources when the temperature exceeds this criteria. Any source contributing to stream warming where the basin exceeds the criteria would be required to develop and implement a management plan to reverse the individual contribution to the warming trend. Sources having individual permits would develop and include their management plans as part of their National Pollutant Discharge Elimination System (NPDES) permits. According to DEQ's implementation guidance, permittees will be allowed up to 2 years after permit renewal to develop the management plan, and up to 2 years after approval of the management plan for implementation.

Toxic Substances

Toxic substances shall not be introduced above natural background levels in the waters of the state in amounts, concentrations, or combinations that may be harmful, may chemically change to harmful forms in the environment, or may accumulate in sediments or bioaccumulate in aquatic life or wildlife to levels that adversely affect public health, safety, or welfare; aquatic life; wildlife; or other designated beneficial uses (OAR 340-41-445[2][p]). Table 20 of OAR Division 41 provides water quality criteria for toxic substances, such as metals, chlorine, and ammonia.

Mixing Zones

A mixing zone is defined as a designated portion of a receiving water that serves as a zone of dilution where wastewaters and receiving waters mix thoroughly. The DEQ may suspend all or part of the water quality standards, or set less restrictive standards, in the defined mixing zone (OAR 340-41-445[4]) under the following conditions:

1. The water within the mixing zone is free of:
   - Materials in concentrations that will cause acute toxicity to aquatic life (bioassay testing required and approved by DEQ). Acute toxicity is lethality to aquatic life as measured by significant difference in lethal concentration between the control and 100 percent effluent in an acute bioassay test. Lethality in 100 percent effluent may be allowed to be caused by ammonia and chlorine only when it is demonstrated on a case-by-case basis that immediate dilution of the effluent within the mixing zone reduces toxicity below lethal concentrations.
   - Materials that will settle to form objectionable deposits.
   - Floating debris, oil, scum, or other materials that cause nuisance conditions.
   - Substances in concentrations that produce deleterious amounts of fungal or bacterial growths.
2. The water outside the boundary of the mixing zone is:
   - Free of materials in concentrations that will cause chronic (sublethal) toxicity. Chronic toxicity is measured as the concentration that causes long-term sublethal effects, such as significantly impaired growth or reproduction in aquatic organisms, during a testing period based on the test species' life cycle.
   - Meeting all other water quality standards under normal annual low flow conditions.

The DEQ defines the mixing zone on the basis of receiving water and effluent characteristics. The mixing zone limits or outfall location may be changed if DEQ determines that the water within the mixing zone adversely affects any existing beneficial uses in the receiving waters.

The current WWRP mixing zone is defined as a 300-foot mixing zone boundary downstream of the existing outfall, with the zone of initial dilution (ZID) no more than 10 percent of the mixing zone boundary (30 feet). At the time of this report, the NPDES permit was in the renewal process. The definition of the Corvallis WRRP outfall mixing zone is undergoing revision with the addition of a second outfall to discharge treated combined sewer wastewater. In November 1997, the City submitted a dilution and mixing zone study (*Combined Discharge Evaluation of Corvallis Existing Wastewater Reclamation Plant Discharge Outfall and New Treated-CSO Outfall*, CH2M HILL, November 25, 1997) to DEQ so that the new outfall would be included in the renewed NPDES permit. The results of this study are used to evaluate potential future compliance with instream water quality criteria.

Treatment Requirements

DEQ establishes treatment requirements for each basin that must be equal to or more stringent than EPA regulations. The existing NPDES permit limits established by DEQ for the Corvallis WWRP reflect the current treatment requirements for the Willamette River Basin. The permit includes limits for winter and summer 5-day carbonaceous biochemical oxygen demand ($CBOD_5$) and total suspended solids (TSS) and for year-round pH, removal efficiency, and fecal coliform bacteria. The permit also states that no wastes shall be discharged that violate water quality standards as adopted in OAR 340-41-445 except in the defined mixing zone. The permit provides for a suspension of daily maximum mass limits

for $CBOD_5$ and TSS in both winter and summer if on any day the total average daily flow to the treatment facility exceeds 19.4 million gallons per day (mgd).

DEQ has allowed for the measurement of $CBOD_5$ as an equivalent substitute for the biochemical oxygen demand (BOD) limits required by EPA. The $CBOD_5$ limits may be adjusted if more accurate equivalency becomes available; however, DEQ has not incorporated a different equivalency in the renewed permit to date.

Key Policies

DEQ policies related to point discharges require that growth and development be accommodated by increased efficiency of waste treatment such that future discharge waste loads do not exceed currently allowed loads (OAR 340-41-026[F]). Exceptions to this policy may be granted to major dischargers as long as new or increased discharges:

- Do not cause a water quality standard violation;
- Do not threaten or impair recognized beneficial uses;
- Do not discharge a pollutant for which the receiving water body is listed as water quality limited; and
- Are consistent with local land use plans, based on the activity necessitating the increased load.

Mass load increases have been granted for other Oregon municipalities but there may be pressure to limit increases in the future considering that the Willamette River is receiving more attention as part of the Governor's Oregon Plan for Salmon and Watersheds, in addition to the potential use of the river as a future drinking water source downstream.

Emerging Regulatory Changes

There are a number of regulatory changes under consideration or in early stages of implementation that may affect the Corvallis WWRP's ability to meet water quality standards. These include new or revised standards developed under the triennial review process, toxics criteria, criteria for sanitary sewer overflows, the 303(d) listing, listing of the Willamette River as Steelhead habitat, changes in the mixing zone regulations, the EPA proposal for nutrient criteria, and the Willamette River Reauthorization Study. These are discussed below.

Triennial Standards Review

DEQ is required to undergo review of selected water quality standards every 3 years and to set new or revise existing standards. During the last triennial review process, new standards for dissolved oxygen, temperature, bacteria, and pH (discussed above) were adopted by the Environmental Quality Commission (EQC). DEQ has not yet selected candidates for the next triennial review. A number of standards were identified, however, in early discussions about triennial review candidates. While it is uncertain when DEQ will review these standards, they could affect long-term planning efforts. These potential standards changes are summarized below.

Metals are currently regulated by Oregon standards as listed in Table 20 of OAR 340-41. DEQ adopted the metals criteria developed in EPA's *Quality Criteria for Water* (1986). Many of the metals criteria are expressed as total recoverable metals. Subsequent to the criteria development, EPA has determined that dissolved metals approximate the biologically available fraction of metals for aquatic organisms better than total recoverable metals and has recommended that the total recoverable metals criteria be converted, using partitioning factors, to dissolved metals criteria.

Historically, DEQ has used a comparison of instream total recoverable metals to water quality criteria to determine compliance. DEQ has recently changed this policy and recognizes that dissolved metals should be used for comparison to instream metals criteria but still uses the total recoverable metals criteria developed by EPA (the criteria have not been converted to dissolved metals criteria). DEQ's policy is to use total recoverable metal concentrations from point source discharges to assess compliance. These policies may change in the triennial review of metals, depending on EPA's action regarding metals criteria.

Sediments can accumulate pollutants originating from non-point and point discharges and can affect bottom-dwelling and bottom-feeding organisms directly or other aquatic organism indirectly through the food chain. There are no state sediment quality criteria; EPA has issued draft sediment criteria for five organic compounds. As part of ongoing inter-agency discussions on the listing of Steelhead as threatened in the Willamette River, the National Marine Fisheries Service has indicated that the Oregon Plan for Salmon and Watersheds would be acceptable to avoid a listing, if the state of Oregon adopted sediment criteria.

Additionally, EPA has issued a *Contaminated Sediment Management Strategy* (EPA, 1998) that discusses EPA goals to establish standard methods for chemical and biological test methods and to develop additional sediment criteria. Based on the state of scientific knowledge and controversy associated with contaminated sediments, the establishment of numerical sediment quality criteria may be difficult for the State of Oregon.

There are several ways that NPDES discharges could be affected by sediment criteria. If the State establishes numeric criteria, permits may reflect additional limits designed to protect sediment quality in addition to water quality. Another method the state may use to determine impacts to sediment quality is to require instream sediment bioassays near the discharge outfall. Testing of sediments could be included as part of future permit requirements.

Biological criteria are currently narrative standards that state "*Waters of the State shall be of sufficient quality to support aquatic species without detrimental changes in the resident biological communities*" (OAR 340-41-027). The triennial review could consider numerical or more specific narrative standards to measure "detrimental changes." Biological criteria may specify levels of changes in species abundance and diversity, or in physiological effects, such as the fish skeletal deformities identified in the Willamette River near the Newburg pool in recent DEQ water quality studies, that signify detrimental changes.

Chlorine is regulated as part of the water quality standards for toxics and as part of the treatment requirements. There is a national trend toward the elimination of chlorine use

because disinfection byproducts, such as chloroform and trihalomethanes, are potentially carcinogenic. DEQ recognizes that it would be a hardship to eliminate the use of chlorine at existing facilities, although DEQ may encourage new or upgrading facilities to use another means of disinfection. The high solids concentration and turbidity of CSO flows does not allow for consistent disinfection of these flow streams using current alternate disinfection technologies (e.g., ultraviolet (UV) inactivation, ozone). Also, because these flows tend to be intermittent and of relatively short duration, the concern over disinfection byproducts is not as great as for the continuous effluent flows from a typical sanitary treatment facility. Therefore, this regulatory emphasis is not expected to extend to disinfection of CSO flow streams in the near future.

Toxics Criteria

As part of the NPDES permitting process, DEQ does not currently evaluate human health criteria for the ingestion of water and fish (as listed in OAR Division 41 Table 20); however, other states such as Washington have begun to incorporate human health criteria into discharge permits. While human health criteria are not expected to be included in the next permit renewal, it could be a future issue. Some criteria, especially arsenic, are exceptionally low and often below background concentrations. EPA is currently evaluating some of the human health criteria to determine if they need to be revised. It is uncertain how the human health criteria will affect long-term planning.

Sanitary Sewer Overflows (SSOs)

SSOs are defined as the discharge of raw sewage (water carried human or animal waste) from residences, buildings, industrial establishments, or other places together with groundwater infiltration and surface water to waters of the State. During the last triennial review for bacteria, DEQ recognized that there are certain sources that are difficult and costly to control, such as SSOs. Therefore, the new bacteria standard provides remedies that utilize a technology-based approach for prescribed storm conditions. SSOs are prohibited, except during the following seasonal storm event conditions:

- Winter (November 1 to May 21): Greater than a one-in-five-year, 24-hour storm event
- Summer (May 22 to October 31): Greater than or equal to a 10-year/24-hour storm event

Facilities that experience sanitary overflows as a result of inflow and infiltration problems must submit an action plan that will prevent overflows by January 1, 2010. On a case-by-case basis, the beginning of winter may be defined as October 15 if it can be demonstrated that the risk to beneficial uses, including water contact recreation, will not be increased. Likewise, the beginning of summer may be defined as June 1 if no increased risk to beneficial uses is demonstrated.

Additionally, EPA issued a draft SSO policy in November 1996 which focuses on avoidable SSOs. The draft policy outlines EPA's preference for controls as (1) infiltration and inflow control, (2) expansion of conveyance and treatment capacity, and (3) storage. EPA is preparing a cost/benefit analysis for different versions of potential regulations.

303(d) Listing for the Willamette River

Section 303(d) of the Clean Water Act requires States to develop a list of water bodies that are not expected to meet State surface water quality standards after implementation of technology-based controls. The State is then required to develop a Total Maximum Daily Load (TMDL) program for water bodies on the 303(d) list. The TMDL program must address water quality on a basinwide approach to ensure that overall water quality standards, including water quality criteria and beneficial uses, be met. The TMDL establishes the total pollutant load that can enter the waterbody without violating water quality standards. The total load is then allocated to current and future point and nonpoint sources.

The 1998 303(d) list is currently in draft form and is expected to be finalized in September 1998. The draft list shows the Willamette River adjacent to Corvallis (from the Calapooia River to Long Tom River) as water quality limited for the following:

- Summer temperature
- Dioxin in aquatic tissue and the water column
- Mercury in aquatic tissue
- Fall to spring bacteria levels

DEQ has targeted TMDL development for the Willamette River in 2007. Significantly, the Clean Water Act prohibits additional discharges or increased discharges from existing permits until a TMDL has been established for 303(d) water bodies unless the discharge can be shown to have no measurable impact on beneficial uses.

Steelhead and Chinook Listings

The Upper Willamette River Steelhead trout and Chinook salmon stocks were proposed for listing (Federal Register, March 10, 1998) as threatened species. A final decision is not expected before February 1999; however, a species proposed for listing by National Marine Fisheries Service (NMFS) is afforded the same protection as if it were listed. Resource agencies involved in endangered species recovery, such as NMFS, may become more active in NPDES permit reviews. The listings will likely have an effect on toxics, temperature, and mixing zone requirements. It is uncertain how the listings will affect long-term planning for the WWRP.

Mixing Zone Rules

DEQ has historically allowed for a ZID for toxic compounds listed in OAR 340-41, Table 20. During development of the rule revision, some environmental interest groups asked that ZIDs not be allowed for anything but chlorine and ammonia. While this was not incorporated into the recent mixing zone rule revision, there may be public pressure to eliminate use of a ZID for other toxics in the future. This would affect discharge limits because acute and chronic criteria would need to be met at the end of the discharge pipe (with no dilution allowed). According to Bob Baumgartner/DEQ Northwest Region, there is no current movement within DEQ to change this policy. However, the environmental community remains interested in this issue and DEQ's policies regarding toxics and mixing zones.

EPA Clean Water Action Plan

EPA's *Clean Water Action Plan: Restoring and Protecting America's Waters* (February 14, 1998, report to the Vice President of the United States from Carol Browner/EPA and Dan Gluckman/USDA) formulates a plan to fulfill the original goal of the Clean Water Act through education, stewardship, and new standards. As part of the plan, EPA proposes to establish by the year 2000 "numeric criteria for nutrients (i.e., nitrogen and phosphorus) that are tailored to reflect the different types of water bodies (e.g., lakes, rivers, and estuaries) and the different ecoregions of the country, and will assist the States in adopting numerical standards based on these criteria over the following three years."

This action plan has not been implemented; congressional funding for the plan will be decided in the fall of 1998. It is uncertain if Congress will fund EPA's Action Plan, and, therefore, if the plan elements will be implemented. Even if the plan is not funded, it appears that EPA is headed toward establishing nutrient standards some time in the future. This could mean that point sources would be faced with nutrient limits in the future.

Willamette River Reauthorization Study

The U.S. Army Corps of Engineers (Corps) and the Oregon Water Resources Division (OWRD) is undertaking a reauthorization study to evaluate water allocations in the Willamette River. Under its current prime authorization, the Corps is responsible for the operation and maintenance of its reservoirs for navigation, flood control, hydropower, and irrigation. Water allocations for other uses, such as fisheries, water quality, and recreation are not explicitly authorized. The reauthorization study objective is to identify all the water uses in the Willamette Basin and determine how the operations and maintenance of the Corps-operated reservoirs can best meet the needs of all users. This long-term study is in the preliminary stages and is expected to continue for 4 years.

The potential effects of the study are unknown at this time; it may either increase or decrease flows near the Corvallis WWRP. If flows decrease, it would effectively reduce the dilution obtained within existing mixing zones and could cause noncompliance for existing discharges. DEQ is concerned that decreased summer flows would have the largest effect on instream algae and dissolved oxygen concentrations. If flows increase, the dilutions obtained would be beneficial for dischargers.

The Corps and OWRD recognize that decisions made for water reallocation will affect permitted effluent dilutions. One of the objectives of the study is to provide schedules for dischargers and other users affected by changes in operations of the reservoirs for planning purposes. Any changes in base flows that would occur would be addressed when the permit comes up for renewal.

Effluent Quality Evaluation

The current and anticipated effluent quality and potential water quality issues related to discharge to the Willamette River are addressed below for treatment standards and water quality criteria.

Treatment Standards

The existing NPDES municipal discharge permit limits, which reflect current treatment requirements, were compared to existing conditions and future (under 60,000 population conditions). Table 2-2 shows the existing and future maximum effluent concentrations and mass loads compared with the NPDES permit limits assuming no increase in the treatment standards at the WWRP. The Corvallis WWRP effluent has consistently met treatment requirements, except for a brief period in late July/early August 1991. During this period, maintenance was being performed at the time a process upset occurred. These conditions are not expected to occur on a regular or predicted basis. Future compliance at the 60,000 population level would meet effluent concentration limits, but would exceed mass load limits. Therefore, to maintain the current effluent concentrations for CBOD and TSS an increase in the permitted mass load limits would be required. Under current regulations, this mass load increase can only be granted by the EQC based on the results of an overall water quality analysis (i.e., it will be a negotiated limit). The results of these negotiations with DEQ will determine the treatment standards required for future plant expansions. It should be noted that mass load limits will become an issue to be addressed with DEQ within the next two to three NPDES permit renewal periods (i.e., 10 to 15 years).

TABLE 2-2
Compliance Evaluation of City of Corvallis WWRP Treatment Requirements

Parameter	Treatment Requirements/ Current Permit Limits				WWRP Effluent Quality: Current Population[a]	WWRP Effluent Quality: 60,000 Population[b]
$CBOD_5$						
Dry Weather	Daily maximum	(lb)	=	1,600	1,094	4,390
	Weekly average	(mg/L)	=	15	15.7[c]	15
		(lb/d)	=	1,200	965	2,240
	Monthly average	(mg/L)	=	10	12.5[c]	10
		(lb/d)	=	810	513	1,259
Wet Weather	Daily maximum	(lb)	=	7,000	3,345	9,340
	Weekly average	(mg/L)	=	40	20	30
		(lb/d)	=	5,700	2,018	7,005
	Monthly average	(mg/L)	=	25	16	20
		(lb/d)	=	3,500	1494	3,880
TSS						
Dry Weather	Daily maximum	(lb)	=	1,600	1,458	4,390
	Weekly average	(mg/L)	=	15	20[d]	15
		(lb/d)	=	1,200	965	2,240
	Monthly average	(mg/L)	=	10	12[c]	10
		(lb/d)	=	810	449	1,259
Wet Weather	Daily maximum	(lb)	=	8,600	5269	9,340
	Weekly average	(mg/L)	=	45	15	30
		(lb/d)	=	6,400	2755	7,005

TABLE 2-2
Compliance Evaluation of City of Corvallis WWRP Treatment Requirements

Parameter	Treatment Requirements/ Current Permit Limits		WWRP Effluent Quality: Current Population[a]	WWRP Effluent Quality: 60,000 Population[b]
Wet Weather (continued)	Monthly average (mg/L) =	30	12	20
	(lb/d) = 4,300		1,596	3,880
Removal Efficiency	≥ 85% monthly average when monthly avg. flow ≤12 mgd	$CBOD_5$	98	98
		TSS	99	98
	≥ 75% monthly average when monthly avg. flow >12 mgd	$CBOD_5$	97	95
		TSS	99	95
pH	≥6 and ≤9[f]		6.0 to 8.1	6.0 to 8.1
Bacteria				
Fecal Coliform	≤ 200 CFU/100 mL as 30-day geometric mean		35	
	≤ 400 CFU/100 mL as 7-day geometric mean		258	
E. coli	≤ 126 org/100 mL as 30-day log mean			≤ 126
	≤ 406 org/100 mL as daily maximum			≤ 406

[a] Plant effluent data summarized from the following:

Samples 3 times per week for $CBOD_5$ (Nov. 1990 to Sept. 1996)

Daily samples for TSS (Nov. 1990 to Sept. 1996)

Daily Samples for pH (Nov. 1990 to Sept. 1996)

Samples 1 per week for bacteria (Nov. 1990 to Oct. 1992) and 3 per week (Oct. 1992 to Sept. 1996)

Treatment limits are implied as moving averages although compliance with permit limits typically evaluated from static weekly or monthly averages, as reported in the DMRs. Moving averages (which typically result in slightly higher values) conservatively used to evaluate potential compliance. Using static averages, the results would be similar, but the number of exceedances would be reduced.

[b] From flow projections and process modeling data.

[c] Exceedance of permit limit only during early August 1991; all other time periods, effluent data below permit limits.

[d] Exceedance of permit limit only during late July 1991; all other time periods, effluent data below permit limits.

[f] Effluent shall not be outside of shown range for more than 7 hours and 26 minutes in any calendar month; and no individual excursion from this range shall exceed 60 minutes.

Removal efficiency, bacteria, and pH have consistently met current permit limits and are expected to be in compliance under all future planning scenarios.

Potential Issues. Effluent concentrations of CBOD and TSS would need to decrease in the future to meet current mass load limits, or a permitted increase in mass load limits would be required. It would be difficult to decrease the CBOD and TSS concentrations to a sufficient level to accommodate growth expected beyond the 60,000 population scenario without providing additional treatment at the WWRP. The practical limit for CBOD and TSS, under current technologies, is about 5 mg/L but even this concentration would not meet mass load limits at the 70,000 population level. Therefore, it is likely that both an increase in the plant's

treatment standards and an increase in the permitted mass load limit will be required in the future.

In addition, the current permit allows for the measurement of $CBOD_5$ as an equivalent substitute for the BOD limits required by EPA. The $CBOD_5$ to BOD relationship should be evaluated to determine if it is less than a 1:1 relationship. This would effectively allow greater amounts of $CBOD_5$ to be discharged without increasing the BOD mass load limits.

Instream Water Quality Standards

Water quality standards are addressed for general parameters and for toxic substances. General parameters include dissolved oxygen, temperature, and bacteria, while toxic substances include metals, chlorine, and ammonia.

Dissolved Oxygen

DEQ, as part of the 303(d) review of river water quality, did not identify this segment of the Willamette River as having a dissolved oxygen problem. No limits are anticipated.

Potential Issues. Dissolved oxygen is not expected to be an issue for future permit limits unless increased mass loads are required. If this happens, the WWRP will need to demonstrate that this will not negatively affect dissolved oxygen concentrations.

Temperature

This segment of the Willamette River has been designated as water quality limited for temperature in DEQ's 303(d) report. DEQ's temperature standard implementation guidance for point source requires the permit writer to conduct an analysis to determine the estimated change in stream temperature as a result of the point discharge. If the point discharge does not cause a measurable increase in temperature (defined as less than or equal to 0.25 °F) at the edge of the mixing zone, then compliance with the NPDES permit will be considered to satisfy the requirement for developing and implementing a temperature management plan (i.e., a management plan is not required).

To verify no measurable increase in temperature, a mass balance for heat is conducted, assuming that temperature is a conservative parameter. Existing influent data were used to estimate the temperature increase caused by the WWRP effluent and to evaluate whether DEQ may require a temperature management plan. DEQ's guidance for mass balance calculations is to estimate the temperature change using the dilution ratio at the edge of the mixing zone. The DEQ-approved mixing zone study (CH2M HILL, November 25, 1997) modeled several scenarios representing overlapping mixing zones using the maximum daily flow. Resulting dilution estimates ranged from 50:1 to 79:1. For the temperature evaluation, the dilution of 79:1 was used because the temperature standard is a 7-day average and the 79:1 dilution is more representative of the 7-day average dilution at the mixing zone boundary.

The mass balance for temperature was calculated for six year (1991 to 1996) and the temperature change at the edge of the mixing zone was estimated based on the annual maximum dry weather (May through October) 7-day moving average influent temperature (Table 2-3). For all six years, effluent temperatures do not appear to have an effect on downstream temperatures (less than 0.25°F increase at the edge of the mixing zone). Even under a conser-

vative 50:1 dilution at the edge of the mixing zone, the mass balance calculation for temperature indicates no significant increase in instream temperature.

TABLE 2-3
Mass Balance Calculations for Temperature

The mass balance equation for temperature is as follows:

$$T_{mz} = [(D_s * T_s) + (D_e * T_e)]/ (D_s + D_e)$$

where,

T_{mz} = the temperature at the edge of the mixing zone

D_s = instream flow (in relation to effluent flow) = 79 (estimated during 7Q10 flow)

T_s = instream temperature = 71.6°F (maximum temperature in August 1992)

D_e = effluent flow (in relation to instream flow) = 1

T_e = effluent temperature (maximum temperature in each year)

	Instream		Effluent			
Year	D_s	Ts (°F)	D_e	T_e (°F)	Mixing Zone T_{mz} (°F)	Temperature Change (°F)
1991	79	71.6	1	69.1	71.57	-0.03
1992	79	71.6	1	81.4	71.72	0.12
1993	79	71.6	1	71.6	71.60	0.00
1994	79	71.6	1	73.7	71.63	0.03
1995	79	71.6	1	71.6	71.60	0.00
1996	79	71.6	1	70.8	71.59	-0.01

Potential Issues. Recently, EPA has commented to DEQ as part of the South Umqua TMDL development that the instream standard should be used rather than the measured instream temperature (T_s). There is internal DEQ discussion of whether they will change the implementation guidance to reflect this comment. If the standard were used (64°F) rather than the instream maximum temperature (71.6°F) and the lower dilution at the edge of the mixing zone was assumed (50:1), the resulting mass balance calculations would indicate a potential for increasing the instream temperature greater than 0.25°F based on data collected in 1992. The WWRP influent temperatures are shown in Figure 2-1.

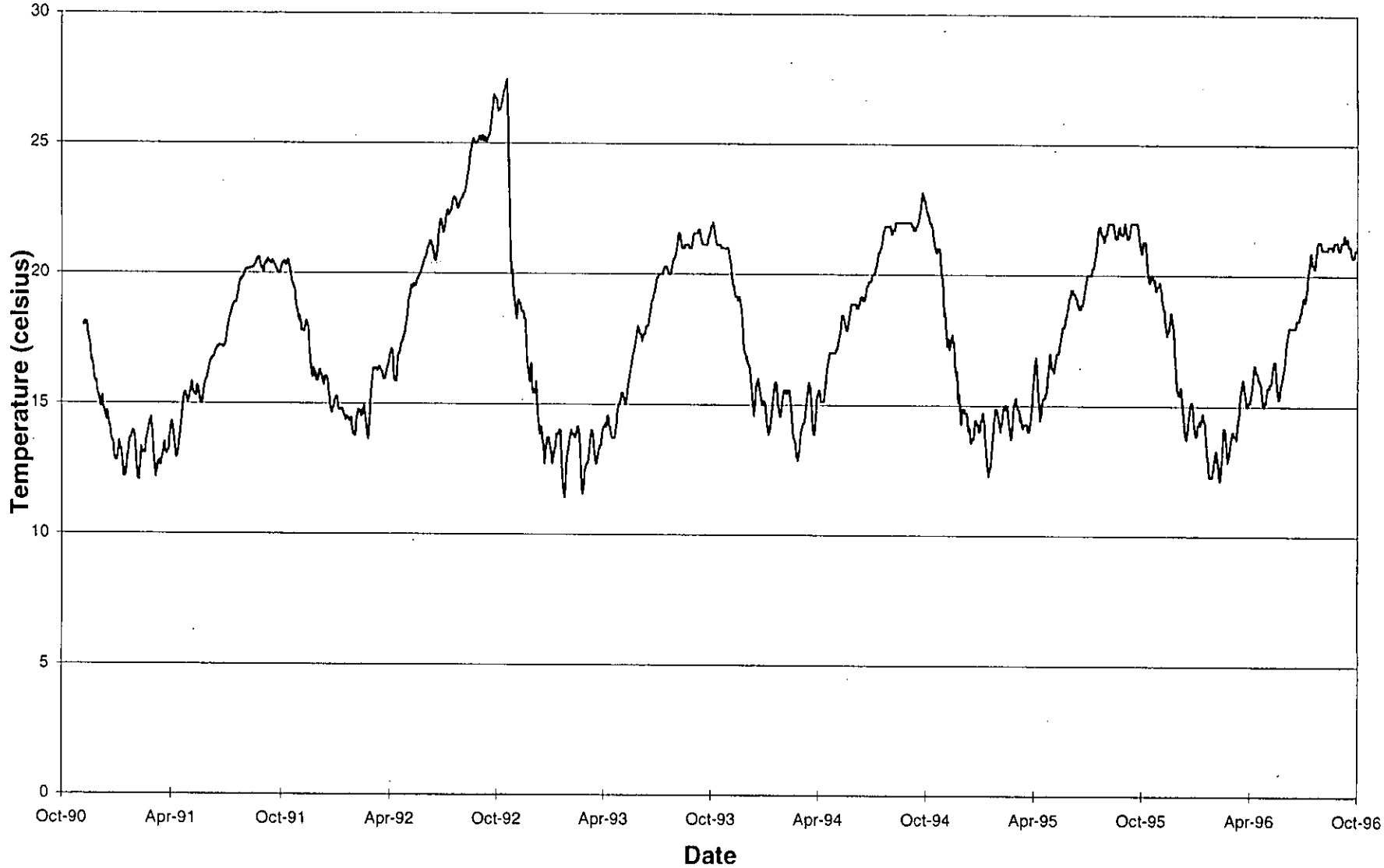
During the drought year of 1992, the WWRP effluent temperatures were higher than typical. Additional data collection is advised to evaluate how representative influent temperature is of effluent temperatures and the potential mixing that would occur during drought conditions.

While no numerical limit is anticipated, impacts on receiving water temperatures should be a consideration for selection and design of treatment processes to ensure that effluent temperatures do not increase under future conditions.

Bacteria

The new bacteria standard is based on *E. coli* and should be included in the permit upon renewal. However, the Corvallis WWRP permit is still based on fecal coliform. It is

Figure 2-1
Corvallis WWRP 7-Day Average of Influent Temperature



anticipated that the next (1998) NPDES permit will be based on the *E. coli* bacteria standard. On the basis of current compliance with the fecal coliform permit limit and limited *E. coli* data, compliance with the bacteria standard is expected.

Potential Issues. Future permits will be based on *E. coli*, but compliance is expected.

Metals

There are currently no permit limits for metals. A compliance evaluation of metals for current and future conditions was conducted as part of the *Combined Discharge Evaluation of Corvallis Wastewater Reclamation Plant Outfall and New Treated-CSO Outfall* (CH2M HILL, November 25, 1997). It indicated that under the 60,000 population scenario, metals are expected to be in compliance.

Potential Issues. The only potential concern was silver, but this is likely a result of high detection limits and one questionable sample result. If this one sample result is eliminated, the effluent would be in compliance. Detection limits have been lowered in recent years and recent data indicate that silver is not likely to be a problem. Silver was analyzed using standard sampling and analytical methods used for routine metals monitoring. Additional silver analyses, using "clean" sampling and analytical techniques is recommended to demonstrate that silver is not a compliance issue.

Emerging or potential regulatory changes, such as the ones discussed above (lower discharge limits from the application of human health criteria, changes in mixing zone rules, or requirements derived from endangered species concerns) could require metal limits in the permit, but the levels are uncertain.

Chlorine

The Corvallis WWRP chlorinates the plant effluent before discharging to the Willamette River, and must limit the chlorine residual in the effluent to below 0.5 ppm on a daily average. The plant has consistently been in compliance since mid-1992 when the limit was established and it is anticipated that it will remain in compliance in the future.

Potential Issues. The need for higher chlorine residuals to meet the *E. coli* bacteria standard or reductions in discharge limits in future permits could mean that dechlorination will be required for the WWRP effluent. Dechlorination facilities are being provided as part of the wet weather treatment facilities.

In addition, there is a national trend toward the elimination of chlorine use because disinfection byproducts, such as chloroform and trihalomethanes, are potentially carcinogenic. DEQ recognizes that it would be a hardship to eliminate the use of chlorine at existing facilities, although DEQ may encourage new facilities, or facilities that are being upgraded, to use another means of disinfection. Discussions with DEQ need to occur during planning of any disinfection facility expansion to determine DEQ's position on chlorine use and outline the City's need to continue the use of chlorine disinfection for the treatment of CSO flows.

Ammonia

Ammonia concentrations have been consistently below water quality criteria. Compliance is expected to continue in the future.

Potential Issues. Although ammonia toxicity is not considered to be a concern for the 60,000 population discharge scenario, ammonia does have oxygen demand. If the WWRP needs to reduce BOD mass loads, additional treatment of ammonia may need to be considered to reduce overall loads. Ammonia as a nutrient is addressed below.

Nutrients

Phosphorus and nitrogen (including ammonia and nitrate) compounds can enrich the receiving waters, causing eutrophication. Currently, nutrients do not appear to be of concern in this stretch of the Willamette River and no limits are expected in the near term.

Potential Issues. If EPA develops nutrient criteria and Oregon adopts these criteria, limits on the discharge of nutrients are possible. The development of nutrient criteria has not been initiated, so it is uncertain what limits might be required. Nutrient limits are not likely during wet weather conditions, when river flows are high, but limits may be imposed during dry weather conditions.

Recommended Planning Criteria

Three planning scenarios were developed. This was done because of the high level of uncertainty regarding potential regulatory issues and pressures to address the potential affects of the WWRP on the immediate neighborhood (environmental adjacency issues). The planning scenarios represent three levels of criteria, as follows:

- Current Conditions
- Most Probable
- Most Stringent

The Current Conditions Scenario assumes that the regulatory requirements for the WWRP will remain unchanged during the planning period, and that no additional mitigation measures will be required to address adjacency concerns. Given the regulatory trends that currently exist and the continued growth around the treatment plant site, this scenario would seem unlikely.

The Most Probable Scenario addresses the planning and facility development that would need to occur to address the regulatory trends that are currently underway in Oregon and in the Willamette Valley. This scenario also considers the values of the Corvallis community regarding community livability and environmental protection and issues being dealt with in other similar communities in establishing the level of environmental adjacency mitigation.

The Most Stringent Scenario represents the greatest level of facility investment that would be required given the highest level of treatment currently being considered in the regulatory arena and the greatest level of mitigation that might be required to respond to the community.

In developing the master plan for the WWRP, each of these planning scenarios was considered in determining future capital improvements and the associated costs. This provides the City with a planning document that is flexible enough to address a broad array of regulatory changes or concerns in the community.

Effluent Planning Scenarios

The recommended effluent planning criteria are provided in Table 2-4. Under the Most Probable Scenario it is assumed that some CBOD/TSS mass load increase will be negotiated with the DEQ through a total water quality assessment. It is further assumed that this negotiation will require summertime nitrification to justify an increase in the BOD load to the receiving water. Nutrient limits, a temperature management plan, and/or higher levels of biosolids stabilization are not considered to be likely during the planning period.

TABLE 2-4
Regulatory Planning Criteria

Component	Current Conditions Scenario	Most Probable Scenario	Most Stringent Scenario
CBOD/TSS			
Mass Limits	Increase corresponding to concentration limits at future flows	Negotiated increase in limits based on environmental assessment	No increase allowed in existing mass limits
Concentration Limits	No change from current (10/10 summer, 25/30 winter)	8/8 summer, 20/20 winter	5/5 summer, 10/15 winter
Ammonia	No limit	Summer limit only – to reduce BOD load	Year round limit - to reduce BOD load
Temperature	No limit	No limit	Summer mitigation measures
Bacteria	Fecal Coliform <200 CFU/100 mL as 30-day mean	E. coli < 126 org/100 mL as 30-day mean	E. coli < 126 org/100 mL as 30-day mean
Toxicity			
Chlorine	Limit = 0.5 mg/L	Limit < 0.5 mg/L	Eliminate by-products of chlorination and dechlorination
Ammonia	River provides adequate dilution	Summer limit only	Year round limit
Metals	River provides adequate dilution	River provides adequate dilution	Year round limit
Nutrients			
Ammonia	No limit - DO depletion and algae growth within acceptable limits	No limit	Summer limit only
Nitrate	No limit	No limit	Summer limit only
Total Nitrogen	No limit	No limit	Summer limit only
Phosphorus	No limit (actual effluent P abnormally high)	No limit, but reduce effluent P to typical levels	Summer limit only
Biosolids	Class B land application	Class B land application	Class A land application

The Most Stringent Scenario was based on no increase in CBOD/TSS mass load being allowed by DEQ, summer limits being required for nutrients and metals, and a summertime temperature management plan being developed to establish mitigation measures. In addition, it is assumed that the most stringent regulatory environment might require Class A stabilization of biosolids and/or the complete elimination of the by-products of chlorination.

Environmental Adjacency Planning Scenarios

Environmental adjacency issues are those issues that effect or are driven by the surrounding community, but are not governed by the WWRP's operating regulatory permit requirements. These issues include odor, noise, traffic, visual/aesthetic, and safety. Table 2-5 shows the environmental adjacency planning scenarios.

Under the Most Probable Scenario, it is assumed that mitigation measures for odor generation and elimination of gaseous chlorine and sulfur dioxide to address safety concerns will be required. Odor control measures will be applied to the primary emission sources under this planning scenario. It is anticipated that no mitigation of noise, traffic, or visual impacts will be required.

The Most Stringent Scenario is based on the following:

- Control of odorous emissions from both primary and secondary sources.
- Reduction in truck traffic as a result of sludge hauling and chemical deliveries.
- Visual screening of the WWRP and/or improvements to the aesthetics of the plant facilities.
- Eliminate or minimize the use of chemicals on the plant site.

TABLE 2-5
Environmental Adjacency Planning Criteria

Component	Current Conditions Scenario	Most Probable Scenario	Most Stringent Scenario
Odor	No change	Control primary emission sources	Control secondary emission sources
Noise	No change	No change	No change
Traffic	No change	No change	Reduce sludge hauling frequency, minimize chemical use
Visual	No change	No change	Aesthetic improvements, visual screening
Safety	EPA RMP implementation	No gaseous chlorine or sulfur dioxide	Eliminate/minimize all chemical use

RMP = Risk Management Plan

SECTION 3

Collection System Hydraulic Analysis

Background

Development of a sanitary sewer collection system for the City of Corvallis began in 1899 with the authorization to construct sewer trunk lines at Van Buren and Jefferson Avenues. Both of these lines were completed in 1902. The Jackson Street trunk was built at about the same time and subsequently reconstructed of cast-in-place concrete pipe in 1924. The Jefferson trunk was subsequently rebuilt in 1938. The Western Boulevard trunk and the lower part of the Fillmore trunk were built in 1911 and 1912 as cast-in-place concrete structures. Further extensions of the Fillmore trunk were constructed in 1917 and 1920 to accommodate residential growth. The original Van Buren trunk is still being used today and is the oldest sewer system in service in Corvallis.

These first City sewers were designed and constructed for "combined" use, to collect and convey both sanitary sewage and stormwater runoff directly to the Willamette River. At that time, discharging sewage wastes into the river was considered standard practice and a single pipe to convey both sewage and stormwater was generally accepted as the most economical and efficient means of disposal. This practice was common throughout the United States. As interest and concern about the quality of Willamette River water increased, reducing the quantity of untreated wastewater being discharged into the river became an important issue. Ultimately, the State of Oregon intervened and established statewide standards for municipal wastewater discharge. New construction of combined sewers became illegal.

All sewers constructed in Corvallis since 1951 have been designed to collect and convey stormwater and sanitary sewage in separate systems. The 1st Street interceptor and the Beca trunk were the first sanitary sewer systems to be constructed in Corvallis using this new approach. The first elements of the Corvallis Wastewater Reclamation Plant (WWRP) were also built at this time and were designed to provide treatment only for the sanitary sewage flows. The 1st Street interceptor collected the sanitary sewer flows from all of the trunks existing at that time and conveyed them to the new treatment plant. The Beca trunk extended the sewer service area into the developing area north of the Fillmore service area and discharged directly into the new treatment plant.

In 1956 a major expansion of the sanitary sewer collection system was constructed along Garfield Avenue, from 9th Street to the west, in order to provide service for the rapidly expanding northwest residential area. Over the years, secondary trunks have been added to the main Garfield trunk to form the largest sewer service area in the City, encompassing most of the Timberhill and Witham Hill subdivisions and much of the area north of Garfield Avenue and west of Highland Avenue.

In 1963, growth and development in the southwestern residential area and expansion of the Oregon State University campus required additional sewer service, but the Western Boulevard trunk could not be extended to serve the area. The Marys River interceptor and

Brooklane pump station were built to provide this service, discharging flows into the 1st Street interceptor at Western Boulevard.

In the mid-1960s significant land development was also occurring in South Corvallis along South 3rd Street (State Highway 99W) and service was provided by the South Corvallis trunk. The South Corvallis trunk was constructed in three segments in 1964 and 1965 and extends south from the Marys River lift station. The Marys River lift station discharges its flows into the Marys River interceptor on the north side of Marys River at 3rd Street. To treat the additional wastewater collected the wastewater reclamation plant was also expanded.

In 1970 the residential neighborhood around the Corvallis Country Club was identified by the Benton County Health Department as a health hazard area because of inadequate individual sanitary waste disposal systems and the City was requested by local residents to annex the area and to provide public sewers for sanitary wastewater collection and disposal. The Brooklane and Country Club trunks were constructed in 1972 with federal financial assistance to provide the necessary sewer service and eliminate the health hazard.

In 1964 and 1965 the area of northeast Corvallis experienced major residential growth. Lack of a major trunk to this area required that sewage be pumped up into the North 9th Street trunk. Development was allowed to continue until the system became overloaded, creating backups and consequent maintenance problems for the City. Federal financial assistance was sought and secured in 1978 for the construction of the first phase of the Crescent Valley interceptor from the treatment plant to Village Green and the subsequent removal of the Village Green pump station. A corresponding expansion of the wastewater reclamation plant also occurred.

These latter sewer system improvements occurred in basins generally surrounding the original combined sewer areas. The combined basins have remained relatively unchanged with the combined sewer outfalls along 1st Street at Fillmore, Polk, Van Buren, Jackson, and Western Avenues providing hydraulic relief during major rainfall events. The outfalls enable the discharge of excess wet weather flows into the Willamette River during rainfall events.

As the above discussion shows, the wastewater collection system has continued to evolve, and is currently a mixture of new and old systems that exhibit varying levels of performance. The following describes the collection system as it exists at present.

Collection System

The main elements (pipe diameters 10 inches and greater) of the City of Corvallis' wastewater collection system are illustrated in Figure 3-1. For the purposes of this study, the Corvallis combined and separated sewage collection system are divided into five major subsystems whose trunk lines are 10 inches or more in diameter.

The major pipeline systems are located within the sewer basins shown in Figure 3-1. The pipeline systems and their respective tributaries are as follows:

- 1st Street interceptor
  - Van Buren trunk
  - Jefferson trunk

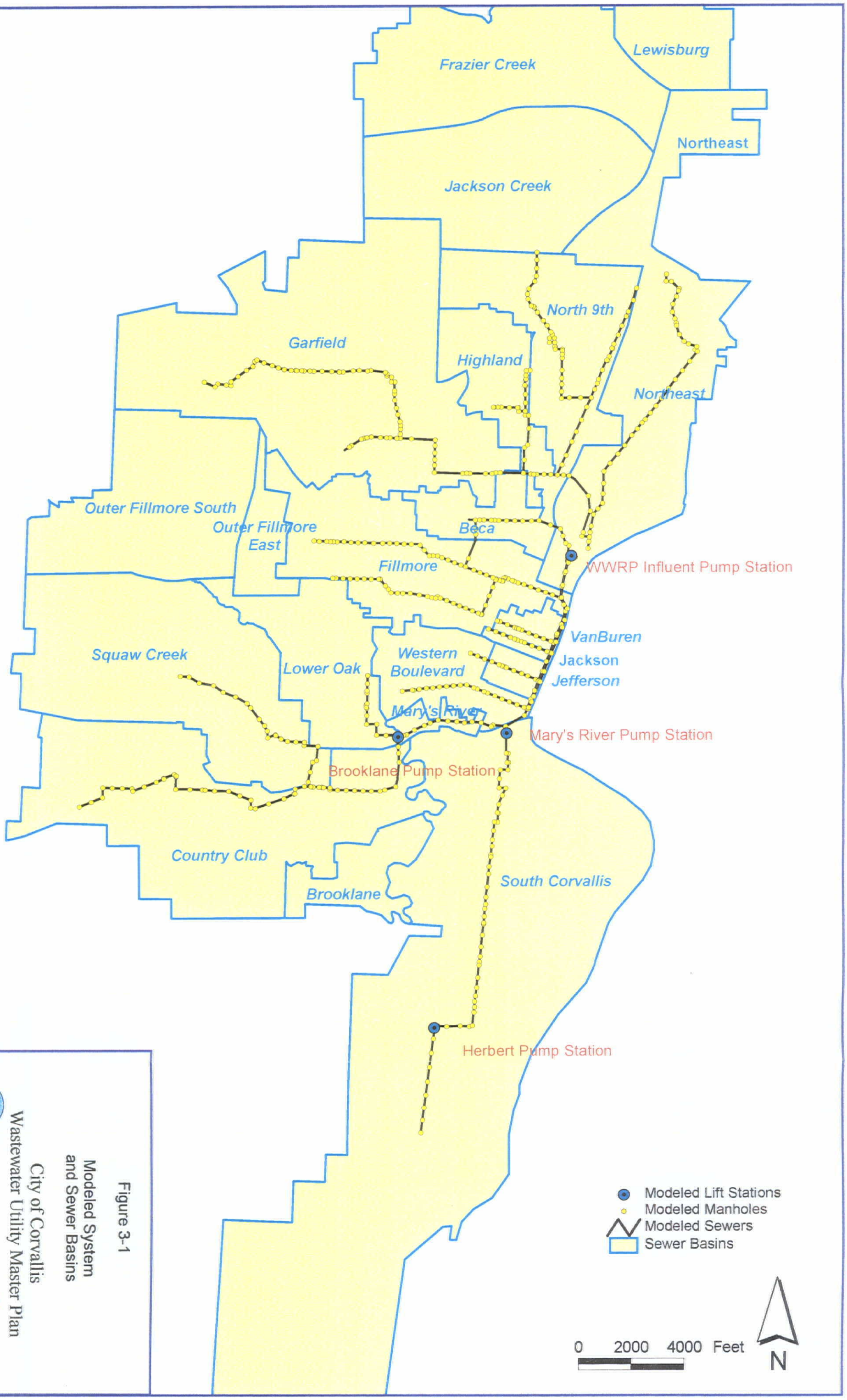


Figure 3-1
Modeled System
and Sewer Basins
City of Corvallis
Wastewater Utility Master Plan

  - Jackson trunk
  - Western Boulevard trunk
  - Fillmore trunk
- Marys River interceptor
  - Brooklane trunk
  - Lower Oak Creek trunk
  - Country Club trunk
  - South Corvallis trunk
- Garfield interceptor
  - Garfield trunk
  - Highland trunk
  - North 9th Street trunk
- Beca trunk
- Crescent Valley interceptor

The sizes, lengths, and types of the existing interceptors and trunk lines are summarized in Table 3-1.

TABLE 3-1
Existing Sewage Collection System (Based on City of Corvallis 1979 Sanitary Sewer Plan; mapping created by City of Corvallis Engineering Dept., May 1994 and 1997 City GIS data)

Interceptor/ Trunk	Size (inches)	Length (feet)	Type	Combined/ Separated
Beca	12-18	13,100	Concrete	Separated
Country Club	12-30	9,200	Concrete	Separated
Crescent Valley	36	10,300	Concrete	Separated
Fillmore	18-76*	29,650	Monolithic concrete	Combined
1st Street	30-36	10,100	Concrete	Combined
Garfield	30-36	37,950	Concrete	Separated
Highland	12-18	11,450	Concrete	Separated
Jackson	10-30	4,500	Poured in place	Combined
Jefferson	12-18	5,500	Clay Tile	Partially separated
Marys River	30	8,750	Concrete	Separated
North 9th St.	8-21	17,600	Concrete	Separated
South Corvallis	12-21	13,100	Concrete	Separated
Van Buren	8-22	4,500	Clay Tile	Combined
Western	24-46*	8,300	Monolithic concrete	Combined

* Sizes are equivalent size of round pipe.

1st Street Interceptor System

The 1st Street interceptor was built in 1951 in conjunction with the Corvallis Wastewater Reclamation Plant (WWRP) to intercept flows from the five original trunk sewers before they reach the Willamette River and convey them to the treatment plant. The 30- to 36-inch-

diameter concrete pipe was designed with the capacity to intercept three to four times the average dry weather flow from the original combined and separated sewers. Any wet weather flows of greater magnitude will discharge directly to the Willamette River through the diversion manholes at the junction of each combined sewer. Figure 3-2 illustrates this system.

Van Buren Trunk

The Van Buren trunk, constructed in 1902, is the oldest sanitary sewer trunk still in use in the City. The original clay tile sewer is still in place, but was identified in the "1975 Sewer Investigation" as being in very poor condition. The report refers to broken and flattened tile and high rates of infiltration through the mains and mortared brick manholes. Maintenance costs on the system are high because of its age and advanced stage of deterioration. The Van Buren trunk is a combined system with the sanitary sewer also carrying storm flows. Along its 4,500-foot length it ranges in diameter from a maximum of 22 inches at its junction with the 1st Street interceptor to a minimum of 8 inches at its upstream terminus at 9th and Van Buren Streets. This trunk is reported to have many problems, with some structural failures.

Laterals in the service area are made of 1902 to 1908 vintage red clay tile. Pipe conditions vary from acceptable to poor, with the better laterals located south of Van Buren. These laterals are fairly round, structurally sound, and have only minor root intrusion problems. The laterals north of Van Buren are invaded with roots, and numerous service line hubs extend into the laterals.

Jefferson Trunk

The original Jefferson trunk was built in 1902 along with the Van Buren trunk but was subsequently abandoned, and a new clay tile sewerline was constructed in 1938. The laterals in the area are predominantly made of 1902 to 1907 vintage red clay tile and suffer from severe root intrusion and some structural failure. The 5,500-foot trunk line ranges in size from 12 to 18 inches in diameter. Its overall condition is considered poor to fair.

The trunk was originally built as a combined sewer, but in 1964 separate storm drains were built in Jefferson, Adams, and Madison Avenues. These storm drains have not been totally effective in eliminating storm flows into the sanitary sewer. The area tributary to the Jefferson trunk has been reduced from its original size by the construction of a bypass line to the Western Boulevard trunk. Most of the combined flows from the area west of Eighth Street and south of Monroe are diverted to the Western Boulevard trunk. The "1975 Sewer Investigation" identified this trunk as containing more sewers in immediate need of attention than any other service area.

Jackson Trunk

The Jackson trunk was first constructed in 1902 and reconstructed in 1924, using cast-in-place concrete pipe ranging in size from 10 to 30 inches in diameter for a total length of 4,500 feet. This trunk functions as a combined sewer, and its overall condition is reported to be fair. This trunk has fewer and shorter service laterals than any of the other service areas in the city. The laterals are generally made of circa 1902 red clay tile, but are in fairly good condition because there are fewer trees, and consequently fewer root intrusion problems than in adjacent service areas. The joints are reasonably tight and infiltration and root

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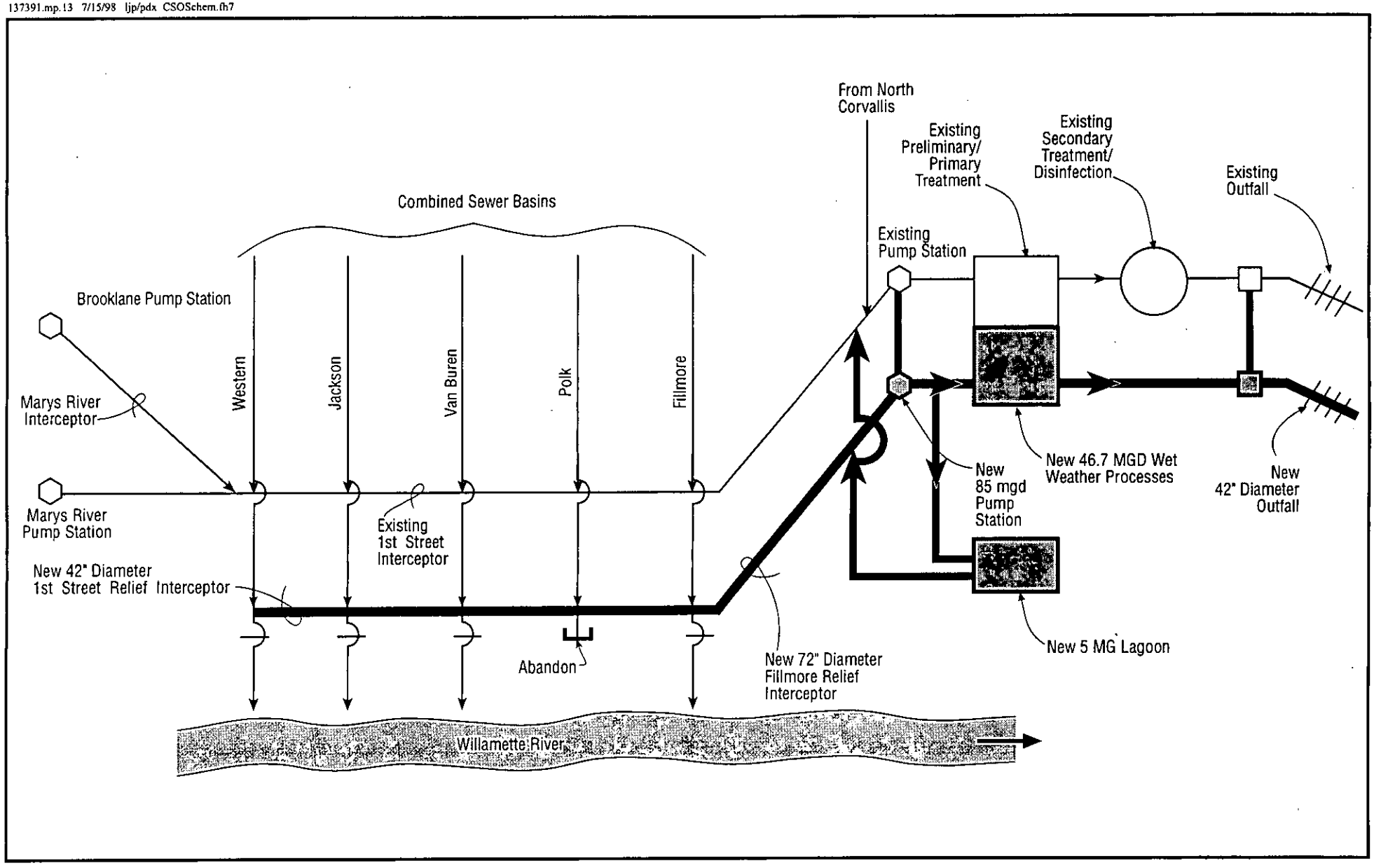


FIGURE 3-2
1st Street Interceptor CSO Control System Schematic

intrusion are not problems. The trunk itself is structurally sound and the system is relatively free from maintenance problems.

Western Boulevard Trunk

The Western Boulevard trunk is a cast-in-place concrete structure built in 1911 that has the approximate capacity of a 46-inch-diameter circular pipe at its downstream end and is still structurally sound. The total length of the trunk is 8,300 feet. Almost all of the laterals from 16th Street to the Willamette River are made of clay tiles installed in 1911. These clay tiles are extremely brittle and were laid with little or no gravel bedding. Many joints are open, alignment is poor, service hubs extend into the laterals, and root intrusion is common.

The upper reaches of this trunk system serve the Oregon State University campus. Maintenance of those laterals within the campus area is the responsibility of the University. There are several areas of the campus that are served by combined sewers and thus contribute wet weather flows to the City's sanitary sewer system during rainfall events.

Fillmore Trunk

The lower portion of the Fillmore trunk system is a combined sewer. It was constructed in 1911 and 1912, and consists of cast-in-place concrete conduit of the same shape as the Western Boulevard trunk. This section of the trunk system extends from the river to 11th Street and then splits into two branches, one going to Van Buren then to Harrison and 29th, and the other following Taylor to Kings Boulevard. An extension of the Harrison/Van Buren branch to Harrison and 35th Street was constructed in 1917, and an extension of the Taylor branch to Fillmore and 36th Street followed in 1920. Both extensions were constructed of round precast concrete pipe. The trunk sewers are structurally sound, and few root problems have been identified.

Laterals on the Taylor branch from 1st Street to 19th Street, and on the Harrison/Van Buren branch to 29th Street are generally constructed of 1912-1913 era clay tile. The laterals were built on back property lines and have only moderate hub and root problems. There have been isolated problems of pipe failure, but in general the structural quality of the laterals is good and the joints remain relatively tight. The laterals west of 19th Street on the Taylor branch were constructed of precast concrete pipe aligned in the street areas and are structurally sound with minimal amounts of infiltration.

Marys River Interceptor System

The Marys River interceptor connects to the upstream end of the 1st Street interceptor at Western Boulevard and conveys flows from the Brooklane and Marys River pump stations. The interceptor itself is a 30-inch-diameter concrete pipe constructed in 1963 and is structurally sound. The interceptor length is 8,750 feet. This is a separated system with no connections for stormwater.

Country Club

The Country Club trunk consists of approximately 9,200 feet of compression jointed concrete pipe ranging in size from 12 to 30 inches in diameter. This line discharges into the Brooklane pump station and from there to the Marys River interceptor. This system was designed in 1972 as a separated system handling only sanitary sewage flows. There have

been no significant maintenance problems within this system. However, the recent flow monitoring study identified this basin as having high infiltration and inflow.

South Corvallis Trunk

The South Corvallis trunk begins at the Marys River pump station on the south side of Marys River, east of State Highway 99W, and parallels the highway south to beyond Goodnight Avenue. The main trunk is 13,100 feet in length and varies from 12 to 21 inches in diameter. This is a separated line, conveying only sanitary sewer flows. No major maintenance problems have been reported on this section of the system. However, the recent study identified this basin as having high infiltration and inflow, and the City has recently conducted smoke testing operations in this area.

Garfield Interceptor System

The original Garfield trunk was constructed of noncompression jointed concrete pipe and was aligned primarily in Garfield Avenue from 9th Street west to Kings Boulevard. From 9th Street flows are conveyed to the Corvallis WWRP. As this section of the City evolved, the Highland and North 9th Street trunk lines were linked with the Garfield trunk to form the interceptor system.

Garfield Trunk

The main Garfield trunk was built as a separated system in 1956 and is 30 to 36 inches in diameter with secondary trunks near 17th Street and at Kings Boulevard and north of Circle Boulevard. The length of pipe in this section of the system is 37,950 feet. With the exception of the original Garfield trunk, the remainder is constructed of rubber gasketed concrete pipe. The trunk is structurally sound, but flow monitoring in this basin has indicated higher than acceptable amounts of rainfall derived infiltration or inflow, or both. Smoke testing was conducted in October 1994 on approximately 16 percent of the Garfield basin. The smoke testing identified a significant number of system defects and possible inflow or infiltration sources. A rehabilitation plan for a portion of the Garfield basin was developed based on the results of the smoke testing.

Highland Trunk

The Highland trunk, built in the late 1950's, is 11,450 feet long and consists of 12- to 18-inch-diameter rubber-gasketed concrete pipe, which is structurally sound and hydraulically capable of handling sanitary sewer flows from its tributary area. This is a fully separated system that is considered to be relatively free from infiltration and inflow.

North 9th Street Trunk

The North 9th Street trunk, built in phases between 1960 to 1969, extends from Garfield to Elks Drive and ranges in size from 10 to 21 inches in diameter. This line consists of 17,600 feet of rubber-gasketed concrete pipe and is considered to be structurally sound.

Beca Trunk

The Beca trunk, built in 1951 and being 13,100 feet long, was constructed of noncompression jointed pipe. Because of this, there were significant infiltration problems. These problems were compounded by many roof drains that were connected to service laterals and

contributed substantial amounts of stormwater inflow to the system. Surcharging of this trunk line resulted in the periodic discharge of raw sewage combined with stormwater infiltration and inflow into Dixon Creek at 16th Street. In 1981, a rehabilitation plan for the Beca basin was developed. A program was conducted to chemically grout all of the leaking and questionable joints in the system, and to eliminate all storm flows by reconnecting roof drains and catch basins to a separate storm drain. The Beca trunk is now considered to be a separated basin. However, because of high flow conditions and the continued potential for basement flooding caused by surcharged pipes, the City recently constructed a relief sewer to divert rainfall-derived infiltration and inflow flows in excess of the capacity of the existing Beca sewer line to the Fillmore basin. This new line begins at Beca Avenue at NW 16th Street and ends at NW 16th Street and Taylor Avenue.

Crescent Valley Interceptor System

The Crescent Valley interceptor, constructed in 1979, consists of 10,300 feet of 36-inch-diameter concrete pipe from the treatment plant north along the Southern Pacific Railroad tracks to Stewart Slough and then west and north along the slough to north of Conifer Boulevard. It is one of the newer additions to the sanitary sewer system and is therefore in good condition.

Pump Station Descriptions

The City's three major pump stations are Brooklane, Marys River, and the Corvallis WWRP Influent Pump Station. These pump stations (shown on Figure 3-1) are integral to the trunk system and were part of the collection system model along with the lesser Herbert lift station which serves the southern portion of the South Corvallis basin. Two additional stations, the 26th Street and Crescent Valley, were included in the pump station assessment documented in Section 4. The three major stations are described in the following text. The Brooklane and Marys River pump stations were constructed in 1962 to 1963 and are of similar design: 15-foot-diameter circular concrete caissons with integral wet pits and dry pits. These stations have adequate capacity under most circumstances. Occasionally, these stations and the section of pipe connecting them have limited capacity to convey high flows as a result of severe rainfall events, resulting in short-term bypasses at each station to the Marys River. The City is committed to correcting these capacity problems.

Brooklane

The Brooklane pump station, located north of Highway 34 at Brooklane Drive, provides capacity for the Brooklane trunk system, and discharges its flow to the Marys River interceptor. It is equipped with two non-clog variable speed centrifugal pumps. The station has a capacity of 3,200 gpm with one pump in operation.

Marys River

The Marys River pump station handles flows from the South Corvallis trunk and discharges across the Marys River into the Marys River interceptor. It has two non-clog variable speed centrifugal pumps and a capacity of 2,700 gpm with one pump in operation.

WWRP Influent Pump Station

All flow from the 1st Street interceptor enters through the regulator gate (also known as the storm gate or I-Gate) from the south, and under normal dry weather flow conditions, passes through the regulator gate opening and flows via a 42-inch-diameter line to the plant influent pump station wet well. A wet well level of 192.0 feet is normally maintained.

The intended design capacity of the influent pump station is 28.0 mgd; however, the pump station output is set at 25 mgd because of suction condition limitations. When flow to the plant reaches 25 mgd and the wet well level continues to rise above the 193.0 foot elevation mark, the regulator gate begins to close to reduce flow to the plant. This limits sewer flows to the reclamation plant and forces combined sewer flows out the overflow points along the 1st Street interceptor rather than entering the interceptor. Separated flows from the southern basins are conveyed in the 1st Street interceptor. Flows from the northern separated sewer basins (Garfield, Beca, and Crescent Valley), however, are not affected by the regulator gate, and continue to flow unabated to the plant. During dry weather, the regulator gate remains open, and all flows can reach the reclamation plant. Operation of the regulator gate has a substantial effect on the quantity, and to a lesser extent, the quality of overflows to the Willamette River.

Flow into the wet well is also regulated by a sluice gate. The sluice gate can be set to operate automatically or manually during periods when flow exceeds the plant pumping capacity. The sluice gate can be manually regulated to limit the flow into the wet well to a maximum flow of 25 mgd.

Wastewater Reclamation Plant Description

Before construction of the Corvallis WWRP in 1952, the City of Corvallis discharged its raw sewage directly into the Willamette River as did many other communities in the valley. The original wastewater reclamation plant provided only primary treatment and chlorine disinfection before the effluent was discharged to the river. The original plant consisted of an influent pumping station, an aerated grit chamber, comminutor, sedimentation tank, two sludge digesters, and control building.

In 1965, the reclamation plant was expanded to provide secondary treatment in the form of fixed film biological treatment (trickling filters) and secondary clarification, and disinfection, which improved the quality of the plant's effluent. This expansion consisted of two 160-foot-diameter trickling filters, a secondary clarification and chlorine contact tank, and a third sludge digester.

In 1973 stricter discharge requirements set by the Oregon Department of Environmental Quality (DEQ) along with increasing population and sewage flows led the City to develop a long-range plan for the treatment of the City's wastewater. This plan called for expansion and improvement of the plant in three separate phases of 10 years each, as required by actual growth within the City. Phase 1 construction, begun in 1976 and completed in June 1978, consisted of an influent pump station, new grit chamber, comminutor, primary sedimentation tanks, aeration tanks, secondary clarifiers, and an operation and maintenance building. By continuing to use all of the existing facilities, the City substantially increased the capacity of the new plant and the quality of its effluent at a lower cost than if a completely new plant had been constructed.

The 1995 capacity analysis by CH2M HILL has determined that the existing treatment plant has a peak influent pumping hydraulic limitation of 25 mgd when two large pumps are operated; this is short of the intended WWRP design capacity of 28.0 mgd. The plant has a historical record of providing excellent treatment over a wide range of influent flow, from a minimum of 5 mgd to peak flows of 25 mgd. Dry season influent flows over the past 3 years have averaged 6.5 mgd with the highest maximum daily flow reaching 23.2 mgd. During the wet season, flows to the treatment plant have averaged 11.0 mgd with a high daily flow of 24.6 mgd. These flow rates do not include combined sewer overflows from the 1st Street interceptor. The per capita carbonaceous 5-day biochemical oxygen demand ($CBOD_5$) value of 0.21 pound per day is consistent with typical averages for domestic wastewater. Dry season influent $CBOD_5$ at the treatment plant has averaged 9,420 pounds per day with a maximum day high of 17,570 pounds per day. Wet season $CBOD_5$ over the past 3 years has averaged 10,847 pounds per day, with the maximum value reaching 24,570 pounds per day.

Historical total suspended solids (TSS) per capita values are 0.33 pound per day during the dry season and 0.46 pound per day during the wet season. These values are significantly above the values of 0.20 to 0.26 pound per day per capita typical for domestic wastewater. The reason is likely two-fold. First, waste activated sludge, which is returned to the headworks of the plant, is included in the influent sample. Second, the combined sewer system would contribute to higher than average values, as has been experienced with other combined sewerage systems. Dry season TSS concentrations have averaged 15,550 pounds per day from 1992 through 1994, with the maximum value reaching 36,460 pounds per day. The wet season average is 21,940 pounds per day, with a maximum value of 59,700 pounds per day.

The analysis also determined that for a population of approximately 50,000 persons, there is sufficient existing capacity to treat up to 5 mgd of additional wet weather flow during maximum week flow conditions during both dry and wet seasons. This is based on meeting 15 and 40 mg/L dry and wet season weekly $CBOD_5$ effluent limits, respectively. A revision to the existing maximum week mass wasteload limits, especially for the dry season, would be required. In addition, it is projected that 2 and 5 mgd of peak day wet weather flow could be diverted to the existing treatment plant during the wet and dry seasons, respectively, if the plant could be operated at its maximum capacity of 28 mgd.

In order to reliably treat peak flows of 28 mgd through the existing secondary plant, the following improvements in the hydraulic and treatment capacity of the plant would be required:

- Correction of influent pumping limitations
- Additional influent screening capacity
- Modifications to the flocculation walls in the existing secondary clarifiers
- Improved dissolution of chlorine
- Selected channel improvements to contain the peak flows
- Yardpiping, electrical, and other support system modifications as necessary

Flow Monitoring

An intensive flow monitoring study was conducted for the City between February 18 and March 13, 1995, by CH2M HILL and ADS Environmental Services at a total of 25 sites

shown on Figure 3-3. These temporary monitors were located strategically within each of the basins with separated sewers. The purpose of the flow and precipitation monitoring program was twofold: (1) the collection system response to rainfall events was needed to perform a rainfall-derived infiltration and inflow (RDII) analysis and (2) typical flows during wet and dry weather periods were required to calibrate the computer model. The model estimates the responses of the existing system and improvement alternatives to the SFO storms. Two technical memoranda (TM), TM 2.4 Infiltration and Inflow Analysis and TM 1.1 Hydraulic Modeling, more fully explain the work performed (see Appendixes A and B). Table 3-2 lists the flow monitors for the separated sewer areas.

TABLE 3-2
Monitor Locations—Separated System

Monitor Designation	Basin	City Manhole No.	Location Description
ADSB1	Beca	4480046	Beca between 99W and 9th St.
ADSB2	Beca	4477060	16th and Beca
ADSBR2	Brooklane	3474016	West of Brooklane Dr. at Stratton Way
ADSBR2A	Brooklane	3674004	30th St. North of Western in front of Veterinary Hospital
ADSBR3	Brooklane	3471010	35th St., south side of Adams School
ADSBR4	Brooklane	3471005	Along bike path west of 35th St., east of Research Way, north of Country Club
ADSBRS1	Brooklane/ South Corvallis	3680016	Just off bike path east of "B" Ave. and 2nd St.
ADSCV2	Crescent Valley	5086012	Just off Sherwood Way, South of Newcastle Place
ADSG2	Garfield	4677088	17th and Garfield
ADSG3	Garfield	4677084	1626 Kings Blvd.
ADSG4	Garfield	4874099	27th and Elmwood
ADSG4A	Garfield	4674006	Circle at 1810 Juniper
ADSG5	Garfield	5071069	3452 Walnut Blvd.
ADSG6	Garfield	5071067	Walnut Blvd. In front of Hoover School
ADSG7	Garfield	5071063	Walnut Blvd. Between Hummingbird and Aspen
ADSG8	Garfield	4868049	2498 NW Maser Dr.
ADSH1	Highland	4483001	3rd manhole upstream from WWTP
ADSH2	Highland	4880070	Highland Dr. in front of Osbourne Aquatic Center
ADSH3	Highland	4877030	2215 13th Ave.
ADSN1	North Ninth	4680028	9th St. in front of Skipper's
ADSN2	North Ninth	4883032	9th St. just north of Burger King
ADSN2A	North Ninth	5080024	Trailer park cul de sac off Satinwood
ADSS2	South Corvallis	3477042	North side Bertsch Moving/Storage, east of 99W
ADSS3	South Corvallis	3277027	99W in front of Lincoln School
ADSS4	South Corvallis	2877007	190 Goodnight Ave.

In addition, the City of Corvallis maintained 10 permanent monitors:

- **Fillmore Monitor.** Located in the conduit just upstream of the Fillmore CSO diversion structure.
- **Tyler Monitor.** Located in the 1st Street interceptor upstream of the Polk CSO.
- **Jefferson Monitor.** Located in the conduit just upstream of the junction with the 1st Street interceptor.
- **Western Monitor.** Located in the conduit just upstream of the Western CSO diversion structure.
- **Garfield Street Monitor.** Located in the conduit at 13th Street and Garfield Avenue.
- **Garfield Meter Monitor.** Located in the Garfield interceptor.
- **Crescent Valley Monitor.** Located in the conduit of the Crescent Valley interceptor upstream from the WWRP.
- **Marys River Pump Station Monitor.** Located in the conduit downstream from the Marys River pump station.
- **Brooklane Pump Station Monitor.** Located in the conduit downstream from the Brooklane pump station.
- **Wastewater Reclamation Plant Monitor.** Located downstream from the storm gate flow regulator.

The City of Corvallis also maintained four rain gages:

- **Hoover Gage.** Located at the Hoover Pump Station in North Corvallis.
- **South Hills Gage.** Located at the South Hills Water Lift Station in Southwest Corvallis.
- **Taylor Gage.** Located at the Taylor Water Treatment Plant in Southeast Corvallis.
- **WWRP Gage.** Located at the Corvallis WWRP.

The locations of permanent and temporary flow monitors and rain gages are shown on Figure 3-3.

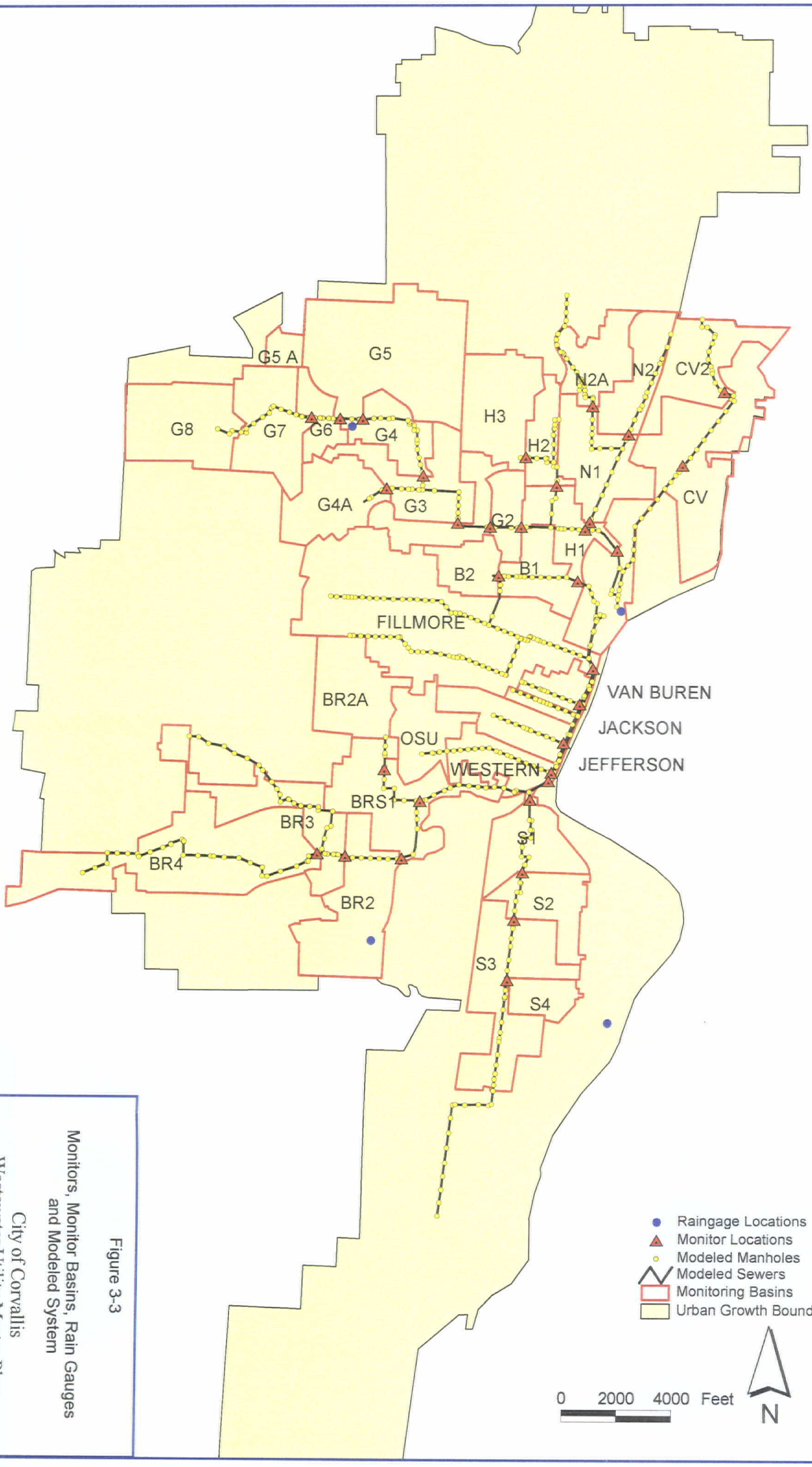


Figure 3-3

Monitors, Monitor Basins, Rain Gauges and Modeled System

City of Corvallis
Wastewater Utility Master Plan

Rainfall-Derived Infiltration and Inflow (RDII) Analysis

RDII is the flow entering the sewer system as a direct result of rain. RDII increases total flow volume and peak flow, and consists of two components: infiltration, which slowly percolates into the collection system; and inflow, which reaches a peak shortly after rainfall intensity is greatest and falls off rapidly when rain subsides. Since the flow monitors directly measure total flow, RDII may be estimated by subtracting the average base flow (ABF), comprising sanitary flow and base groundwater infiltration, from the total flow.

The purpose of the RDII analysis was to identify sewer basins that are large contributors of RDII, to quantify these wet weather flows, categorize the type of RDII as either inflow, infiltration, or combination of both, and to rank the basins for additional investigation and type of investigation. Based upon the intensive flow monitoring during February 1995, regression equations were developed to predict flow based on rainfall and selected dry weather flow patterns. The flow estimates were used in modeling efforts to generate design storm hydrographs in the separated collection system.

Wet Season Average Base Flow (ABF)

The wet weather ABF at a flow monitoring site was developed by selecting 7 days of flow data from a dry period (no precipitation) during the winter study period. An ABF hydrograph, composed of sanitary flow and base groundwater infiltration, was developed for each location. The composite 24-hour ABF hydrograph was created by determining the minimum flow for each hour from flow monitor data recorded over the dry days selected. The average base flow was used in the calculation of RDII. This information was used to develop the rate of RDII to apply to future development.

Flow Estimates

Each flow monitor measures flow from all upstream sources, and in some instances is affected by backwater conditions from downstream pipes. To isolate RDII originating from a contributing area between two monitors, flows from upstream basins were subtracted from the flows measured at the downstream monitor. As is typical during any flow monitoring program, some of the monitors had occasional unreliable or missing data. A method was employed to fill in these gaps and to replace unreliable data. This correction allowed the estimation of RDII for all of the basins for the four storms. One shortcoming of the monitoring program that could not be overcome was the inability to measure CSOs both from the pump stations and from the 1st Street interceptor overflows. This was due to the lack of appropriate locations for overflow monitoring and the desire to make efficient use of the monitors available.

Estimates of RDII are based on a regression relationship between rainfall and flow. RDII is flow resulting from rainfall that has entered the collection system over the past hours, days, and weeks. For this analysis, it was assumed that a multiple linear relationship existed between RDII and past rainfall. Wet weather flow, ABF, and rainfall recorded at the nearest representative rain gage were extracted from the database and imported into an Excel spreadsheet. Rainfall during the 15 days (360 hours) before each hourly flow measurement was summed for the following nine periods: 1 hour, 2 to 3 hours, 4 to 6 hours, 7 to 12 hours,

12 hours to 1 day, 1 to 2 days, 4 to 7 days, and 7 to 15 days. Excel was used to perform multiple linear regression on the correlation between the rainfall sums and the measured flow to determine the regression coefficients for each rainfall sum.

The regression equation was:

$RDII = C1*Rain_{1hr} + C2*Rain_{2hr} + ... + Cn*Rain_{nhr}$

with:

Total Flow = ABF + RDII

Figure 3-4 is an example of the results of this analysis. The analysis was done for each monitoring station using software that allows interaction between the Access database and Excel. Once the regression equations are developed and visually checked, flow may be estimated for any period by applying the equation to precipitation data collected at other periods or for specific design storm events. The regression equations can be used to generate flow when monitor data are missing or unreliable provided rainfall data is available for the period of interest. Figure 3-5 is an example of applying the regression to generate flows at a monitor location for the 5 year rainfall event. These regression equations have been used in CSO modeling efforts to generate design flow input to the collection system. The regression equations were also used to identify flow characteristics within a basin as described in the next section.

Basin Categories

To categorize the response of each monitored basin to wet weather events, the following calculations were made for each of the four storms:

- Total flow volume (base flow and RDII)
- Total RDII flow volume
- The ratio of peak instantaneous flow to ABF
- The percentage of RDII resulting from defined periods of prior rainfall

Sewersheds monitored by a specific monitor (or series of monitors) were placed into categories to help direct source detection and remediation efforts. A sewershed's pipes conveying a high percentage of flow resulting from precipitation that occurred days or weeks before the flow measurement indicates the contributing basin is receiving water from infiltration sources. These sources include defective joints in the sewer, broken and cracked pipe, improper connections, and defects in manhole walls. Table 3-3 shows the percentage of the total peak RDII flow that results from rainfall during each of three time periods before the peak flow: 1 through 6 hours, 7 through 48 hours, and 49 through 360 hours. A monitor that has a higher percentage of the peak flow derived from more recent precipitation (within hours instead of days) indicates the basin is receiving water from inflow sources. These sources include catch basins, roof drains, foundation drains, open manhole covers, and other direct or illicit connections. Table 3-4 categorizes each of the monitor basins with respect to its flow response to rainfall events. The objective in defining these categories is to focus potential RDII reduction activities on the basins where the most significant contributions to peak RDII rates exist (Category 1 being the highest priority). As a result, basins

Figure 3-4
Create Regression
Flow Monitor: ADSG2 Rain Gage: HR

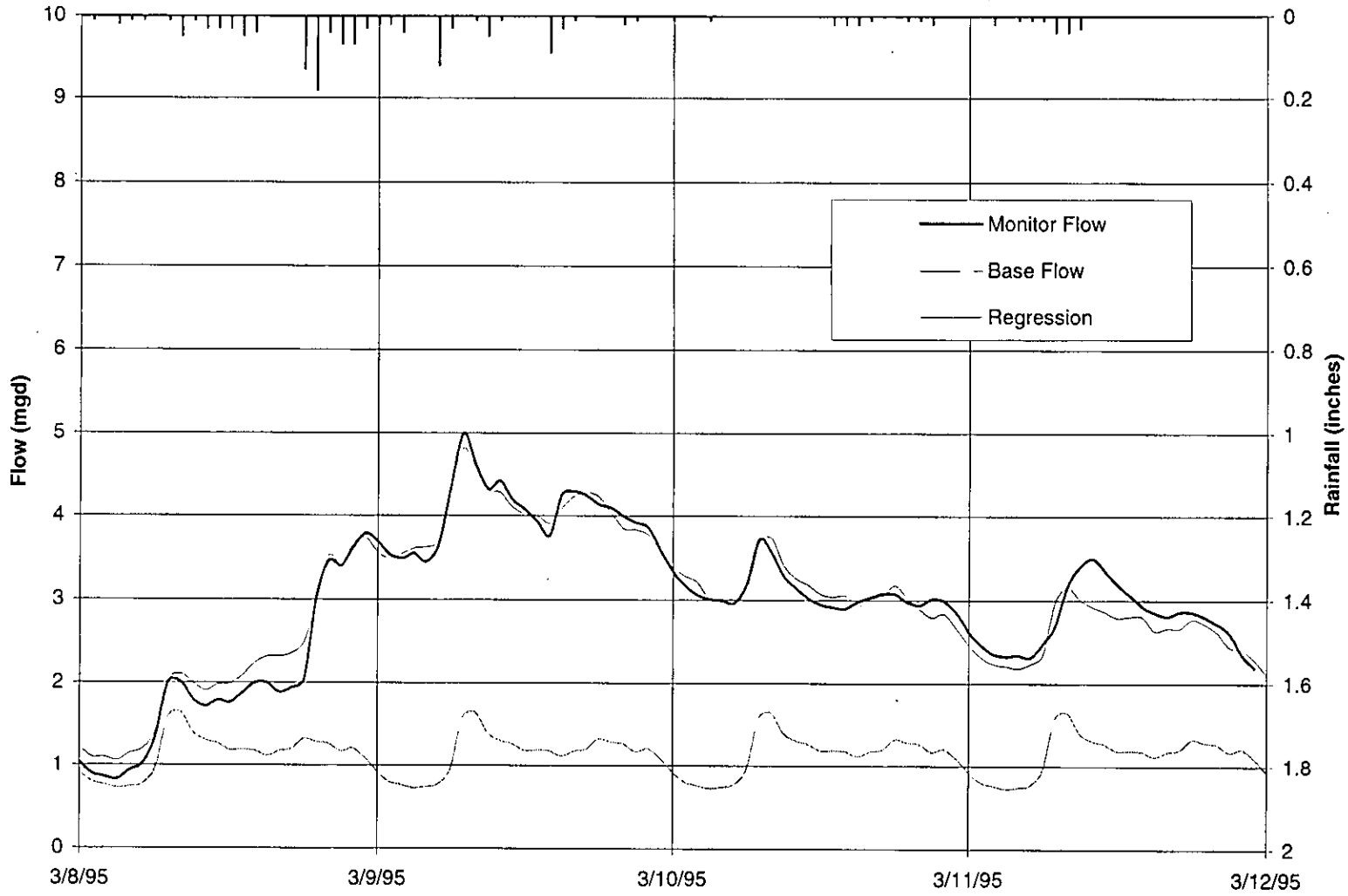


Figure 3-5
Apply Regression
Flow Monitor: ADSG2 Rain Gage: 5-year

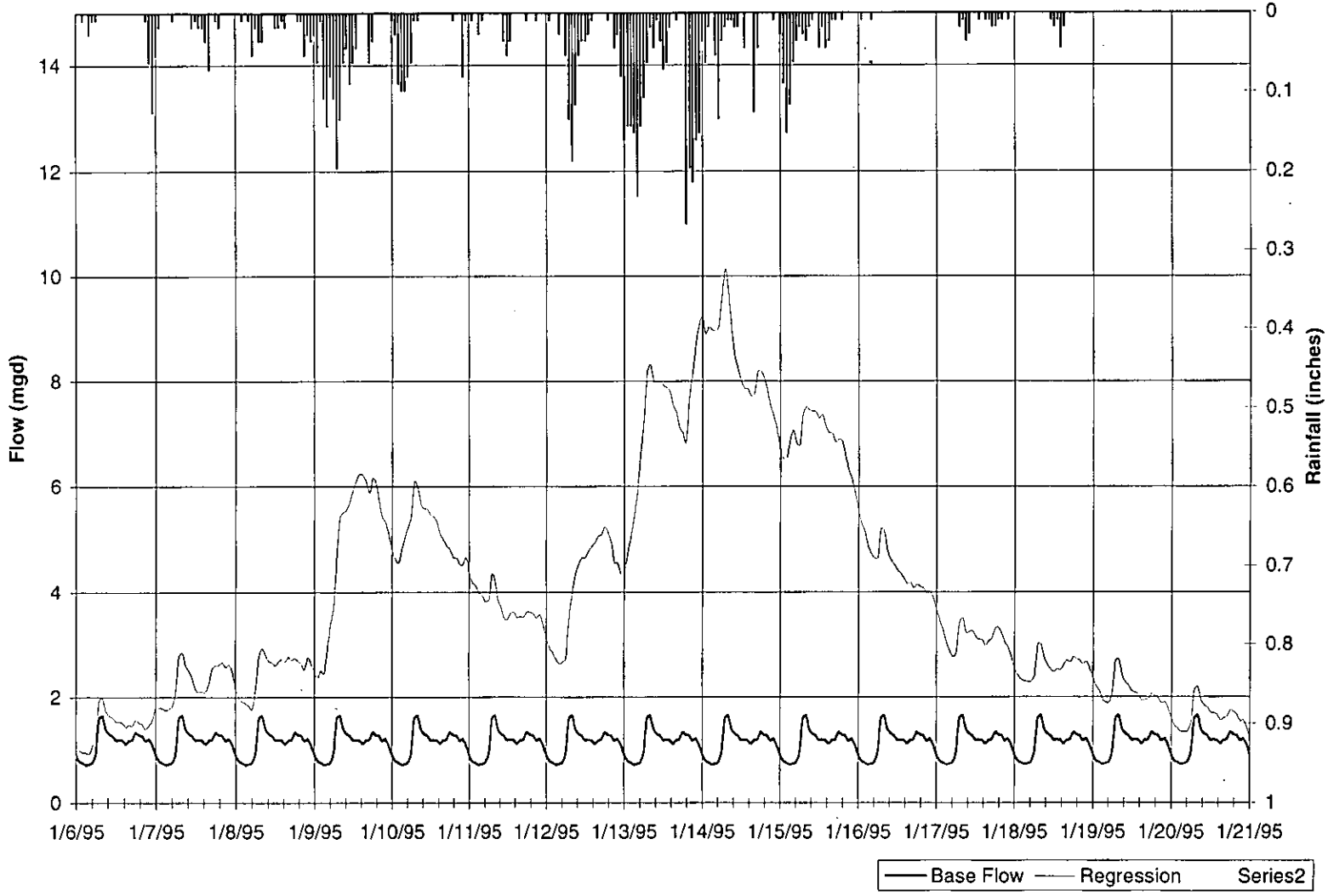


TABLE 3-3
Contribution to Peak Flows

Station	Storm 1, 1/6/95 - 1/19/95				Storm 2, 1/26/95 - 2/2/95				Storm 3, 2/15/95 - 2/20/95				Storm 4, 3/8/95 - 3/16/95			
	Peak RDII (mgd)	Time Period (hours)			Peak RDII (mgd)	Time Period (hours)			Peak RDII (mgd)	Time Period (hours)			Peak RDII (mgd)	Time Period (hours)		
		1 thru 6	7 thru 48	49 thru 360		1 thru 6	7 thru 48	49 thru 360		1 thru 6	7 thru 48	49 thru 360		1 thru 6	7 thru 48	49 thru 360
Brooklane PS	6.69	54%	38%	8%	4.56	58%	27%	15%	5.02	55%	39%	7%	2.51	45%	55%	0%
Crescent Valley	1.75	54%	35%	10%	1.03	51%	33%	16%	1.22	43%	48%	9%	0.86	55%	45%	0%
Fillmore	31.70	70%	27%	4%	23.78	78%	18%	4%	31.44	65%	32%	2%	18.14	85%	13%	2%
Garfield Meter	19.67	53%	35%	12%	14.31	49%	36%	15%	16.69	52%	39%	9%	7.56	74%	20%	6%
Garfield Street	13.88	56%	37%	7%	9.68	60%	31%	10%	12.83	53%	42%	5%	5.62	75%	22%	4%
Jefferson	1.05	77%	23%	0%	0.86	88%	12%	0%	1.10	79%	21%	0%	0.65	93%	7%	0%
Mary's River PS	7.89	46%	46%	8%	5.09	55%	30%	16%	5.17	52%	41%	7%	2.67	33%	67%	0%
Tyler	16.83	49%	46%	5%	11.22	55%	37%	7%	15.71	46%	51%	3%	6.91	29%	71%	0%
Western	2.44	87%	13%	0%	1.89	97%	3%	1%	1.53	93%	7%	0%	1.22	49%	4%	0%
ADSB1	1.58	54%	41%	4%	1.00	71%	23%	6%	1.53	54%	43%	3%	0.72	40%	60%	0%
ADSB2	0.96	44%	45%	11%	0.67	48%	38%	14%	0.84	47%	45%	7%	0.38	33%	67%	0%
ADSBR2	2.32	29%	61%	9%	1.62	11%	67%	22%	1.61	23%	69%	8%	0.85	14%	86%	0%
ADSBR2A	0.91	51%	44%	5%	0.64	63%	28%	8%	0.69	52%	45%	4%	0.41	47%	53%	0%
ADSBR3	1.36	32%	60%	8%	0.87	11%	68%	21%	0.88	29%	63%	8%	0.52	15%	85%	0%
ADSBR4	0.94	8%	87%	5%	0.65	10%	68%	21%	0.59	22%	69%	9%	0.30	11%	89%	0%
ADSBRS1	11.79	44%	46%	10%	7.97	46%	35%	18%	8.01	48%	44%	9%	3.81	28%	72%	0%
ADSCV2	1.14	52%	44%	4%	0.80	57%	38%	5%	1.00	52%	45%	3%	0.44	34%	66%	0%
ADSG2	4.81	42%	51%	7%	3.26	47%	44%	10%	4.15	43%	52%	5%	1.87	26%	74%	0%
ADSG3	3.84	42%	50%	8%	2.66	46%	43%	11%	3.28	44%	50%	6%	1.46	25%	75%	0%
ADSG4	2.07	42%	50%	7%	1.41	47%	43%	10%	1.80	43%	51%	5%	0.80	27%	73%	0%
ADSG4A	1.05	44%	48%	9%	0.72	48%	41%	11%	0.86	46%	47%	6%	0.38	30%	70%	0%
ADSG5	1.60	40%	52%	8%	1.08	45%	44%	10%	1.40	41%	54%	5%	0.63	25%	75%	0%
ADSG6	1.12	14%	82%	5%	0.79	13%	72%	14%	0.96	39%	54%	6%	0.44	24%	76%	0%
ADSG7	0.75	11%	85%	4%	0.52	12%	77%	12%	0.66	3%	92%	5%	0.31	17%	83%	0%
ADSG8	0.48	38%	56%	6%	0.32	13%	76%	11%	0.45	10%	86%	4%	0.21	23%	77%	0%
ADSH1	9.60	49%	47%	5%	6.62	54%	40%	6%	8.60	49%	48%	3%	3.89	31%	69%	0%
ADSH2	0.78	51%	43%	6%	0.56	57%	36%	7%	0.78	49%	48%	4%	0.35	29%	71%	0%
ADSH3	0.53	60%	21%	19%	0.42	60%	19%	22%	0.47	61%	27%	13%	0.22	73%	15%	13%
ADSN1	2.28	53%	41%	6%	1.61	58%	34%	8%	2.10	51%	45%	4%	0.92	32%	68%	0%
ADSN2	0.57	52%	42%	6%	0.40	56%	36%	8%	0.53	52%	44%	4%	0.24	35%	65%	0%
ADSN2A	0.44	45%	48%	7%	0.30	49%	42%	9%	0.38	46%	49%	5%	0.17	28%	72%	0%
ADSS2	6.20	43%	52%	5%	2.48	82%	18%	0%	2.04	41%	57%	3%	1.17	24%	76%	0%
ADSS3	3.47	35%	61%	5%	1.19	35%	61%	4%	1.15	19%	79%	2%	0.69	24%	76%	0%
ADSS4	1.02	42%	55%	4%	0.40	93%	7%	0%	0.33	34%	64%	2%	0.20	25%	75%	0%

with small contributions to RDII are not considered in further evaluations of potential cost effective removal. The categories selected were based on a combination of:

- Timing of the RDII response to rainfall (Table 3-3)
- Return percentage (calculated as the percentage of rainfall that enters the system as measured at the monitor locations)
- Peak to average flow ratios
- Actual peak flow rates measured

The following definitions were used to categorize each basin:

- **Category 1:** Return Greater than 20 percent (High Inflow and Infiltration Response). RDII volume is greater than 20 percent of precipitation; 50 percent of peak flow results from precipitation occuring more than 6 hours prior to peak. These basins "return" a high percentage of the rainfall falling on the basin and show a high inflow response. These basins were selected because of their high RDII response.
- **Category 2:** Return Greater than 10 percent (Inflow Response). RDII volume is greater than 10 percent of precipitation volume; 50 percent of peak flow results from precipitation occurring within 6 hours prior to peak. These basins have peak flows with more than 50 percent of the RDII peak flow resulting from precipitation within the previous 6 hours and a lower return percentage for RDII. These basins were selected for additional analysis because of their high inflow potential.
- **Category 3:** Return Less than 10 percent (Inflow Response). RDII volume is less than 10 percent of precipitation volume; 50 percent of peak flow results from precipitation occuring less than 6 hours prior to peak. These basins have a relatively low "return" percentage for RDII with more than 50 percent of the RDII peak flow resulting from precipitation within the previous 6 hours. Basins fitting this profile have less priority for continued investigation.
- **Category 4:** Return Less than 20 percent (Infiltration Response). RDII volume is less than 20 percent of precipitation volume; 50 percent of peak flow results from precipitation occuring more than 6 hours prior to peak. These basins show low inflow response and exhibit a range of "return" values. Infiltration is a larger contributor than inflow, and therefore these basins are not critical contributors to the peak flow condition. Basins fitting this profile are not ranked as a high priority for continued investigation primarily because of their low inflow potential.
- **Category 5:** Total Wet Weather Peak Flows Less than 1 mgd. Total flow is less than 1.0 mgd. These basins are small contributors to peak flow and are not critical to reduction of the peak flow. There is less benefit in continuing to study these basins.
- **Category 6:** Monitored Special Interest Basins. This category consists of basins that have had indications of RDII problems in the past or require further study to determine whether RDII problems exist. Inconclusive monitoring and data analysis did not allow for their placement in a specific category.

Table 3-4
Percentage Performance for Metered Basins

Basin Designation	Flow Monitor	Total Upstream RDII Volume (MG) Storm 1	Storm 2	Storm 3	Storm 4	Upstream Monitors	Local Subbasin RDII Volume (MG) Storm 1	Storm 2	Storm 3	Storm 4	Peak RDII Flow (mgd) Storm 1	Storm 2	Storm 3	Storm 4	ABF (mgd)	Rain Gage	Precipitation Volume (MG) Storm 1	Storm 2	Storm 3	Storm 4	Area (acres)
CATEGORY 1: Return > 20 % - High Inflow and Infiltration Response																					
(RDII volume greater than 20% of precipitation, 50% of peak flow results from precipitation occurring more than 6 hours prior to peak)																					
South Corvallis	ADSS2	30.08	7.81	3.72	4.11	ADSS3	13.37	3.44	1.62	1.82	6.20	2.48	2.04	1.17	0.82	TR	44.9	13.0	6.2	6.4	152
																				Subtotal:	152
CATEGORY 2: Return > 10% – Inflow Response																					
(RDII volume less than 10% of precipitation volume, 50% of peak flow results from precipitation occurring within 6 hours of peak)																					
Highland	ADSH1	47.28	17.22	16.66	14.90	ADSN1, ADSH2, ADSG2	6.20	2.69	2.61	2.04	9.60	6.62	8.60	3.89	2.18	HR	49.2	20.7	19.6	15.9	223
North Ninth	ADSN1	11.20	3.99	3.85	3.51	ADSN2, ADSN2A	5.23	1.73	1.77	1.41	2.28	1.61	2.10	0.92	0.54	HR	49.6	20.9	19.8	16.0	225
Crescent Valley	ADSCV2	5.43	2.01	1.94	1.72		5.43	2.01	1.94	1.72	1.14	0.80	1.00	0.44	0.36	HR	48.7	20.5	19.4	15.7	221
																				Subtotal:	669
CATEGORY 3: Return < 10% - Inflow Response																					
(RDII volume less than 10% of precipitation volume, 50% of peak flow results from precipitation occurring less than 6 hours prior to peak)																					
Beca	ADSB1	7.44	2.73	2.64	2.34	ADSB2	1.92	0.89	0.86	0.64	1.58	1.00	1.53	0.72	0.30	HR	28.7	12.1	11.4	9.2	130
																				Subtotal:	130
CATEGORY 4: Return < 20% - Infiltration Response																					
(RDII volume less than 20% of precipitation volume, 50% of peak flow results from precipitation occurring more than 6 hours prior to peak)																					
Brooklane	ADSBR2	13.58	4.59	3.77	4.01	ADSBR3	5.85	1.94	1.58	1.72	2.32	1.62	1.61	0.85	0.60	SR	76.5	33.2	26.1	24.0	320
Brooklane	ADSBR3	7.73	2.65	2.19	2.29	ADSBR4	2.43	0.88	0.73	0.73	1.36	0.87	0.88	0.52	0.57	SR	60.5	26.2	20.6	19.0	253
Garfield	ADSG2	25.99	9.14	8.85	8.13	ADSG3	4.51	1.71	1.66	1.43	4.81	3.26	4.15	1.87	1.14	HR	34.6	14.6	13.8	11.2	157
Garfield	ADSG3	21.49	7.43	7.19	6.69	ADSG4, ADSG4A	4.43	1.50	1.45	1.37	3.84	2.66	3.28	1.46	1.03	HR	42.3	17.8	16.9	13.7	192
Garfield	ADSG4	11.26	3.94	3.81	3.52	ADSG5	2.45	0.86	0.83	0.77	2.07	1.41	1.80	0.80	0.56	HR	27.8	11.7	11.1	9.0	126
Garfield	ADSG5	8.81	3.08	2.98	2.75	ADSG6	2.32	0.85	0.82	0.73	1.60	1.08	1.40	0.63	0.47	HR	35.3	14.9	14.1	11.4	160
Garfield	ADSG6	6.49	2.23	2.16	2.02	ADSG7	2.37	0.78	0.76	0.73	1.12	0.79	0.96	0.44	0.35	HR	15.9	6.7	6.3	5.1	72
South Corvallis	ADSS3	16.72	4.37	2.10	2.29	ADSS4	12.31	3.20	1.53	1.68	3.47	1.19	1.15	0.69	0.67	TR	69.8	20.3	9.7	9.9	236
South Corvallis	ADSS4	4.41	1.17	0.56	0.61		4.41	1.17	0.56	0.61	1.02	0.40	0.33	0.20	0.13	TR	23.7	6.9	3.3	3.3	80
																				Subtotal:	1596
CATEGORY 5: Total Wet Weather Peak Flows Less than 1 mgd																					
(Total flow less than 1.0 mgd)																					
Beca	ADSB2	5.52	1.85	1.78	1.70		5.52	1.85	1.78	1.70	0.96	0.67	0.84	0.38	0.11	HR	21.8	9.2	8.7	7.0	99
Garfield	ADSG4A	5.80	1.99	1.92	1.80		5.80	1.99	1.92	1.80	1.05	0.72	0.86	0.38	0.18	HR	33.5	14.1	13.4	10.8	152
Garfield	ADSG7	4.12	1.45	1.41	1.29	ADSG8	1.58	0.54	0.52	0.49	0.75	0.52	0.66	0.31	0.24	HR	43.2	18.2	17.2	13.9	196
Garfield	ADSG8	2.54	0.91	0.88	0.80		2.54	0.91	0.88	0.80	0.48	0.32	0.45	0.21	0.06	HR	3.7	1.6	1.5	1.2	17
Highland	ADSH2	3.89	1.40	1.35	1.22	ADSH3	0.68	0.45	0.44	0.26	0.78	0.56	0.78	0.35	0.21	HR	22.3	9.4	8.9	7.2	101
Highland	ADSH3	3.20	0.95	0.91	0.96		3.20	0.95	0.91	0.96	0.53	0.42	0.47	0.22	0.17	HR	37.9	16.0	15.1	12.2	172
North Ninth	ADSN2	3.61	1.43	1.28	1.36		3.61	1.43	1.28	1.36	0.57	0.40	0.53	0.24	0.09	HR	33.5	14.1	13.4	10.8	152
North Ninth	ADSN2A	2.36	0.83	0.81	0.74		2.36	0.83	0.81	0.74	0.44	0.30	0.38	0.17	0.09	HR	25.1	10.6	10.0	8.1	114
Brooklane	ADSBR2A	4.12	1.51	1.23	1.24		4.12	1.51	1.23	1.24	0.91	0.64	0.69	0.41	0.23	SR	49.5	21.5	16.9	15.5	207
Brooklane	ADSBR4	5.30	1.77	1.46	1.56		5.30	1.77	1.46	1.56	0.94	0.65	0.59	0.30	0.32	SR	120.4	52.3	41.1	37.8	504
																				Subtotal:	1714
CATEGORY 6: MONITORED SPECIAL INTEREST BASINS																					
Brooklane/South Corvallis	ADSBRS1	65.67	21.90	17.92	19.33	ADSBR2A, ADSBR2, ADSS2	17.89	7.99	9.21	9.97	11.79	7.97	8.01	3.81	0.57	SR	98.0	42.5	33.4	30.7	410
																				Subtotal:	410
PERMANENT MONITORS																					
Brooklane	Brooklane PS	32.35	12.00	8.84	8.08	ADSBR2, ADSBR2A	14.65	5.91	3.84	2.83	6.85	6.17	6.15	2.26	1.62	SR	47.8	20.7	16.3	15.0	200
Crescent Valley	Crescent Valley	7.76	2.45	2.37	2.37	ADSCV2	2.33	0.44	0.43	0.65	1.75	1.03	1.22	0.86	0.36	HR	67.0	28.2	26.7	21.6	304
Fillmore (CSO)	Fillmore	106.52	39.35	37.84	33.50		106.52	39.35	37.84	33.50	31.70	23.78	31.44	18.14	5.07	HR	373.5	157.3	149.0	120.5	1694
Garfield	Garfield Street	66.76	18.51	22.20	18.89		66.76	18.51	22.20	18.89	16.11	9.09	15.23	4.20	1.27	HR	253.8	106.9	101.3	81.9	1151
Garfield	Garfield Meter	113.72	35.29	34.91	32.57	Garfield Street	46.96	16.78	12.71	13.69	30.90	19.49	16.69	7.78	1.82	HR	183.7	77.4	73.3	59.3	833
Jefferson (CSO)	Jefferson	2.82	1.16	1.11	0.91		2.82	1.16	1.11	0.91	1.05	0.86	1.10	0.65	0.18	HR	37.5	15.8	15.0	12.1	170
Brooklane/South Corvallis	Mary's River PS	34.70	15.18	11.46	13.87		34.70	15.18	11.46	13.87	3.87	3.41	3.54	2.26	0.29	SR	321.9	139.7	109.7	101.0	1347
Tyler (CSO)	Tyler	36.82	31.16	22.59	25.15		36.82	31.16	22.59	25.15	9.81	12.33	8.47	7.36	1.06	HR	26.2	11.1	10.5	8.5	119
Western (CSO)	Western	12.29	2.48	4.33	6.64		12.29	2.48	4.33	6.64	6.98	2.47	4.50	2.35	0.68	SR	83.6	36.3	28.5	26.2	350
																				Subtotal:	6168

Figure 3-6 shows the categorized monitor basins. The return percentages are a direct indicator of a basin's contribution to total RDII. The greater the percentage, the greater the amount of rainfall in the basin entering the sanitary sewer system. The figure provides a quick overview of one of the key measures used to characterize the basins.

Hydraulic Analysis Model

The Corvallis sewer system is comprised of collector pipes, interceptors, junctions (manholes), pump stations, CSO diversion structures, and CSO outfalls. Inflow received by the sewers includes base sanitary flow, groundwater infiltration, RDII, and direct stormwater inflow. Proper analysis of the Corvallis sewer system required that a comprehensive hydraulic simulation model be developed.

The model was developed using the U.S. Environmental Protection Agency (EPA) Storm Water Management Model with graphical interface by XP Software (XP-SWMM). SWMM was originally developed for the EPA specifically for the analysis of combined sewer systems, and is considered an excellent modeling tool for storm, sanitary and combined sewer collection systems. The graphical interface allows easy data input and viewing of results. Over the course of this study City staff were trained in the use of the XP-SWMM model during five training sessions. Three components (blocks) of SWMM were used for the Corvallis model: RUNOFF, TRANSPORT, and EXTRAN. The RUNOFF block simulates surface runoff from defined subcatchments. TRANSPORT is a simple flow transport module. For the Corvallis application, it was used to generate diurnal base sanitary inflows within the CSO area. The EXTRAN block simulates the routing of flow through the system. EXTRAN is a fully dynamic model that solves the complete dynamic wave equations (St. Venant) for gradually varied flow. It can simulate backwater, surcharging, looped connections, and various hydraulic appurtenances that typically occur in sewer systems. Results of the model are in the form of flow and hydraulic gradients at all nodes over the duration of simulation.

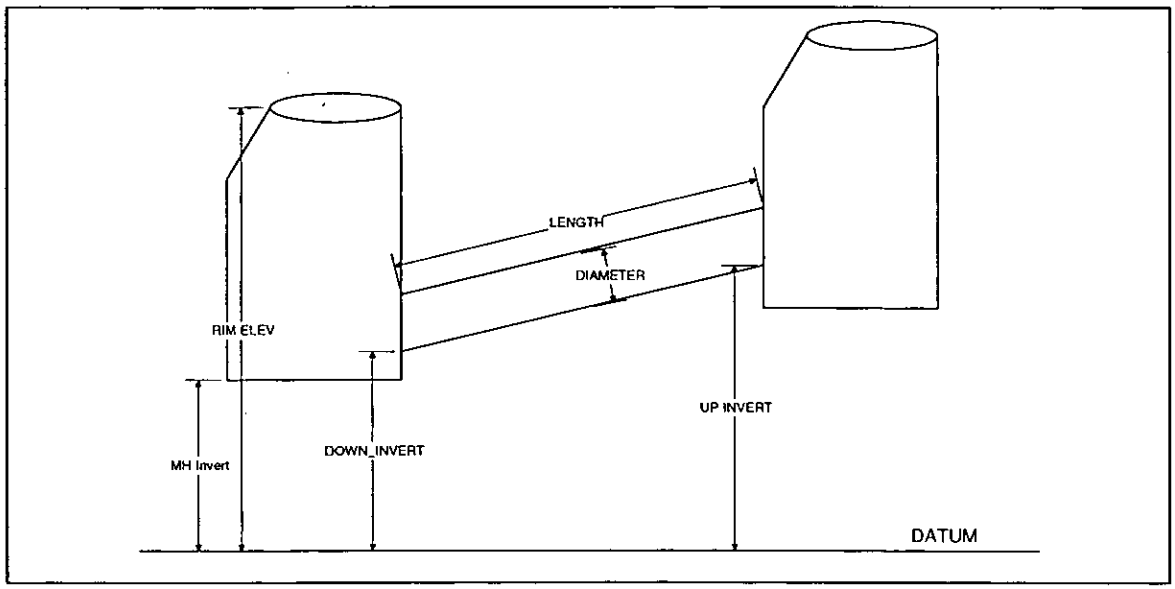
Building upon the previously developed Corvallis CSO model, the hydraulic model represents the major elements of the Corvallis sewer system. It includes all city-owned pipes with diameters greater than 10 inches. Model flow inputs were located at all nodes upstream from a flow monitoring location. Flows at inflow nodes were derived externally from flow monitor data and input directly into the model. The magnitudes of the flows distributed at inflow nodes within a monitor basin are based on pipe length (i.e., flow inputs at the upstream end of long pipes are proportionally greater than flow inputs at the upstream end of shorter pipes). Model outflow boundary nodes include the four CSO locations (Western, Jackson, Van Buren, and Fillmore—the Polk overflow will be eliminated as part of the combined sewer overflow project) and the WWRP and wet weather treatment facility (WWTF) pump stations.

Hydraulic Model Input Data and Design Criteria

Information used to extend the CSO model into the separated areas of the collection system were provided by City staff in the form of ArcView shape files. The following diagram in Figure 3-7 summarizes information contained in these files necessary for modeling the system.

In addition, information on pipe material was provided by the City from which Manning's "n" values were assigned to each pipe segment. Using software developed by CH2M HILL, these data were translated into a SWMM model database.

FIGURE 3-7
Graphic Summary of ArcView Shape File Information



Base Flows

Existing (1995) wet season base flows were obtained for each of the flow monitor basins from data collected at the respective monitor locations. Diurnal flow patterns were developed by selecting seven days of flow data from a dry period (no precipitation). Future base flows were derived by extrapolating the existing flows according to population increase.

Sanitary base flows associated with an ultimate population of 120,000 were used for sewersheds draining to or through the combined sewer area. This is appropriate because CSO remediation alternatives (for example, relief interceptor along 1st Street) were designed for a minimum 50-year life.

Rainfall-Derived Infiltration and Inflow (RDII)

RDII predictive equations were generated by performing multiple linear regression on rainfall versus flow over a specific wet-weather period at each of the 25 monitor locations. Groundwater infiltration is part of the flow data used to derive predictive equations and is therefore implicitly a part of the RDII equations. The equation for each area was used to simulate an inflow hydrograph to the model representing existing RDII. RDII under future conditions was estimated by applying an inflow hydrograph corresponding to 4,000 gallons per acre per day (gpad) for the future development acreages. This value was arrived at by analysis of flow monitor data. At least three basins, Lower Oak Creek, Country Club, and Squaw Creek were observed to have I/I rates at or less than 4,000 gpad. These basins have

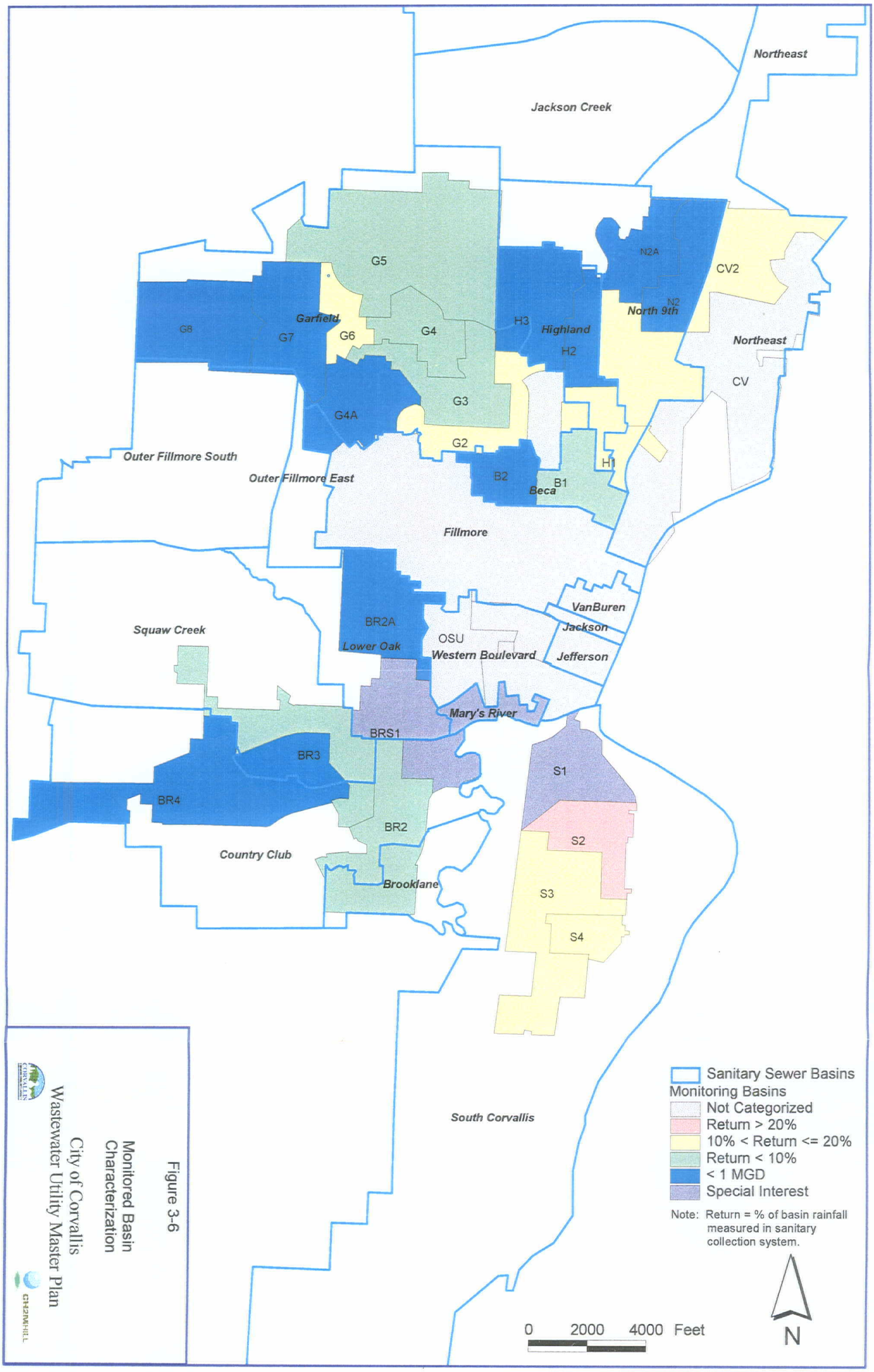


Figure 3-6

Monitored Basin Characterization

City of Corvallis
Wastewater Utility Master Plan

been described generally as being representative of areas constructed using current techniques and can be considered representative of future construction. This rate is also consistent with other Willamette Valley communities that have established a future I/I rate.

Accuracy of Flow Estimates

In order to verify that the regression equations produced accurate flow predictions, flows calculated from the regression equations were plotted and compared against measured flows for the four storms. The majority of the regression plots showed that the regression equations produced flow estimates that closely matched measured flows. Differences between flows computed from the regression equations and measured flows are a result of one or more of the following:

- **System Operation.** The effects of flow diversions, pump stations, and wet weather bypasses are not consistent from storm to storm and result in potentially irregular system flows under similar rainfall events.
- **Rainfall Distribution.** The regression equations were generated from the rain gage that was thought to best represent the rainfall distributed over the entire monitor basin. However, variability of rainfall volume and intensity is normal across basins, resulting in differences in flow volume and timing.
- **Monitor Data.** It is common to have intermittent problems with flow measurement, particularly because of mismeasurement of velocity. The velocity probe on a flow monitor can be fouled by debris and scum, or backwater effects can change the velocity to depth relationships. In addition, some monitors were not operating during portions of the monitoring period. The regression equations were produced from storm events during periods where the monitor data appeared to be the most reliable. The majority of the monitored data are reasonable and appropriate for the uses of this study.
- **Antecedent Conditions.** RDII predicted by the regression equations will be most accurate when applied to periods when the storm intensity, duration, and antecedent conditions are similar to those used to generate the regression equations.

Direct Stormwater Inflow

Direct stormwater runoff inflows from the combined sewer areas are generated from within the surface runoff component (RUNOFF) of the model. Surface runoff hydrologic parameters were developed and calibrated for each of the combined sewer surface drainage subcatchments. Parameters included subcatchment area, shape, slope, percent imperviousness, vegetative cover, pervious and impervious roughness, depression storage capacity, and soil infiltration capacity.

Model Calibration and Verification

A comprehensive flow monitoring program was initiated during the months of January and February 1995 to obtain data with which to refine a preliminary model of the Corvallis CSO area. Data obtained from this study were used to calibrate the original CSO model. This model was used to characterize existing CSO volume and to evaluate the effectiveness of the remediation alternatives under the 5-year winter storm SFO design criterion specified by the

stipulation and final order (SFO) agreed to by the City of Corvallis and DEQ. The 5-year winter storm was determined to be the controlling condition for evaluating CSO alternatives as opposed to the 10-year summer storm.

The 5-year, 24-hour winter storm for the Corvallis area is 2.4 inches. The 5-year storm would not occur as an isolated event, but as a part of an overall storm pattern with different antecedent conditions. Therefore, a storm pattern with antecedent conditions was developed from a January 6 to January 19, 1995, storm period. Because the peak 24-hour period had rainfall greater than 2.4 inches, the rainfall values were scaled downward by a factor that equates the peak 24-hour rainfall to the desired 5-year event.

The model was extended from the CSO area to include city-owned pipelines with diameters greater than 10 inches in the separated sewer areas of the collection system. Calibration of the extended collection system model involved adjusting flows and hydraulic parameters in the separated system such that model-predicted flows, depths, and velocities more closely matched observed data. Calibration was performed based on model predicted versus monitored flows at a total of 25 locations using monitor data from the separated sewer system (March 4 to March 11, 1995, period).

Figures 3-8 through 3-10 compare modeled flows with monitored data at the monitors for the model calibration storm.

The model was verified by comparing model predicted versus actual flows at the WWRP for flows resulting from the January 6 to January 20, 1995, storm event.

Future Flow Monitoring Recommendations

Because of uncertainty about the I/I rate for future development, it is suggested that the recommended rate of 4,000 gpad be reviewed through future ongoing monitoring of representative City basins and that the flow input for the model be adjusted in the future as appropriate.

It is also advisable to monitor collection system I/I improvements or remediation measures, both before and after installation, to verify that they have produced the desired results.

FIGURE 3-8
Model Calibration: Comparison of Monitored versus Modeled Flow
Pipe 404083, Monitor H1

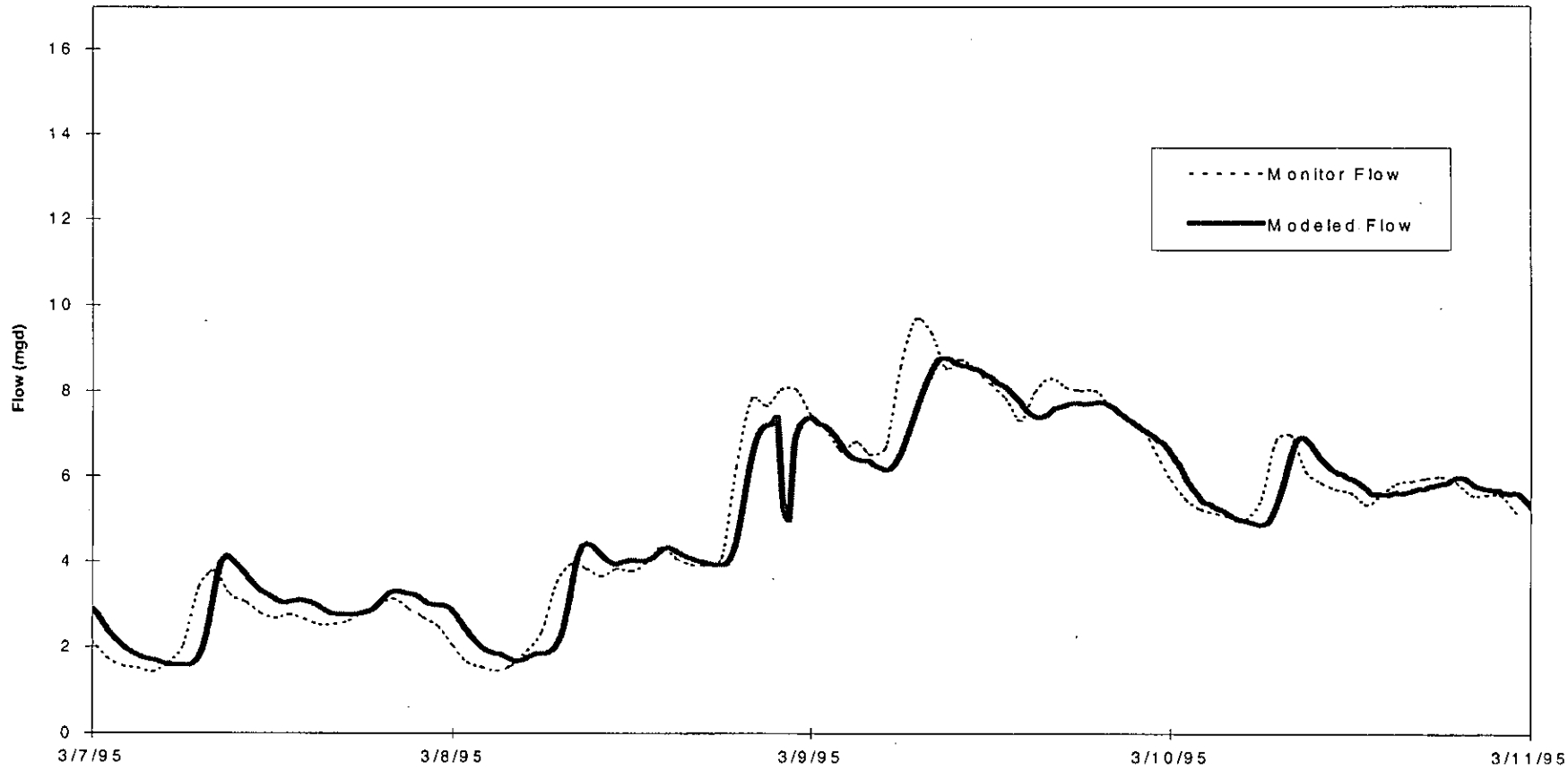


FIGURE 3-9
Model Calibration: Comparison of Monitored versus Modeled Flow
Garfield Basin – Pipe 402286, Monitor G2

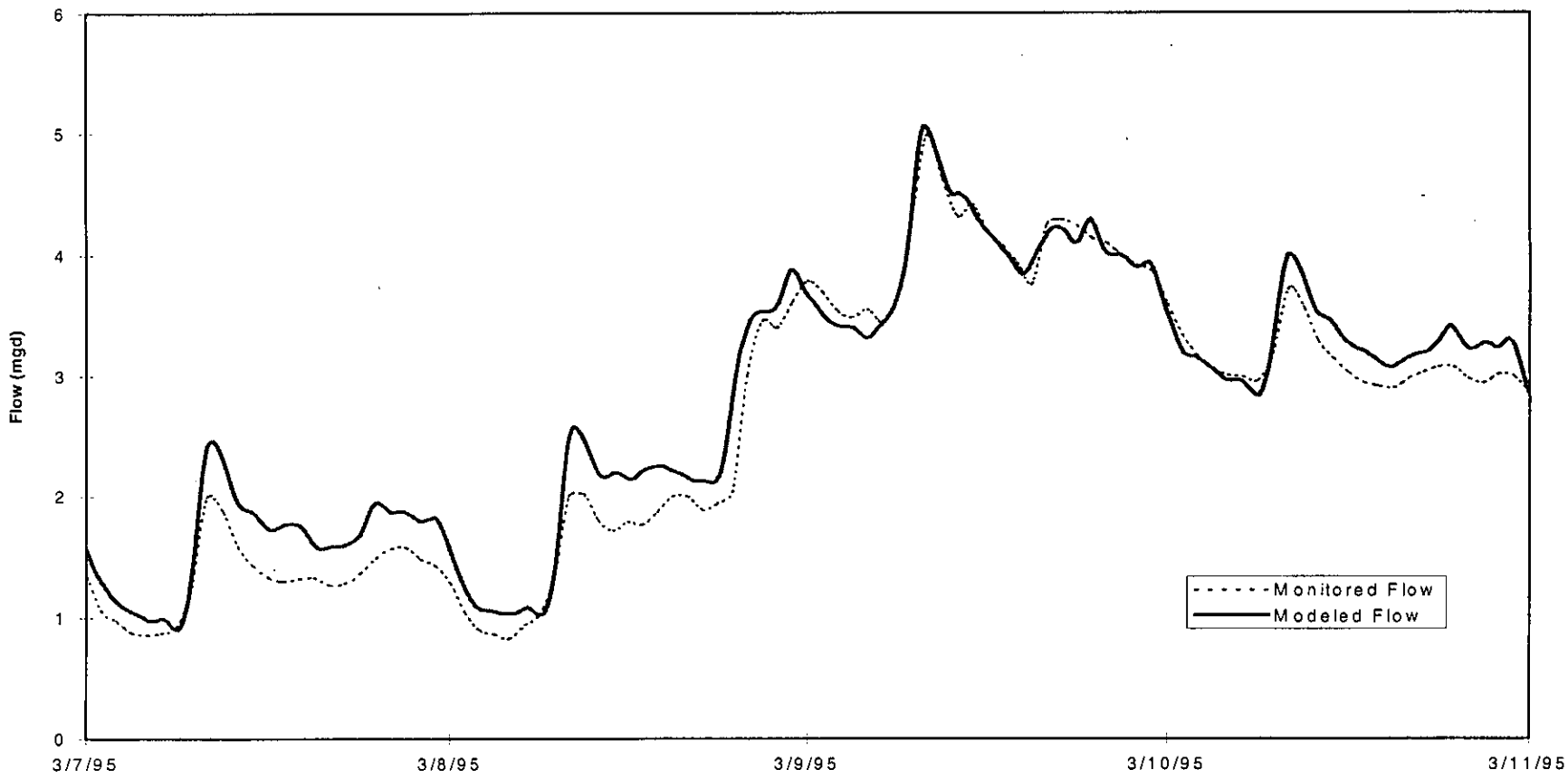
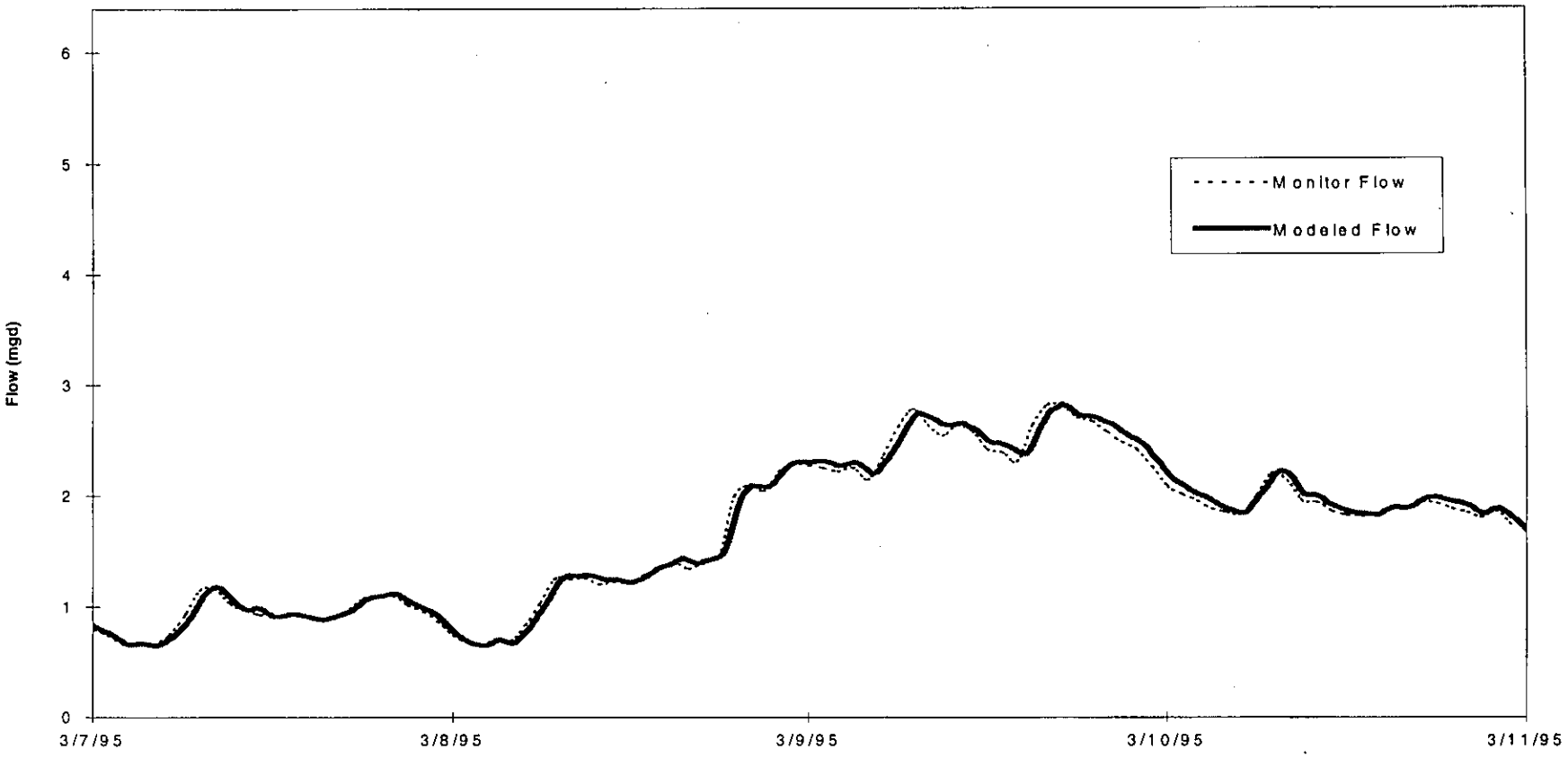


FIGURE 3-10
Model Calibration: Comparison of Monitored versus Modeled Flow
South Corvallis Basin – Pipe 4000519, Monitor S2



SECTION 4

Conveyance Facilities Assessment and Planning

Existing Conveyance System Capacity Assessment

Using the calibrated model described in Section 3, multiple model runs were performed to identify locations where system capacity does not meet the hydraulic grade line (HGL) elevation criteria developed for the analyses, as discussed below. An assessment of deficiencies was performed at three population demand levels and the 5-year, 24-hour storm condition:

- Existing (1995) population (47,485)
- 60,000 population
- 80,000 population

The existing and 60,000 population deficiency assessments are used only as input to the development of an implementation plan because they indicate the necessary timing for the improvements. With the exceptions noted below, the deficiencies associated with the 80,000 population demand as adopted in the Corvallis Comprehensive Plan were used to determine system improvement needs.

In addition, peak flows were generated for two other conditions—120,000 population and buildout population—but a system-wide assessment of deficiencies was not performed at these levels. The 120,000 scenario was used to determine peak flows for facilities in locations that are identified to have a potential maximum demand associated with a population of 120,000. The flows associated with probable maximum buildout (120,000) and a maximum buildout population were used only to size the collection system where deficiencies were identified for the 80,000 flows, and to assess system needs in individual basins for cases where a specific basin reaches buildout population. These locations are typically major system interceptors or pump stations that are downstream in the system and serve multiple basins. The locations are:

- Marys River interceptor/pump station
- Brooklane pump station
- 1st Street interceptor and relief line
- Crescent Valley interceptor
- Influent pump station at the wastewater reclamation plant

As a first step toward identifying system capacity deficiencies, model runs were performed with all pump stations set at existing capacities. Results of these model runs for the 80,000 population demand showed peak flow greater than capacity at the Marys River, Herbert, and Brooklane pump stations, as well as numerous locations where the water surface elevation was not within the acceptable limits (8 feet below the ground surface) set by the City because of backwater from the pump stations. The 8-foot criterion was selected to

identify deficiencies associated with basement flooding. It was estimated that if the HGL was 8 feet or less from the ground surface, that floor drains or fixtures at the basement level could allow for flow back-up into the basements which could be defined as an SSO. Some portions of the City were identified as areas where basements are not located. In these areas the criterion was relaxed and a deficiency defined as location where the HGL was 4 feet and less from the ground surface. The locations where the criterion was relaxed are noted in the following sections. Figures 4-1 and 4-2 show peak flows and capacities at Marys River and Brooklane pump stations. Subsequent runs assumed that necessary capacity upgrades to the pump stations were in place. Figures 4-3 through 4-5 show the deficient locations where the hydraulic grade line was estimated to be less than 8 feet below the ground surface at manholes for existing, 60,000, and 80,000 population demands, respectively. Table 4-1 lists the manholes where the hydraulic grade line is within 8 feet of the ground surface for flows associated with the 80,000 population land use. These locations are shown on Figure 4-5 as blue, yellow or red circles. Table 4-2 also lists manholes where this occurs, but the manholes listed represent flows in shallow pipelines where the crown (top of the pipe) is not buried deeper than 8 feet. These locations are also shown on Figure 4-5 and are within highlighted yellow areas. Figures 4-3 through 4-5 were used to identify locations of required improvements and their timing. In general, locations with less than 8 feet of HGL freeboard, that were not shallow pipes were considered to be deficient and warranted the development of an improvement project. The model results also assume pump station upgrades as well as pipeline additions that reflect anticipated near term improvements to the system, or new pipe that will convey flow from existing unserved areas that are developable. The upgrades are:

- Brooklane pump station
- Marys River pump station
- Herbert lift station
- Pipeline to take all future flows from newly developed areas in the Fillmore basin (outside the current city limits and inside the urban growth boundary) to the south through the Brooklane pump station
- Diversion pipeline from Beca at 16th Street to Taylor at 16th Street (completed, 1997)
- 1st Street CSO Relief Pipeline (will be completed with CSO Project, 2000)
- Influent pump station capacity increase at the treatment plant to 45 mgd (increased from 28 mgd, with the exception of the existing population condition which maintains the current design pumping rate of 28 mgd)

The addition of these system modifications allows for a better assessment of pipeline capacity deficiencies that are not the result of system back-ups caused by inadequate pumping capacity.

The following is a brief summary of the discussion of the deficiencies by basin for the flows associated with the 80,000 population demand shown on Figure 4-5.

FIGURE 4-1
Brooklane Pump Station
5-year Storm Flow, Existing Land Use

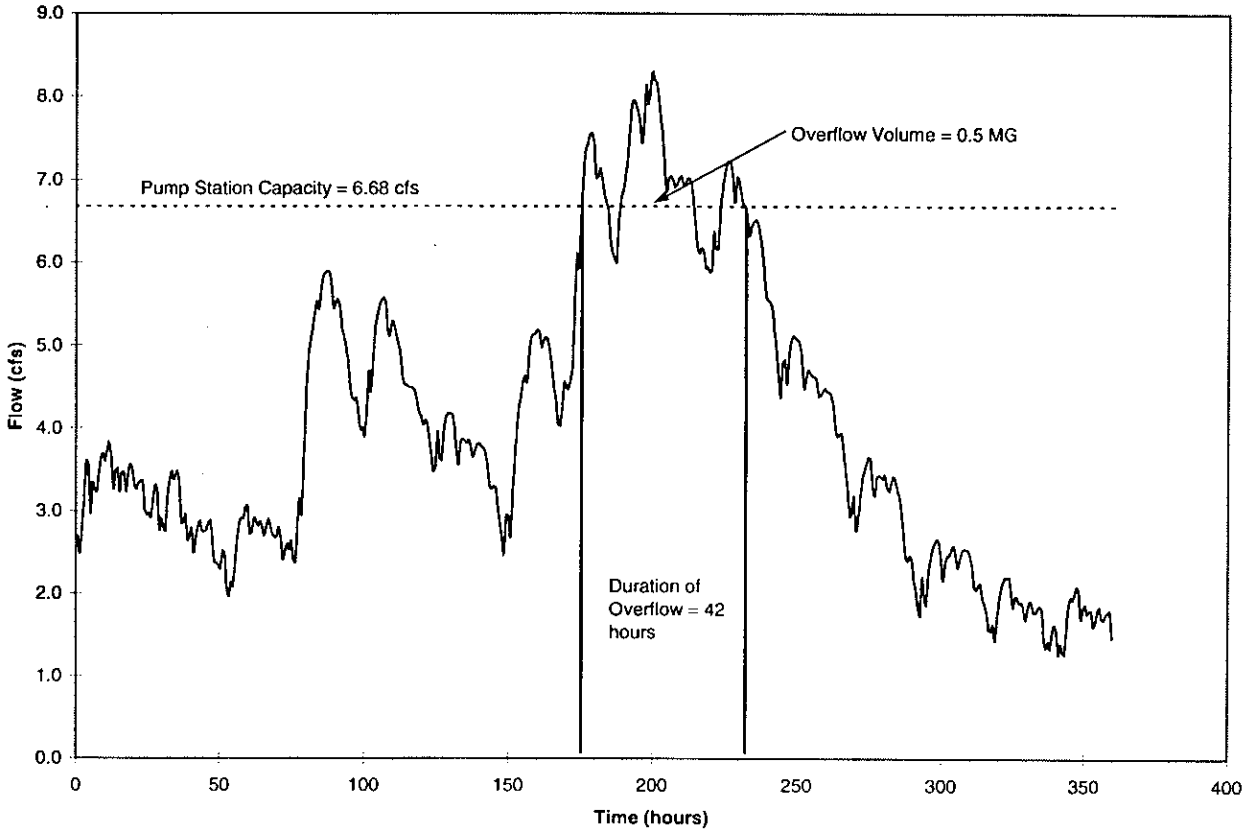
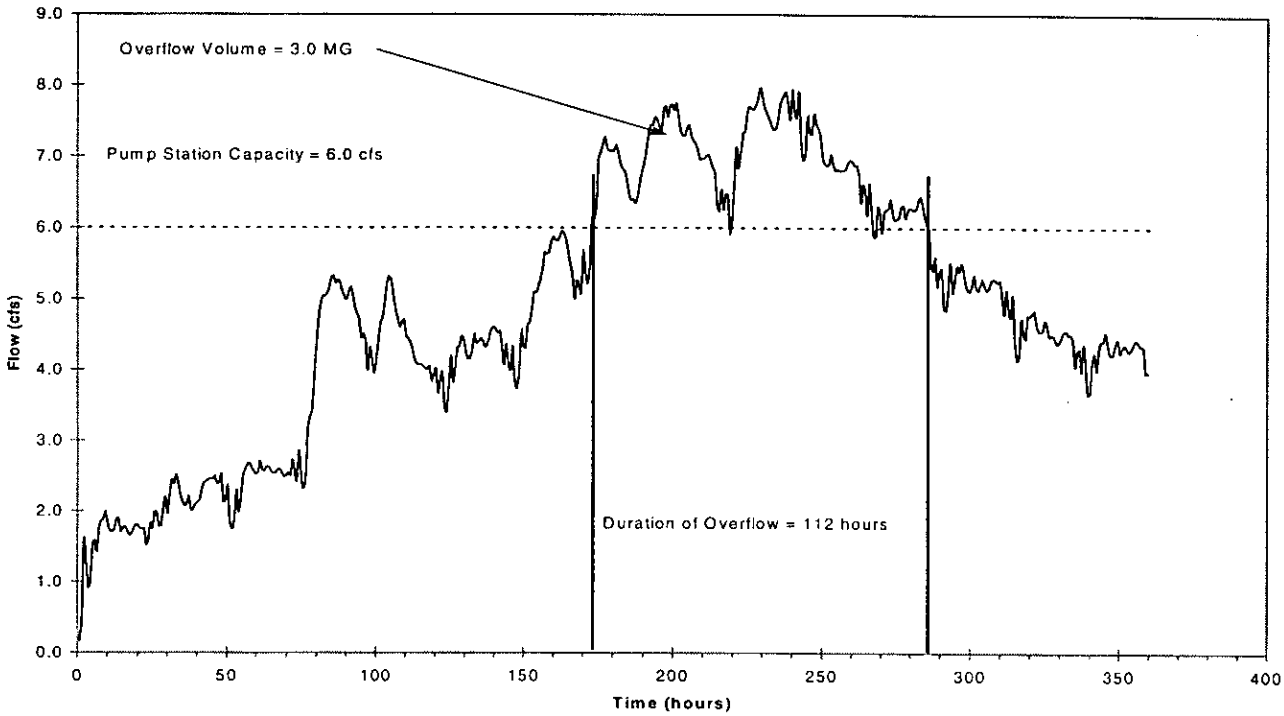


FIGURE 4-2
Marys River Pump Station
5-year Storm Flow, Existing Land Use



HGLDEFS.APR

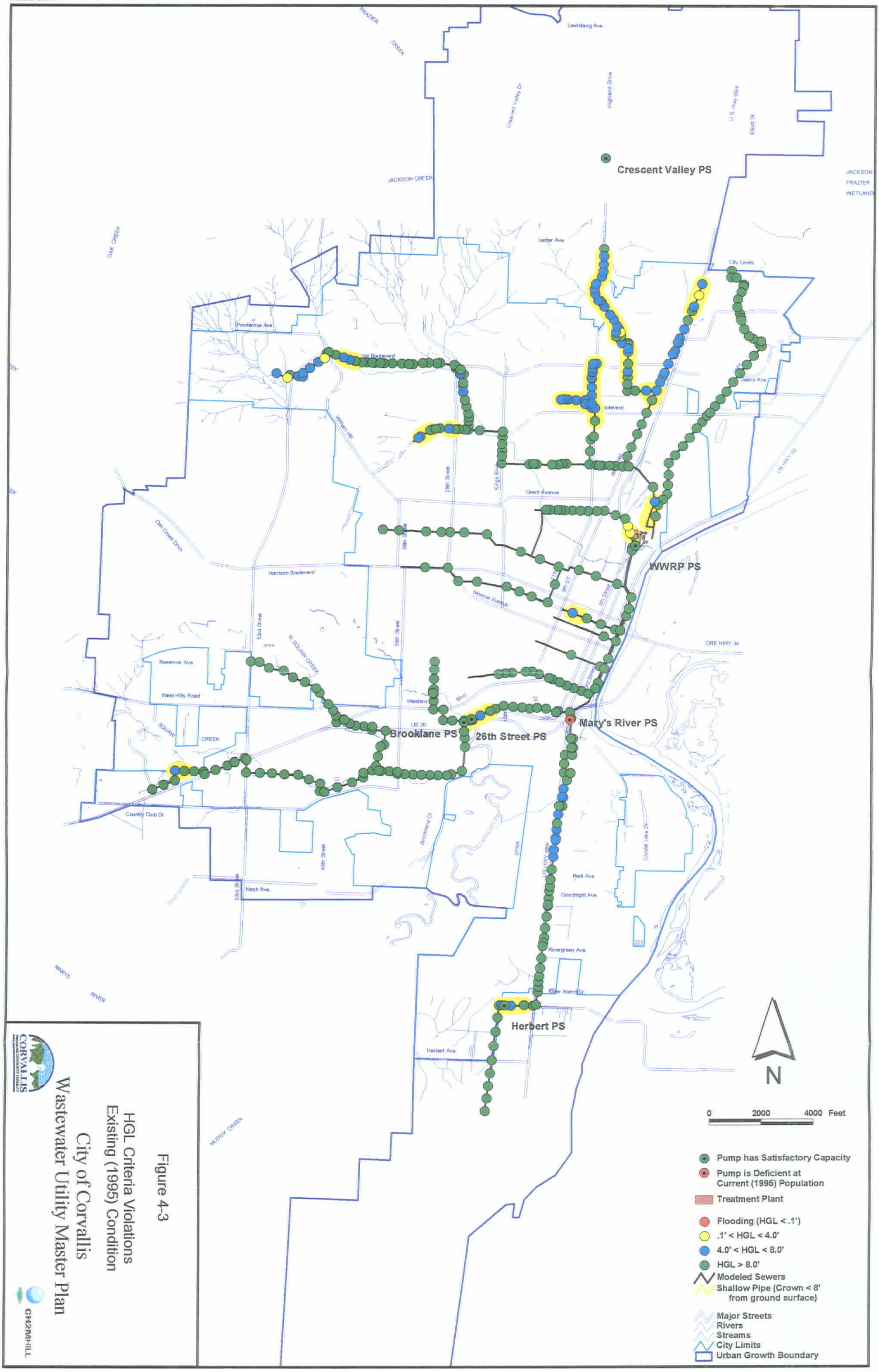


Figure 4-3

HGL Criteria Violations
Existing (1995) Condition

City of Corvallis
Wastewater Utility Master Plan

CORVALLIS

CH2MHILL

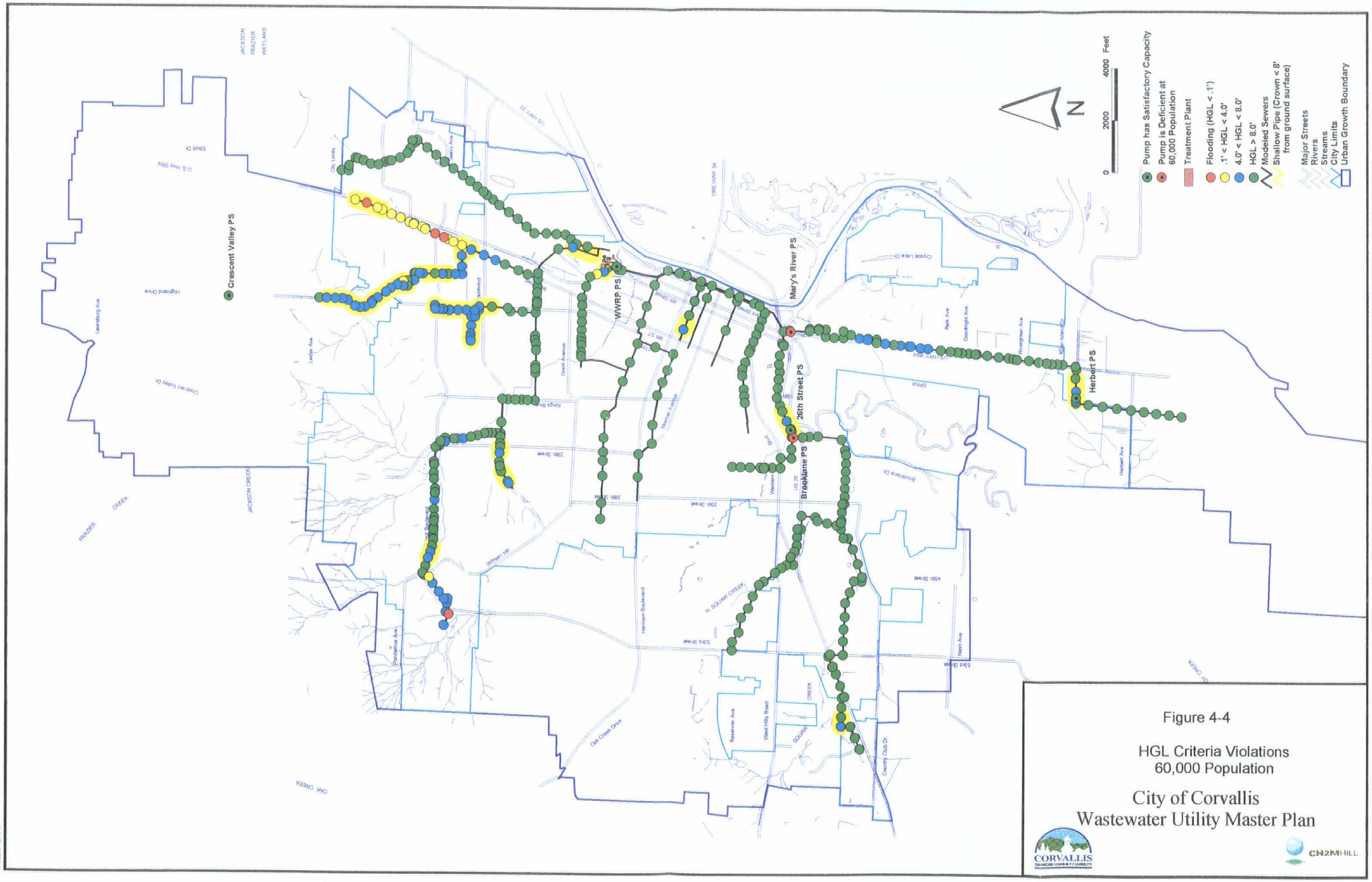


Figure 4-4

HGL Criteria Violations
60,000 Population

City of Corvallis
Wastewater Utility Master Plan

CORVALLIS

CH2MHILL

HGLDEFS.APR

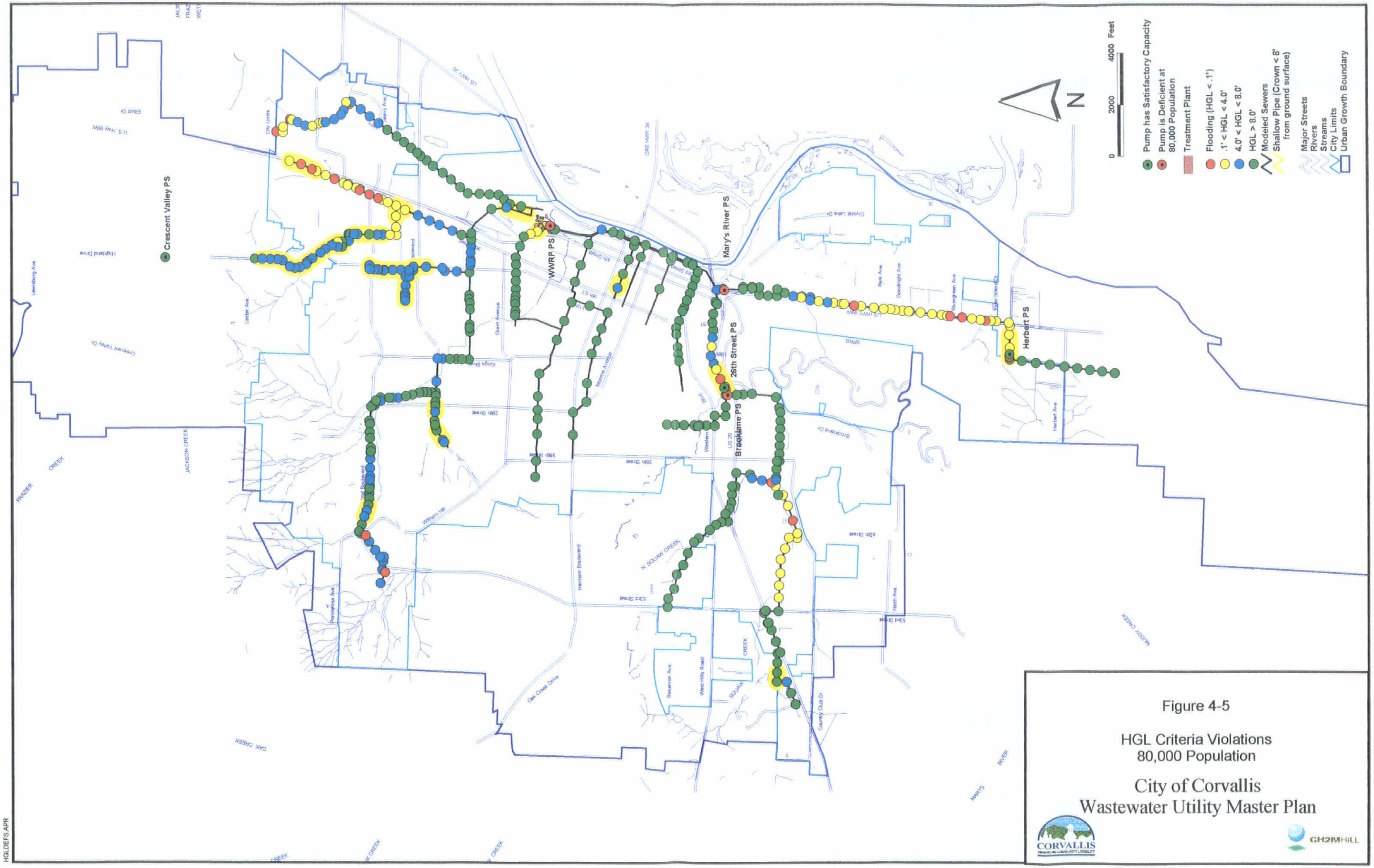


Figure 4-5

HGL Criteria Violations
80,000 Population

City of Corvallis
Wastewater Utility Master Plan

TABLE 4-1
Deep Pipeline Manholes with HGLs Less than 8 Feet Below Ground Surface for 80,000 Population Conditions during a 5-year Storm Flow[a]

Manhole Name	Distance of Water Surface Above Pipe Crown (ft)	Manhole Rim to Maximum Water Surface Elevation (ft)
4868051	2.0	0.79
5083074	2.4	2.97
5068044	3.7	3.15
4883032	2.4	3.46
5083070	2.8	3.48
3077004	8.6	4.29
4883003	1.8	4.35
3277035	8.3	4.79
5083061	2.7	4.98
3277034	8.3	5.15
2477008	16.3	5.24
5083062	2.4	5.39
4883006	0.9	5.41
5083032	1.4	5.92
5083033	1.2	5.98
5083013	1.3	5.99
3277017	6.0	6.28
5083060	2.2	6.29
5083034	1.3	6.48
Western	6.4	6.60
4880027	0.3	6.70
4868048	7.3	6.74
3277008	3.7	6.81
5283018	1.1	6.87
3077009	8.6	6.88
4880039	1.2	6.95
5080093	1.0	7.07
4868076	0.9	7.18
2274001	13.3	7.23
2474001	14.0	7.23
3277009	3.8	7.30
3277027	7.8	7.48
3477042	2.7	7.56
Jackson	9.4	7.60
VanBuren	8.4	7.60
2274003	11.0	7.72
2274002	12.1	7.73
2477007	14.9	7.73
4868047	3.9	7.78
5283017	1.1	7.91

[a] Model results assume existing land use patterns and that the new 1st Street relief with Beca diversion has been implemented. Total Flooding locations = 0. Total HGL violations = 104. Total HGL violations due to pipe elevation = 64. This list does not include nonsurcharged pipes (shallow pipes) that violate the 8-foot HGL criterion (see Table 4-2).

TABLE 4-2
Shallow Pipeline Manholes with HGLs Less than 8 Feet Below Ground Surface for 80,000 Population Conditions during a 5-year Storm Flow[a]

Manhole Name	Distance of Water Surface Above Pipe Crown (ft)	Manhole Rim to Maximum Water Surface Elevation (ft)
5080003	0.0	2.99
5283021	0.0	3.78
4480049	0.0	3.79
5283022	0.0	4.15
WWTF_OUT	0.0	4.40
WWTF_n2	0.0	4.40
WWTF_n1	0.0	4.42
5480003	0.0	4.48
5080051	0.0	4.49
4880046	0.0	4.68
4280019	0.0	4.71
4880043	0.0	4.74
5080067	0.0	4.78
5280008	0.0	4.92
5280007	0.0	4.92
5280006	0.0	5.07
5280009	0.0	5.18
5280034	0.0	5.22
4880028	0.0	5.23
3674032	0.0	5.30
4880045	0.0	5.35
4868049	0.0	5.37
2477005	0.0	5.46
4880021	0.0	5.46
4880030	0.0	5.50
5280001	0.0	5.53
5283010	0.0	5.53
4880044	0.0	5.55
3674030	0.0	5.56
4880005	0.0	5.60
4880004	0.0	5.61
5280005	0.0	5.63
4877029	0.0	5.63
5280003	0.0	5.67
5280004	0.0	5.70
5071057	0.0	5.75
4880012	0.0	5.75
3674031	0.0	5.76
4877032	0.0	5.83
5068063	0.0	5.96
5080001	0.0	6.00
5280002	0.0	6.11
5080032	0.0	6.65
5071061	0.0	6.71
5080031	0.0	6.78

TABLE 4-2
Shallow Pipeline Manholes with HGLs Less than 8 Feet Below Ground Surface for 80,000 Population Conditions during a 5-year Storm Flow[a]

Manhole Name	Distance of Water Surface Above Pipe Crown (ft)	Manhole Rim to Maximum Water Surface Elevation (ft)
4877030	0.0	6.79
5080100	0.0	6.99
5068045	0.0	7.01
4674023	0.0	7.18
4874022	0.0	7.24
5080002	0.0	7.31
4874006	0.0	7.36
3462002	0.0	7.41
5080029	0.0	7.53
5080030	0.0	7.47
4674002	0.0	7.57
4880025	0.0	7.57
2477004	0.0	7.69
5480002	0.0	7.69
5080024	0.0	7.70
5080038	0.0	7.79
4874033	0.0	7.83
4880026	0.0	7.91
MH_5	0.0	7.91

[a] Model results assume existing land use patterns and that the new 1st Street relief line has been implemented.
Total Flooding locations = 0.
Total HGL violations = 104.
Total HGL violations due to pipe elevation = 64. This list does not include the HGL criterion violations that did not occur in shallow pipes.

Fillmore

The previous analysis of the combined system indicated that the entire Fillmore basin could be directed to the east through the combined portion of the system without causing flooding in the Fillmore basin. However, this routing conflicts with the City's desire to minimize the addition of flow to any portion of the combined system. Directing more flow to Fillmore would add to existing high water and basement flooding conditions in upper Fillmore between 29th and 35th Streets, as well as basement flooding along Harrison.

Therefore, the initial analysis routed all future flow from Fillmore south to the Brooklane basin. The model showed that for this routing condition the collection system capacity was adequate in the Fillmore basin and hydraulic grade lines were lowered to an acceptable level. However, as described below this routing created concerns regarding high water surface elevations in the Brooklane basin.

South Corvallis

The model predicted flooding in the south Corvallis interceptor near Richland Avenue for the 80,000 population level assuming the pump station capacity at Marys River was increased to match the peak flow into the station. City staff confirmed that this is a problem area. The entire line between Cummings Avenue and Kiger Island Drive showed water surface elevations within 8 feet and less of the ground surface. The cause of the flooding and high water surface elevation was observed to be inadequate pipe capacity at the downstream end of the south Corvallis interceptor.

Brooklane, Country Club

Water surface elevations within 8 feet and less of the ground surface were predicted at several locations. The pipeline system upstream of the Brooklane pump station generally has adequate capacity through the 60,000 population demand. However, at 80,000 population deficiencies occur on the Country Club trunk in the form of flooding and multiple additional locations where the HGL is less than 4 feet from the surface.

Marys River Interceptor

As described in previous sections, the flow used for the determination of deficiencies in the Marys River Interceptor is based on a population of 80,000, but improvement sizing is based on 120,000. Flooding and high water surface elevations were predicted at the upstream end of the Marys River interceptor near the Brooklane pump station. Surcharged conditions were observed throughout the Marys River interceptor. The cause was identified as backwater from the existing 1st Street interceptor as well as inadequate pipe capacity from the point where flow enters the interceptor from South Corvallis to the 1st Street interceptor. Water surface elevations within the 1st Street interceptor are set based on the weir elevations at the diversion points to the combined sewer overflow (CSO) 1st Street Relief sewer. These elevations cannot be lowered without negatively affecting the desired diversion of flow from the combined portion of the system to the new wet weather treatment facilities. Shallow pipe is also a cause of HGL violations in the vicinity of the Brooklane pump station. Elevation of the top of pipe is within 5 feet of the ground surface for the first four pipe segments immediately downstream of the pump station (26th Street to 17th Street). Water surface elevations within 4 feet of the ground surface were predicted at

two locations between 15th Street and the discharge from the South Corvallis basin because of the combination of backwater and shallow pipe.

Beca

With the diversion at 16th Street in place, no flooding or hydraulic grade line problems were observed for this basin. HGL violations are seen at two locations at the lower end of Beca where the pipe crosses through a depressed area where the top of pipe is within 3 feet of the ground surface. This was identified by City staff as an acceptable violation of the HGL criterion.

Garfield

Flooding and high water surface elevations were most concentrated at the upper end of the Garfield basin. Since there are no basements in upper Garfield, higher water surface elevations can be tolerated. Therefore, an HGL elevation greater than 4 feet from the ground was determined to be acceptable and an HGL less than or equal to 4 feet from the ground was considered a deficiency that warrants improvements.

Highland

Water surface elevations within 8 feet of the ground surface were predicted in the upper portion of the Highland basin. Closer inspection showed that this was due to shallow pipe depths. Model flows were observed to be contained in the pipe, with no surcharges.

North Ninth

Modeling predicted several flooding locations, as well as numerous instances where water surface elevations were within 4 feet of the ground surface. City staff indicated that there are existing flooding problems in this area due to high wet season groundwater levels and infrequent, high-intensity, long-duration, winter storm events.

Crescent Valley Interceptor

The Crescent Valley interceptor in the Northeast basin is another location where the population associated with a total of 80,000 was used to determine system deficiencies and 120,000 for improvement sizing to provide the flexibility to accommodate higher future densities in these basins should they occur. Flooding was predicted at three locations and high water surface elevations were predicted in several others in the upper sections of the Northeast basin on the Crescent Valley interceptor. The source of the deficiencies was observed to be backwater from the influent pump station of the wastewater treatment plant (at 45 mgd). However, during subsequent model runs the capacity was increased to 60 mgd and backwater from the influent pumping system at the plant was eliminated. In turn, the flooding deficiencies were eliminated, but the last five manholes on the interceptor surcharge to within 4 feet of the ground surface. The Crescent Valley interceptor surcharges for most of its length, but there are only a few places where it is less than 8 feet below ground. These locations are concentrated at the upper end of the interceptor and where it crosses Stewart Slough. City staff pointed out that much of this area was within the 100-year floodplain, and there are no basements. For this reason it was determined that a hydraulic grade line less than 8 feet, but no less than 4 feet from the ground surface would be allowable.

Pump Stations

Existing Pump Stations

The approach and data sources used to complete the evaluation of five selected sanitary pump stations for the master plan are summarized here. The stations are: Brooklane, Marys River, Herbert, Crescent Valley and 26th Street. Information used for the assessment was taken from operations and maintenance manuals provided by the City, facility design drawings, discussions with pump station operations personnel, and visual inspection of the pump stations.

The assessment included review of the general structural, electrical, and mechanical condition and capacity of the pump stations, a comparison of existing capacities to projected flows, and the development of planning-level cost estimates for facilities to meet the projected flow requirements.

Capacity and Condition Assessment

In general, all of the facilities are in fair to good condition. There are no existing chronic maintenance or operating problems. Table 4-3 summarizes the pump station analysis, recommendations, and expansion or replacement costs.

Existing capacity of the pump stations was taken from the single pump performance conditions included in the literature provided by the City. It was assumed that the pump stations were designed for normal operation with full redundancy of one pump. Because no system performance curves are available, and pumping tests or detailed hydraulic analysis were not part of this analysis, an estimated peak station capacity with one or both pumps operating was not available. Therefore, the peak was assumed to be 150 percent of the single pump design operating capacity. The only exception to this assumption was for the Marys River lift station, where, as reported by the City, it only achieves 47 percent of its expected performance.

Replacement or upgrade costs were estimated for the three pump stations expected to see significant flow increases. The estimates were performed using an *Engineering News-Record* Seattle cost of construction index of 6500.

Combining flows from the existing and new South Corvallis interceptors and pumping through a new pump station at Marys River is considered a more favorable option than maintaining the existing pump station and constructing a new pump station to handle the new interceptor flows. Eliminating the existing pump station will result in reduced operations and maintenance costs. In addition, the existing pump station site is severely constrained, limiting its expansion potential. A new pump station can be designed with flexibility to cost-effectively handle both near- and long-term flow requirements by phasing the installation of the number and/or size of pumps.

Because the Herbert Pump Station lacks capacity for the existing condition, an alternative to an upgrade or station replacement was investigated. This was considered appropriate because the station is proposed for abandonment once the new South Corvallis interceptor is constructed, as described later in this section.

TABLE 4-3
Corvallis WWRP Pump Station Evaluation Summary

Pump Station Name	Existing Type	Number of Pumps	Pump Station Design Capacity[a]	Estimated Peak Capacity[a] (mgd)	Installed/ Maximum Impeller Diameter (inches)	Existing Condition	Existing Flows (mgd)	Future Flows by Population (mgd)			Expansion Type to Serve Population Listed[d]		Cost through 80,000 Population Conditions[b], as appropriate	Comments
								60,000	80,000	120,000	60,000	80,000		
Marys River	Wet well/dry pit	2	3.9 mgd @ 20 ft TDH	3.9[c]	17.25/22.0	Good	5.1	6.7	9.9	16.1	New Submersible	New Submersible	$4,170,000	Pump station upgraded in 1987. Current wet well configuration and site size and location are limiting constraint to increase capacity. Existing structure could possibly be used with modification for 60,000 if wet-pit pumps were installed and dry-well dividing wall were removed. New pump station for all South Corvallis flows is recommended for 80,000.
Brooklane	Wet well/dry pit	2	4.3 mgd @22 ft TDH	6.5	14.75@15 degrees	Good	5.3	7.1	10.1	17.2	Modify Existing	New Submersible	3,810,000	Upgraded capacity in 1983; new variable speed controls were installed in 1996. Current wet well configuration is limiting constraint for capacity increases. New pump station is recommended to serve existing and future population; phasing in additional and/or larger pumps as required.
Herbert Street	Pre-packaged, submersible	2	0.3 mgd @ 22 ft TDH	0.5	Imp 434/max size	Good	0.7	1.0	--	--	Construct Bypass	Abandon	$30,000	Cost associated with high flow bypass for existing and 60,000 conditions. At 80,000 flow conditions, a new South Corvallis is recommended, eliminating the need for the Herbert Station.
Crescent Valley	Prepackaged, flooded suction	2	0.3 mgd @ 158 ft TDH	0.5	12.63/14.12	Fair		--	--	--	--	--	--	Pump station is currently dedicated to the Crescent Valley High School. Any significant development in this service area will be served by a new gravity interceptor connected to the Crescent Valley interceptor.
26th Street	Prepackaged, submersible	2	0.1 mgd @ 17 ft TDH	0.2	6.9/7.1	Fair	0.15	0.15	0.15	0.3	Modify existing submersible at time of development by OSU		$225,000	This station currently serves a single apartment complex. City personnel pointed out an existing parking lot in the service area for this pump station which was slated for development by OSU. It is estimated that future flows will double at buildout to 0.3 mgd, requiring only new mechanical components and electrical gear.

[a] Existing pump station capacities were determined assuming full redundancy with single pump rated capacity used for the station capacity. Peak station capacity was assumed to be 1.5 times the single pump rated capacity reflecting current operating practices of running both pumps simultaneously during wet weather events.
[b] Costs were estimated using EPA cost tables and Northwest construction cost data updated using the *Engineering News-Record* construction cost index of 6500 for Seattle (includes 45 percent contingency, engineer, legal and administration costs).
[c] Peak capacity with both pumps on has been observed by City staff to be equal to design capacity.
[d] Population condition.
TDH = total dynamic head.

A high flow bypass was evaluated at the Herbert Pump Station, which would remain in service until the construction of the new interceptor. Currently, the station lifts the flow from approximately elevation 211 feet to elevation 229 feet. The ground surface elevation at the station is approximately 234 feet. As an interim solution to the capacity deficiency at the station, a high flow bypass is proposed from the station to the downstream gravity system. An 18-inch pipe at a minimum slope would have sufficient, if not excess capacity for the total flow projection of 1.0 mgd for the 60,000 population condition.

During the bypass condition, the 21-inch line upstream of the station would be subject to approximately 20 feet of surcharge. However, little additional surcharge is required to convey the 1.0 mgd flow, given the 21-inch diameter. Further, this 21-inch line is steeper than the ground surface, resulting in a pipe that becomes shallower as one moves upstream. The flat HGL, combined with the relatively steeper pipe, decreases the surcharge to between 10 and 15 feet at a distance of approximately 4,000 feet upstream of the pump station. The HGL is approximately 6 feet below the surface at this location. While not an ideal condition, the bypass is recommended to avoid pump station upgrades that will be abandoned once the interceptor is constructed.

Infiltration and Inflow Reduction Analysis

An infiltration and inflow reduction analysis was performed as a part of the CSO planning and pre-design projects. This analysis has been updated to reflect the addition of the separated sanitary sewer system to the collection system analysis model. The cost-effectiveness of rainfall-derived infiltration and inflow (RDII) reduction was evaluated considering both the combined and separated sewer systems for the 5-year design storm assuming flows associated with the 80,000 population conditions.

Prior Flow Monitoring Analyses

As part of the development of the *City of Corvallis Combined Sewer Overflow Remediation Plan*, flow monitoring and smoke testing were performed for the purpose of identifying the sewerage basins that were the largest contributors of RDII. In addition, the wet weather flows were quantified, and the impacts to the wastewater reclamation plant, combined sewer system, and combined sewer overflows were determined. Flow monitoring was performed during February 1995 throughout the sanitary system, and smoke testing was performed in October 1994 on approximately 16 percent of the Garfield basin. The results are documented in Technical Memorandum 2.4, dated July 28, 1995, which included a recommendation to perform a cost effectiveness analysis for the reduction of RDII. The cost-effectiveness analysis was performed and documented in *City of Corvallis Preliminary Design Report, CSO Remediation Project* (CH2M HILL, December 1996).

The analysis concluded that the cost effective solution included the construction of CSO conveyance and treatment facilities with capacity to handle the estimated peak flows and that RDII reduction was not cost-effective.

Assessment Methodology

Conveyance and Treatment

Using the existing hydraulic model of the conveyance system for 80,000 population conditions and the 5-year winter storm, model runs were performed in which the peak sanitary flow was reduced by 0, 10, 20 and 30 percent. This determined the effect of RDII flow reduction on the conveyance and treatment system. The 5-year storm pattern includes the 5-year, 24-hour rainfall depth of 2.4 inches as part of a typical wet season rainfall pattern lasting 15 days. It was assumed that the maximum achievable reduction in RDII was 30 percent. This assumption is based on past experience which has shown that RDII reduction beyond the level of 30 percent is very difficult to achieve, and is almost never cost-effective since it requires replacement of large portions of the collection system.

The system response was examined for each level of RDII reduction using the hydraulic model. Minimum component sizes required to convey the 5-year, 24-hour winter storm flows with a reasonable factor of safety to prevent overflows were determined.

Capital costs and operation and maintenance costs associated with conveyance and treatment of the reduced sanitary flows and reduced system component sizes were estimated using information generated initially in the Corvallis CSO remediation plan.

RDII Reduction

Using the regression models of RDII, peak flow rates caused by the 5-year winter storm were determined for each of the twenty-five sub-basins. Peak flows from upstream monitors were subtracted in order to isolate the sub-basin and calculate the local RDII flow from that sub-basin alone. The average base sanitary flow for each sub-basin was then subtracted out to give the peak RDII flow for the localized basin. The total length of each size of pipe in each of the 25 sub-basins was determined, and a weighted average pipe diameter was computed for use in estimating the costs associated with conducting a sanitary sewer evaluation study (SSES) and making general sewer main improvements.

Results of smoke testing conducted in portions of seven of the 25 separated sewer sub-basins monitored as part of the Corvallis CSO remediation plan were used in the I/I analysis. The seven sub-basins (H1, N1, G2, G3, G4, N2, and N2A) were selected for testing after initial flow monitoring indicated high RDII quantities. The smoke testing identified a significant number of system defects and possible inflow or infiltration sources. Defects were identified for the following types: manhole, downspout, catch basin, cleanout, lateral, sewer main, open plumbing. Estimates of RDII quantities in the smoke-tested basins for a 5-year, 24-hour winter storm were made from the smoke testing results. Costs to repair defects were also estimated as part of the smoke test study. Results of the smoke test study are documented in a memorandum prepared by CH2M HILL for the City of Corvallis dated March 7, 1995 (provided in Appendix C).

Using the characterization of known defects and the per-defect RDII flow estimates from smoke testing of portions of the north Corvallis sanitary system, an estimate was made of the percentage of defects which would need to be repaired to achieve a given level of RDII reduction in the smoke tested basins. Table 4-4 shows the percentage of each type of defect which would need to be repaired system-wide to achieve a given level of RDII reduction.

For the case where fixing 100 percent of known defects system-wide did not achieve the required level of RDII reduction it was assumed that additional lateral and sewer main defects would be identified and repaired. The level of repair percentages for these general laterals and sewer mains are the additional repairs needed to achieve the desired target RDII reduction level. SSES percentages were set at the highest repair level since the defects would have to be identified in order to be repaired.

The number and type of defects in non-smoke tested sub-basins were estimated by applying the rate of defects per RDII flow from smoke tested basins to the non-smoke tested basins. For example, it was estimated that the smoke tested areas produced 5.6 mgd of RDII from all of the identified defects. Forty-two of these defects were leaking manholes. From the 1995 wet season flow monitoring data, it was estimated that the entire G2 sub-basin contributed a peak RDII of 1.6 mgd during the 5-year storm. Therefore, it was estimated that sub-basin G2 would contain a total of 12 leaking manhole defects, as follows:

$$\frac{\text{42 Identified Manhole Defects}}{\text{5.6 mgd RDII Caused by Identified Defects}} \times \text{1.6 mgd Actual Basin RDII} = \text{12 Estimated Manhole Defects}$$

The totals for each type of defect shown in Table 4-4 were estimated using this methodology.

TABLE 4-4
Estimated I/I Removal for a Given Level of System Repair

Defect Type	Number of Defects	Repair Cost (Each)[a]	Cost per Gallon I/I Removed	Contribution (gal/day)	Level of Repair per Reduction				I/I Removed (MG)			
					0%	10%	20%	30%	0%	10%	20%	30%
Manhole	48	217	$0.06	3,540	0%	80%	100%	100%	0.00	0.14	.017	.017
Downspout	29	1,050	$0.33	3,177	0%	80%	100%	100%	0.00	0.07	0.09	0.09
Catch Basin	17	7,500	$0.87	8,583	0%	80%	100%	100%	0.00	0.12	0.12	0.15
Cleanout	0	1,500	$1.04	1,440	0%	80%	100%	100%	0.00	0.00	0.00	0.00
Lateral	83	3,000	$2.08	1,440	0%	60%	100%	100%	0.00	0.07	0.12	0.12
Sewer Main	15	3,750	$2.60	1,440	0%	40%	100%	100%	0.00	0.01	0.02	0.02
Open Plumbing	16	375	$375.00	1	0%	0%	0%	0%	0.00	0.00	0.00	0.00
Totals	**208**								**0.00**	**0.41**	**0.55**	**0.55**
Additional Systemwide Improvements Needed												
General Laterals	9,557	6,000[b]	$4.17	1,309	0%	0%	10%	20%	0.00	0.00	1.25	2.5
General Sewer Main	955,656 lf			13	0%	0%	15%	50%	0.00	0.00	1.88	6.25
SSES	955,656 lf				0%	80%	100%	100%				
Total Reduction									**0.00**	**0.41**	**3.68**	**9.30**

[a] Cost per defect.
[b] Cost per lateral; includes costs for defect identification and its repair.
lf = lineal feet
MG = million gallons

The cost to achieve a particular level of RDII reduction was estimated using the estimates of the percentage of each type of defect required to be repaired to achieve a particular level of RDII reduction, estimates of the repair cost for each type of defect, and the collection system inventory data supplied by the City. The cost of conducting an SSES was also included in this cost calculation. The costs associated with an SSES and the repair of general sewer main defects were estimated per lineal foot based on the total lineal feet of sanitary sewer pipe in the separated sewer areas. SSES costs were estimated as $0.90 per lineal foot. The collection system inventory data were used to compute a weighted average pipe diameter in the separated system of 10 inches. Pipe replacement was estimated to cost $40 per lineal foot; the estimate includes materials, excavation and backfill, pavement restoration, and contractor's overhead, contingency, and profit.

For each of the RDII reduction levels assumed (expressed as a percentage of wet weather flow reduction) the costs associated with conveyance and treatment and the cost to achieve RDII reduction were estimated. Total cost is then the sum of the conveyance and treatment cost and the reduction cost. By comparing total costs for various reduction levels, a total minimum cost can be selected along with its associated level of RDII reduction. It is this level of reduction that is defined as the most cost effective.

I/I Reduction Cost

The I/I reduction costs for 10, 20 and 30 percent reduction levels reflect the current data for the collection system inventory within each sewerage basin and the corresponding flow data. This resulted in a slight increase in reduction costs that were developed in 1996 and documented in the memorandum included in Appendix A. The costs are shown in detail in Table 4-5.

Collection System Costs

Collection system costs associated with peak flow conveyance were the most significant addition to the updated analysis and include costs for the following facilities not included in prior studies:

- Marys River pump station
- Brooklane pump station
- Herbert lift station
- Marys River interceptor improvements
- South Corvallis interceptor and associated diversion pipes
- North Ninth pipe improvements
- Garfield pipe improvements
- Crescent Valley interceptor improvements

These costs are reflected in Table 4-5. The costs produced for the cost effectiveness analysis were developed specifically for purposes of comparing relative total costs of varying levels of RDII reduction. They were developed prior to much of the final selection, sizing, and costing of the recommended conveyance alternatives. Therefore, the alternative conveyance analysis performed in south Corvallis is not reflected, nor are contingency, engineering and administrative costs. Further pipe sizing for each of the reduction levels was based on the 80,000 population demand versus the buildout or 120,000 demand level used for the final project recommendations included in this document. For all of these reasons, the costs do

not match those shown elsewhere in the document, specifically in Tables ES-1 and 4-8. Modifying the costs to match the others was not considered necessary as the costs for each of the reduction levels would have changed by the same relative amount, and the conclusion regarding the cost effective level of RDII reduction would be the same.

TABLE 4-5
Cost-Effectiveness Analysis—80,000 Population

	0% Reduction		10% Reduction		20% Reduction		30% Reduction	
System Component	**Size**	**Cost**	**Size**	**Cost**	**Size**	**Cost**	**Size**	**Cost**
Conveyance and Treatment Costs—Capital Costs								
Total Wet Weather I/I Convey and Treat Cost		$25,856,375		$24,186,371		$22,075,767		$19,875,270
Dry Weather Treatment Plant Improvements		$2,085,000		$1,985,000		$1,985,000		$1,985,000
Marys River Pump Station Improvements		$722,000		$662,000		$632,000		$582,000
Brooklane Pump Station Improvements		$685,000		$630,000		$600,000		$560,000
Herbert Pump Station Improvements		$719,088		$719,088		$719,088		$719,088
Collection System Pipe Improvements								
Marys River Interceptor		$1,000,781		$1,000,781		$1,000,781		$921,756
Country Club		$762,469		$762,469		$762,469		$762,469
South Corvallis								
Diversion Pipe to New South Corvallis Interceptor @ Tunison	18-inch	$240,845	15-inch	$200,970	15-inch	$200,970	15-inch	$200,970
New South Corvallis Interceptor Downstream of Tunison	24-inch	$1,171,600	24-inch	$1,171,600	24-inch	$1,171,600	24-inch	$1,171,600
New South Corvallis Interceptor Upstream of Tunison	18-inch	$656,850	18-inch	$656,850	18-inch	$656,850	18-inch	$656,850
North Ninth		$1,188,426		$1,174,940		$1,154,911		$1,154,911
Garfield		$178,304		$178,304		$178,304		$178,304
Crescent Valley Interceptor		$1,172,496		$1,172,496		$1,172,496		$1,172,496
Total Convey and Treat Cost		$36,439,234		$34,500,869		$32,310,237		$29,940,714
RDII Reduction Costs								
Total RDII Reduction Cost		--		$2,574,918		$12,187,734		$33,040,845
Total Costs		$36,439,234		$37,075,787		$44,497,971		$62,981,559

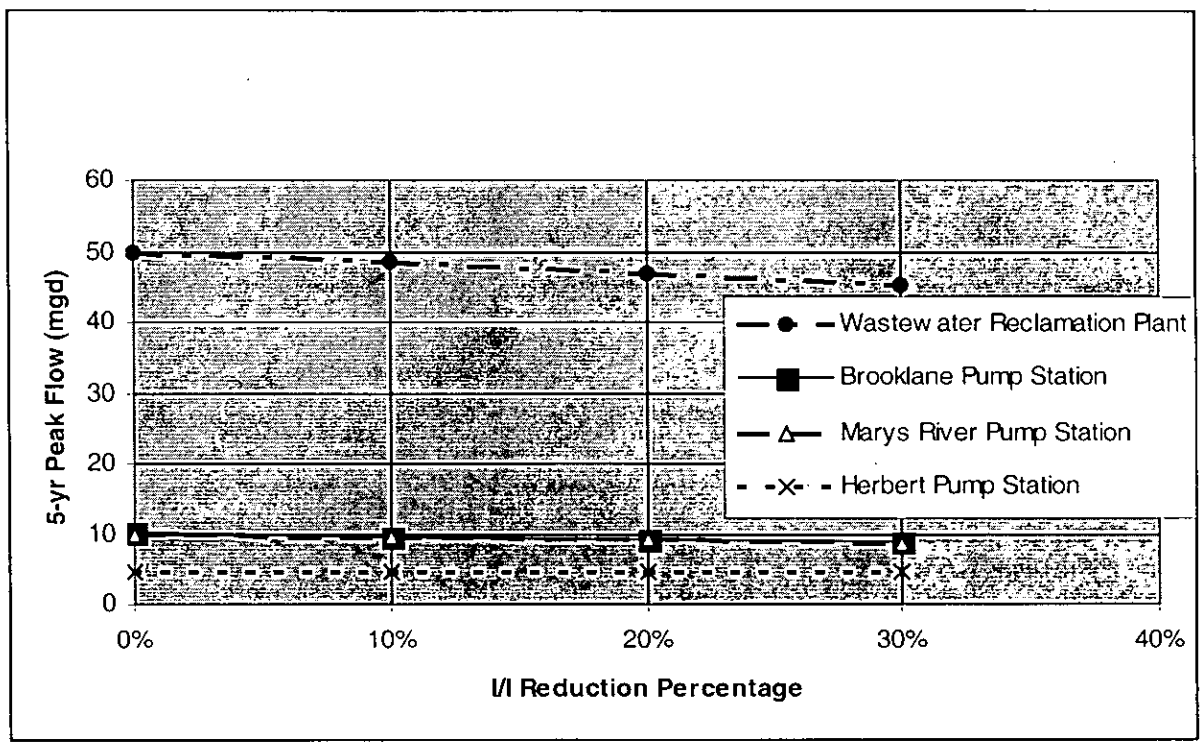
Wastewater Treatment Costs

Treatment costs for the Wet Weather Treatment Facility (CSO Facility) remained essentially the same as those prepared for previous studies with costs ranging from over $25 million assuming no RDII reduction to over $19 million assuming a 30 percent reduction. Additional costs that were not included as a part of the prior analysis and can be impacted by reduced flows are selected components of the conveyance and pumping elements of the Wastewater Reclamation Facility. These costs are approximately $2 million; however, they did not change appreciably with increased RDII reduction. These costs are shown in Table 4-5.

Alternative Reduction Levels

Consistent with the analysis performed when only the combined system was evaluated, the current assessment was based on reduction levels of 0, 10, 20 and 30 percent. The impact to peak flows at selected locations where collection system improvements were identified is shown in Figure 4-6. As shown, the decrease in peak flow rate is 10 percent for a 30 percent reduction in RDII. This type of response is due to the fact that flows from the different basins combine at different times and are not the result of peak flow hydrographs that coincide in time. RDII reduction likely has a more significant impact on peak flows at the basin or subbasin level. However, given the configuration of the City's collection system, relatively small decreases in peak flow are realized in downstream portions of the system where many of the more significant collection system needs have been identified.

Figure 4-6
Peak Flows vs. I/I Reduction Levels (80,000 Population)



Cost Evaluations

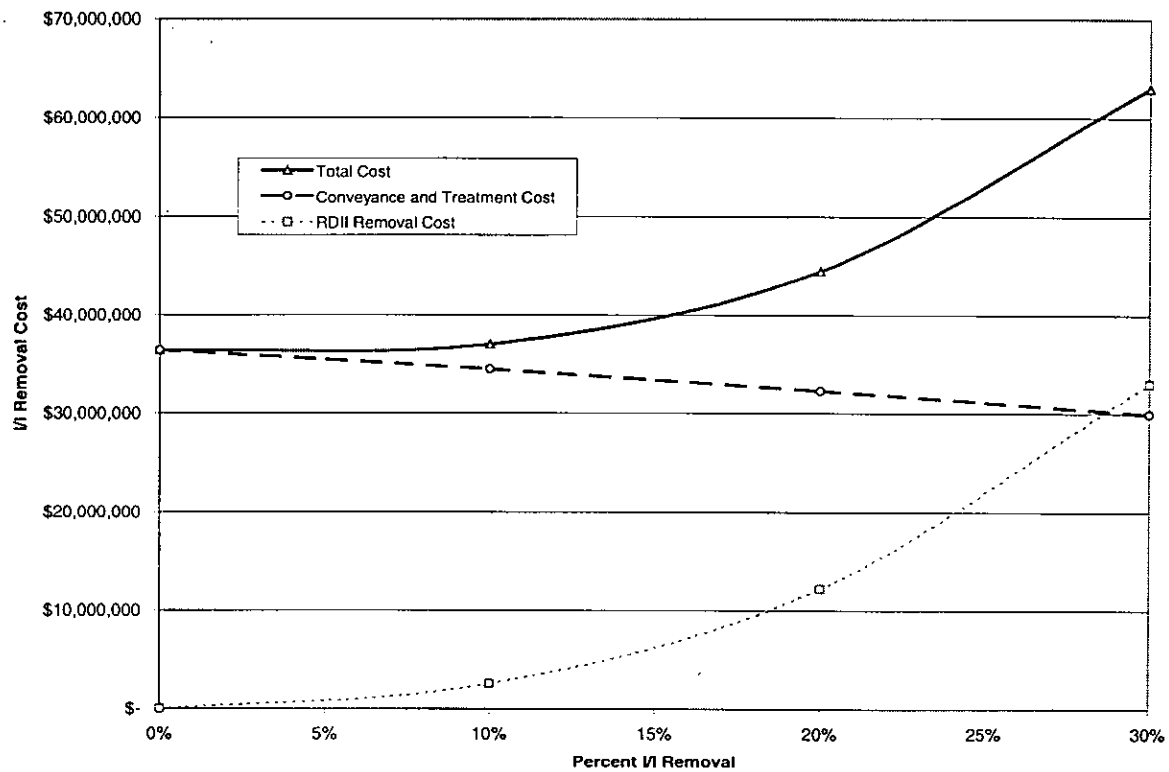
Costs are summarized in Table 4-5 for all of the system elements that comprise the cost effective analysis. The cost-effectiveness curves are shown in Figure 4-7.

Conclusions and Recommendations

As concluded from the analysis and shown graphically in Figure 4-7, the conclusion has not changed from the prior analysis. Further RDII reduction is not cost-effective for the City's separated and combined sewer systems even with the addition of conveyance system needs

in the separated system. The costs for further RDII reduction do not create enough off-setting decreases in conveyance system costs because peak flows are not significantly reduced. However, it is critical the City take measures to not allow existing RDII to worsen and control RDII from newly developed areas to acceptable levels.

Figure 4-7
Cost-Effectiveness Analysis Results *Including* Separated Sanitary Sewer System Conveyance Costs
80,000 Population



This conclusion does not imply that no corrective actions should be focussed on the collection system. It is recommended that the City's comprehensive system maintenance program that includes the elimination of known inflow sources; repair, rehabilitation and replacement of system elements that are in poor condition; and system monitoring should be continued and improved. Additional funding should be committed to improve and strengthen all aspects of the program. Consideration should be given to the following suggestions for improving the City's maintenance and I/I correction program:

- Implement a more comprehensive continuous flow monitoring program in the collection system to assess changes/trends over time with respect to I/I flows. The program should be directed primarily on selected basins with the greatest I/I. All basins should be monitored on at least a 5-year cycle; priority basins on a more frequent cycle.

- Summarize defects from current and prior smoke testing activities and develop a 3 to 5-year program to remove inflow sources where practical system-wide
- Perform smoke testing of the entire collection system on a 5 to 10-year cycle
- Continue manhole inspection and rehabilitation activities (i.e., grouting and plugging pick holes). Manholes should be inspected on a 1 to 2-year cycle.
- Perform pre and post flow monitoring of sub-basins selected for more intense corrective measures to assess the effectiveness of the I/I corrective actions. Perform TV inspection of lines in all basins on a 5 to 7-year cycle, targeting the basins with the highest I/I levels first.
- Perform pipeline repair/replacement where pipe defects are identified.
- Perform pipe cleaning based on a 4 to 5-year cycle
- Implement a lateral inspection and testing program at the time of all property sales
- Lateral replacement should accompany any main line replacement work.

For new developments, consideration should be given to the following suggestions:

- Review current City standards to insure construction materials and methods are reflective of the latest industry best practices.
- Ensure the City's developer review and approval process is comprehensive and thorough.
- Require rigorous testing and certification of all new developments, i.e. City witnessed vacuum testing of all collection lines, manholes, and laterals.
- If necessary, increase the resources available to implement the more comprehensive I/I best management programs; by either increasing the commitment of City staff and/or contracting with the private sector. Additional resources could be directed at the new development program, i.e. new construction inspections allowing City resources to focus on existing system I/I corrective activities.

Private service laterals are included as one of the elements of the system that should be monitored and maintained. RDII from new developments must be controlled by more stringent construction standards that require new construction materials and installation methods as they become available and more stringent inspection programs to ensure proper workmanship. The City of Corvallis has initiated a program to assist homeowners with the cost of installing backflow devices to help prevent basement flooding. Homeowners are responsible for the first $100 of installation costs. Under this program, the backflow prevention devices belong to the homeowners who are responsible for their continued maintenance and repair.

These efforts are essential to management of the collection system and a basic assumption regarding future RDII levels. The assumption is that no increases in RDII from *existing sources and sewered areas* are included in the wet weather flow estimates for the 5-year storm. RDII from existing sources and sewered areas shall not increase. This should not be

confused with the allowance for future RDII from newly developed areas, which has been built into the flow projections.

Alternative Conveyance System Configurations Assessment

South Corvallis

Replacement of the downstream portions of the existing interceptor was considered in light of existing plans for a parallel line to serve future south Corvallis flows. It was determined that a diversion of flow at several locations to a new parallel interceptor that utilizes the capacity of the existing system would be the preferred alternative. The new line runs parallel to the existing railroad track west of the existing line. An analysis was also performed to determine the feasibility of eliminating the Herbert Street lift station by constructing a deeper, new parallel interceptor. The alternative necessitates the use of special construction techniques (boring methods) for the new parallel interceptor due to increased depth requirements. The analysis showed that elimination of the lift station was feasible and also cost effective over the life of the new interceptor. The cost comparison is shown in Table 4-6. Costs should be refined and results verified at the time either the pump station must be improved or the new parallel interceptor is constructed.

Fillmore

As described in previous sections, there is a desire to minimize the amount of additional area directed to the combined portion of the system. However, it is desired to serve the eastern portion of the outer Fillmore basin by gravity and to follow the City's plan to serve some of outer Fillmore through the combined system. Modeling analyses were performed based on this routing scheme and it was determined that there was some available capacity in the combined system. In addition, impacts to the Brooklane and Marys River systems would be greater if the entire outer Fillmore basin was directed to the south. Therefore, the flows from the western portion of the Fillmore basin were split. The flow split routes flow west of Walnut Boulevard south to Brooklane, and flows east of Walnut to the combined system in the Fillmore basin.

Marys River Interceptor

Several potential alternatives were suggested as solutions to the problems in this area including increasing the size of the Marys River interceptor, creating in-line storage in South Corvallis, and lowering the elevations of weirs separating the existing 1st Street interceptor and the 1st Street CSO relief interceptor. The weir improvements could decrease the effect of backwater in the interceptor. However, adjustment of weirs could not be accomplished while still meeting the desired CSO diversion requirements to the wet weather relief and treatment system. Therefore, only a replacement option was considered for future improvements.

North Ninth

Possible solutions included replacement of the existing pipe that has inadequate capacity, as well as a diversion of some flow from the North Ninth basin to the Crescent Valley interceptor. A cost comparison was performed to assess the potential diversion of the North

Table 4-6
Herbert Street Pump Station Cost Effectiveness Analysis

Alternative	Item	Size	Quantity	Unit Cost	Total Cost
New Herbert Street Pump Station to Meet 80,000 Population Planning Criteria	Herbert Lift Station (80,000 Milestone)	8.5 mgd		$195,000/mgd	$ 2,100,000
	South Corvallis Parallel Interceptor (Trench @ 15' average depth)[a]	30-inch	7000 lf	$435/lf	$ 3,000,000
		36-inch	4000 lf	$470/lf	$ 1,900,000
	Operation & Maintenance for 20 Year Pump Life (Present Dollars)[b]	8.5 mgd		$140,000/yr	$ 2,800,000
	Total Cost				**$ 9,800,000**
Deeper New Parallel Interceptor to Meet 80,000 Population Planning Criteria	South Corvallis Parallel Interceptor (Bore - 25' Average Depth)[1]	30-inch	7000 lf	$875/lf	$ 6,100,000
		36-inch	4000 lf	$875/lf	$ 3,500,000
	Total Cost				**$ 9,600,000**

[a] Cost includes 25% contingency

[b] Annual O&M calculated as: $O\&M = \$22.9 * Q^{0.818} * D^{0.939} * 6500/4900$, where Q = design flow in mgd and D = hours per year of operation (estimated at 1200).

Ninth excess flows to the Crescent Valley interceptor. Several diversion locations were tested along with a no diversion alternative. Cost estimates were produced for two diversion alternatives and the no diversion option. As shown in Table 4-7, the lowest cost and preferred option is to divert at Ponderosa to the Crescent Valley interceptor. Either diversion option is less expensive than no diversion, which would require extensive replacement of the North Ninth Street line and result in considerable construction impact to this business district.

Crescent Valley Interceptor

The only significant system located and modeled in the Jackson Creek, Frazier Creek, Lewisburg, and Northeast basins is the Crescent Valley interceptor. Because the HGL deficiencies in this line are primarily created through backwater caused by insufficient pumping capacity at the influent pump station, alternative solutions focused on varying the increase to pump rates. The alternative pumping rates and the hydraulic conditions in the interceptor were influenced by the selected alternative in the North Ninth basin which includes a diversion to the Crescent Valley interceptor. Alternative pumping rates were investigated with the diversion from the North Ninth system in place. An alternative was selected that increases the pumping rate but minimizes the pumping required by continuing to allow surcharge to occur in the interceptor within the acceptable HGL criteria.

Wastewater Reclamation Plant

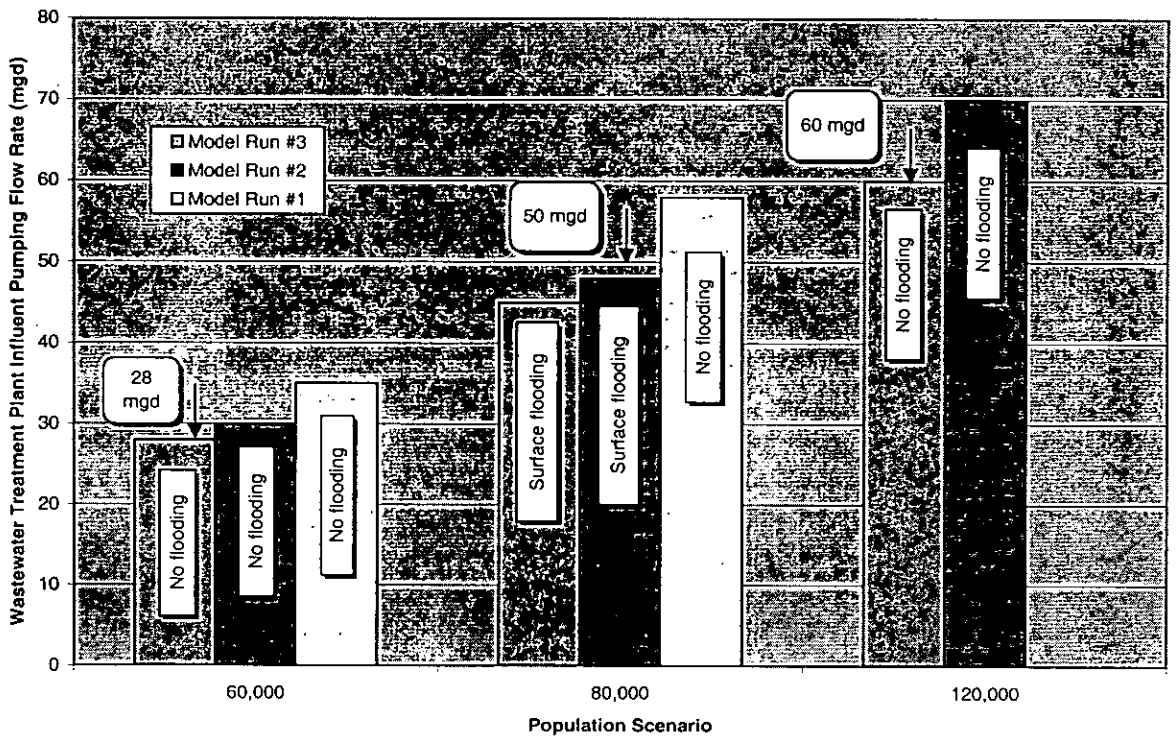
Series of model runs were performed to determine the level of influent pumping at the wastewater reclamation plant necessary to avoid hydraulic backups in the collection system that result in flooding or HGL elevations too close to the surface. Figure 4-8 shows the results of these model runs for 60,000, 80,000, and 120,000 population levels. The multiple bars on the graph indicate the results of iterative model runs at various influent pumping rates for the three population levels. For each model run, HGL results were reviewed to determine if surface flooding was present. Influent pumping was reduced until flooding was present or immanent. The lowest resulting rate for each population demand was selected as the recommended rate. As indicated, the existing design rate of 28 mgd is adequate to serve the flows up to a 60,000 population scenario. At 80,000 population, the rate increases to an estimated 50 mgd, and for 120,000, population a rate of 60 mgd was recommended.

Table 4-7
Replacement Pipe Sizing and Alternative Costs
North Ninth Improvements

							No Diversion		Diversion to Crescent Valley Interceptor @ Ponderosa		Diversion to Crescent Valley Interceptor @ Sequoia	
Pipe	Upstream Manhole	Length (ft)	Pipe slope (%)	Existing Dia. (in.)	Existing Capacity (mgd)	120,000 Peak Flow (mgd)	120,000 Replacement Diameter w/ System Improvements (in.)	Approximate Construction Cost (No Contingency or ELA)	120,000 Replacement Diameter (in.)	Approximate Construction Cost (No Contingency or ELA)	120,000 Replacement Dia. (in.)	Approximate Construction Cost (No Contingency or ELA)
403624	5283022	467.001	0.37	8	0.5	0.3	0	$ -	0	$ -	0	$ -
403623	5283021	463.105	0.35	8	0.5	0.5	10	$ 56,406	0	$ -	10	$ 56,406
403622	5283010	198.667	0.73	10	1.2	0.7	0	$ -	0	$ -	0	$ -
403621	5283017	174.312	0.08	10	0.4	0.9	15	$ 31,847	15	$ 31,847	15	$ 31,847
403471	5283018	372.01	0.22	10	0.7	1.1	15	$ 66,954	15	$ 66,954	15	$ 66,954
403458	5083013	378.113	0.27	10	0.7	1.3	15	$ 75,109	15	$ 75,109	15	$ 75,109
403451	5083034	174.416	0.18	12	1.0	1.4	15	$ 31,790	15	$ 31,790	15	$ 31,790
403450	5083032	43.1285	0.21	12	1.1	1.5	15	$ 7,856	15	$ 7,856	15	$ 7,856
403441	5083033	333.326	0.42	12	1.5	1.7	15	$ 60,899	0	$ -	15	$ 60,899
403439	5083060	148.804	0.19	12	1.0	1.8	15	$ 27,186	0	$ -	15	$ 27,186
403436	5083062	43.9402	0.46	12	1.6	1.8	15	$ 8,028	0	$ -	15	$ 8,028
403434	5083061	411.104	0.25	12	1.2	2.0	15	$ 75,109	0	$ -	15	$ 75,109
403418	5083070	366.472	0.17	12	0.9	2.2	18	$ 80,239	0	$ -	18	$ 80,239
402921	5083074	168.321	0.20	12	1.0	2.3	18	$ 36,854	0	$ -	18	$ 36,854
402919	4883032	239.824	0.17	12	1.0	2.5	18	$ 52,509	0	$ -	18	$ 52,509
402915	4883003	347.848	0.43	12	1.5	2.6	15	$ 63,552	0	$ -	15	$ 63,552
402914	4883006	392.529	0.12	21	3.6	4.8	24	$ 101,885	0	$ -	0	$ -
402881	4880039	528.174	0.11	21	3.4	5.0	27	$ 129,201	0	$ -	0	$ -
402863	4880055	489.722	0.08	21	2.9	5.3	27	$ 173,848	0	$ -	0	$ -
404089	4880062	404.071	0.18	21	4.4	5.4	24	$ 143,440	0	$ -	0	$ -
404561	4680056	560.212	0.14	21	3.9	5.7	27	$ 133,000	0	$ -	0	$ -
404589	4680057	392.529	0.17	21	4.2	5.9	24	$ 114,972	0	$ -	0	$ -
404088	4680028	168.321	0.27	21	5.4	6.1	24	$ 49,301	0	$ -	0	$ -
Subtotal: Pipe Improvements								**$ 1,519,985**		**$ 213,556**		**$ 674,338**
Diversion from Ponderosa to Conser		2800	0.50						18	$ 788,220		
Diversion from Sequoia to Circle		2150	0.50								18	$ 605,240
Totals								**$ 1,136,381**		**$ 1,001,776**		**$ 1,279,578**

ELA = Engineering, legal, and administrative.

Figure 4-8
WWRP Influent Pumping Flow Rate



Selected Collection System Configuration Improvements

Collection System Capital Improvement Project Summary

Through the review of the deficiency maps which identify where HGL criterion violations occurred, the comparison of pump station peak capacities to projected flows, and improvement alternatives and RDII reduction analyses, a listing of recommended projects was developed. The complete listing of system improvements required to meet the HGL criteria for the peak 5-year, 24-hour design storm assuming an 80,000 land use condition are as follows:

- Marys River pump station
- Brooklane pump station (based on 120,000 population demand)
- Marys River interceptor improvements (based on 120,000 population demand)
- South Corvallis interceptor and associated diversion pipes
- Herbert Pump Station High Flow Bypass
- Country Club interceptor improvements
- North Ninth pipe improvements and interim grinder pump station
- Garfield pipe improvements
- Crescent Valley interceptor improvements (based on 120,000 population demand)

- Influent pumping at the wastewater treatment plant to 45 mgd to serve 80,000 population demand and 60 mgd to serve 120,000.

The recommended improvements are shown in Figures 4-9, 4-10 (A-G), and 4-11 provided at the end of this section. The associated costs are listed in Table 4-8. Pump station peak flow capacity and demands are shown in Table 4-3. Diameters are based on a demand associated with the maximum buildout condition or the 120,000 probable buildout condition as explained previously, given the desired service life of a collection system pipeline (80 years). Pump station capacities are based on the flow produced for the 80,000 population.

TABLE 4-8
Cost Estimates for Conveyance Improvements

Project	Total Project Costs[a]		
	Existing	60,000	80,000
Marys River Pump Station	$ 3,320,000	$ 340,000	$ 510,000
Brooklane Pump Station	$ -	$ 3,320,000	$ 490,000
South Corvallis Parallel Interceptor & Diversion	$ -	$ -	$ 12,410,000
Herbert Pump Station High Flow Bypass	$ 30,000	$ -	$ -
Marys River Interceptor	$ -	$ -	$ 1,000,000
Country Club Trunk	$ -	$ -	$ 1,200,000
North Ninth Street Interim Grinder Pump Station	$ 40,000	$ -	$ -
North Ninth Street Diversion	$ -	$1,570,000[b]	$ -
Garfield Relief Sewers	$ 110,000	$ 90,000	$ 330,000
Crescent Valley Interceptor	$ -	$ -	$ 2,300,000
Total Project Cost	$ 3,500,000	$ 5,320,000	$ 18,240,000

[a] Total Project Cost includes 25% Contingency and 25% Engineering, Legal, Administration. ENR-CCI=6500.
[b] Postponement of diversion to potentially 60,000 population is dependent on the installation of an interim grinder pump station to serve a single existing business.

Based on the review of the deficiency maps for the various population levels, the required timing and phasing of the projects was established. The information presented below begins with the projects required to address existing deficiencies followed by those additional projects required for the 60,000 and 80,000 population demand levels. Also provided are the projects required to serve maximum buildout (180,000 population) within a sewer basin. When pipelines were sized, independent of what population level triggered their need, they were sized for maximum buildout. This decision was based on the estimated service life of a pipeline (80 years) and the potential for buildout within most City basins.

Collection System Capital Improvement Projects to Serve Existing Conditions

As the existing deficiencies map (Figure 4-3) indicates, the current pipeline deficiencies that do not result from shallow pipe conditions are upstream of the Marys River pump stations, along North Ninth Street and at the upstream end of the modeled system in the Garfield Basin on Walnut Boulevard. Increasing the capacity of the Marys River pump station (see Table 4-3) eliminates the identified deficiency pipeline improvements in South Corvallis, and the North Ninth Street diversion project addresses the other two deficiencies in that basin. The Ninth Street Diversion can be potentially postponed by the installation of an interim grinder pump station to serve a business currently experiencing infrequent surcharging. Because long-term service to South Corvallis is proposed using a gravity line, the Herbert Street station would be abandoned when the parallel interceptor is constructed. To extend the service life of the station, it is proposed that upstream pipelines be allowed to surcharge and a high flow bypass that discharges to the downstream gravity sewer be constructed. The projects are as follows:

- Marys River Pump Station
- Upper Garfield Pipeline
- North Ninth Street Interim Grinder Pump Station
- North Ninth St. Diversion (potentially can be postponed)
- Herbert Pump Station High Flow Bypass

Collection System Capital Improvement Projects to Serve 60,000

Through review of Figure 4-4 and Table 4-3 for pump stations, the additional needs to serve the 60,000 population demand are listed below. As indicated above, the North Ninth Street Diversion can potentially be postponed to about a 60,000 population by installing an interim grinder pump station to serve an existing single business. Regardless, the diversion should be planned as an early, priority 60,000 population project. As shown on Figure 4-4, there are additional deficiencies in the Garfield system, and, as indicated in Table 4-3, the peak flow is greater than the peak design capacity for the Brooklane pump station. Therefore, the project needs are:

- Pipe replacement in the Garfield basin
- Brooklane pump station (structure sizing to be based on 120,000 population demand)
- Installation of additional pumps at Marys River Pump Station

Collection System Capital Improvement Projects to Serve 80,000 Population

The system improvements in addition to those required for existing and 60,000 conditions and that are required to meet the HGL criteria for the peak 5-year, 24-hour design storm are as follows:

- Marys River interceptor improvements (sizing based on 120,000 population demand)
- South Corvallis interceptor and associated diversion pipes
- Country Club interceptor
- Garfield relief sewer improvements
- Crescent Valley interceptor improvements (sizing based on 120,000 population demand)
- Installation of additional pumps at Marys River pump station

- Installation of additional pumps at Brooklane pump station
- Influent pumping at the wastewater treatment plant to 45 mgd to serve 80,000 population demand and 60 mgd to serve 120,000

Collection System Capital Improvement Projects to Serve Maximum Buildout Population

To provide the flexibility in the plan and to provide information to support uncertain locations and rate of development, an analysis of each sewerage basin was performed for buildout conditions. The modeling approach used was to isolate each basin from the rest of the system and increase the flow values to those associated with the buildout populations and associated acreages. Significantly more system needs were identified for this demand level including most of the existing modeled lines in the Squaw Creek and Country Club basins. The Crescent Valley interceptor and Garfield basin also showed increases in system needs. The projects are shown on Figures 4-10 (A-G) and 4-11 (oversize figure in pocket at the end of this section). Project costs are summarized by basin in Table 4-9. Detailed project listings and costs are included in Appendix D. Projects associated with the probable buildout population of 120,000 are also included in Appendix D.

Projects to Serve Unserved Areas

System extensions in the following basins are necessary to serve areas currently not served:

- Squaw Creek (serves outer Fillmore)
- Jackson Creek
- Frazier Creek
- Northeast
- Lewisburg

The projects are identified in Table 4-10 and shown in Figures 4-10 (A-G) and 4-11.

TABLE 4-9
Conveyance Project Costs to Serve Maximum Buildout within Basin
(in addition to 80,000/120,000 projects)

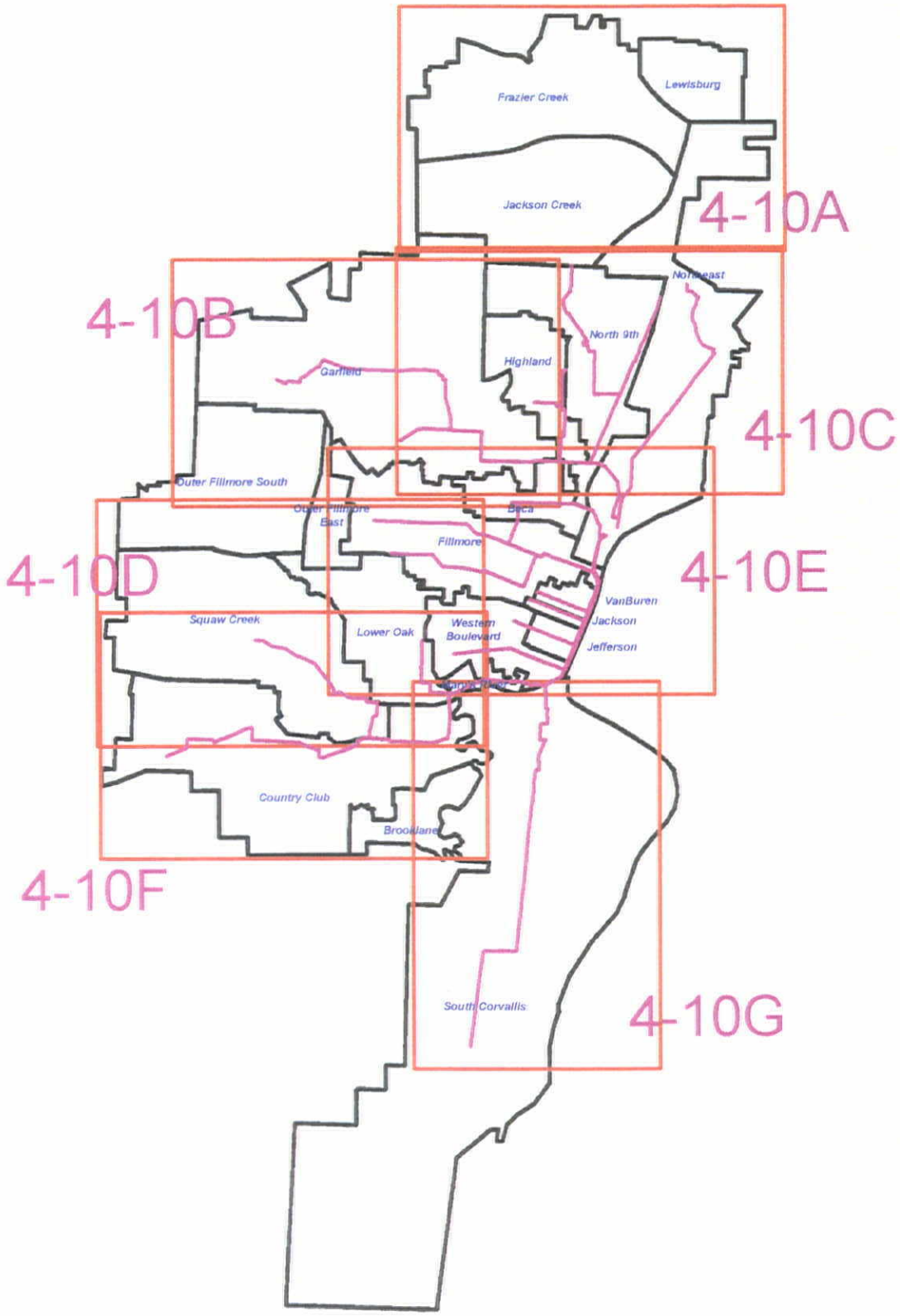
Subbasin	Description	Pipe Length (feet)	Cost*
Garfield	(in addition to Garfield Relief Sewers)		
Subtotal		3,562	$ 1,050,000
Total with 25% Engineering, Legal, Administration			$ 1,312,500
Highland	No Projects		
Subtotal			$ -
North Ninth	(in addition to North Ninth St. Diversion project)		
Subtotal		3,197	$ 1,200,000
Total with 25% Engineering, Legal, Administration			$ 1,500,000
Northeast		8,625	
Total with 25% Engineering, Legal, Administration			$ 8,546,875
Beca	No Projects		
Subtotal			$ -
Lower Oak Creek	No Projects		
Subtotal			$ -
Squaw Creek			
Subtotal		8,131	$ 2,325,000
Total with 25% Engineering, Legal, Administration			$ 2,906,250
Country Club	(in addition to Country Club Trunk)		
Subtotal		9,735	$ 3,350,000
Total with 25% Engineering, Legal, Administration			$ 4,187,500
South Corvallis	(in addition to Marys River PS, and Parallel Interceptor)		
Subtotal		2,200	$ 400,000
Total with 25% Engineering, Legal, Administration		-	$ 500,000
Marys River	No projects in addition to Marys River Interceptor and Brooklane PS		
Subtotal			$ -
Total Cost All Basins		-	**$ 18,953,125**

* Total Project Cost includes 25% Contingency and 25% Engineering, Legal, Administration.
ENR-CCI=6500.

TABLE 4-10
Pipeline Extension to Currently Unserved Areas

Subbasin	Pipe Diameter (inches)	Length (feet)	Slope (%)	Cost*
Outer Fillmore East	15	2,400	0.5	$468,750
Outer Fillmore South	15	5,500	0.5	$1,093,750
Northeast	30	3,860	0.5	$1,562,500
Lewisburg	18	4,010	0.5	$937,500
Frazier Creek	27	9,950	0.5	$3,593,750
Jackson Creek	27	10,170	0.5	$3,593,750
Total Project Cost				**$11,250,000**

*Project cost includes 25% contingency and 25% Engineering, Legal, Administration.
ENR-CCI = 6500.



0 2 4 Miles

Figure 4-9

1:1000 Detail Map Tiles
Location Key

City of Corvallis
Wastewater Utility Master Plan

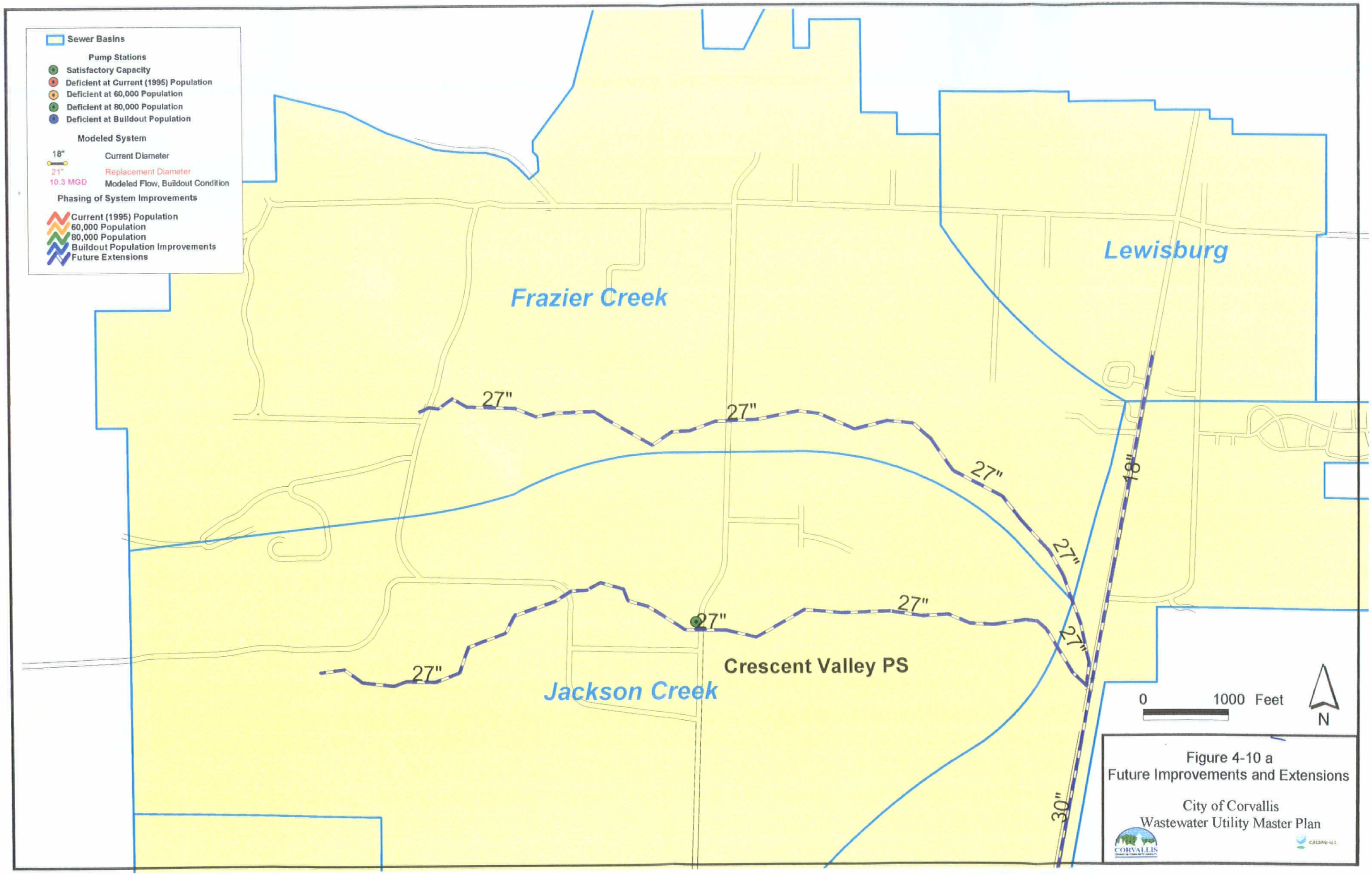


Figure 4-10 a
Future Improvements and Extensions

City of Corvallis
Wastewater Utility Master Plan

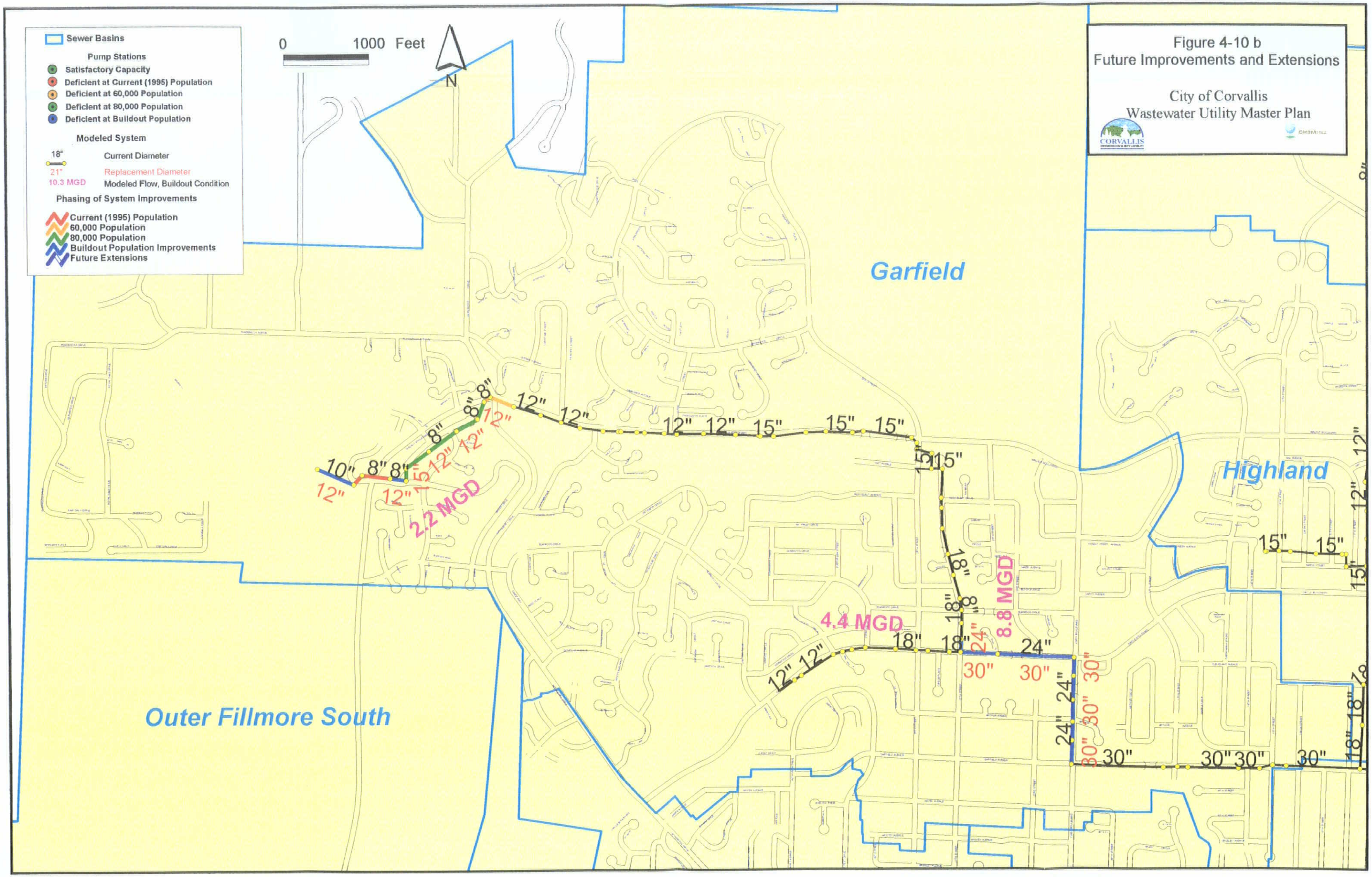


Figure 4-10 b
Future Improvements and Extensions
City of Corvallis
Wastewater Utility Master Plan
Sewer Basins
Pump Stations
Satisfactory Capacity
Deficient at Current (1995) Population
Deficient at 60,000 Population
Deficient at 80,000 Population
Deficient at Buildout Population
Modeled System
18" Current Diameter
21" Replacement Diameter
10.3 MGD Modeled Flow, Buildout Condition
Phasing of System Improvements
Current (1995) Population
60,000 Population
80,000 Population
Buildout Population Improvements
Future Extensions
0 1000 Feet
N
Garfield
Highland
Outer Fillmore South
2.2 MGD
4.4 MGD
8.8 MGD

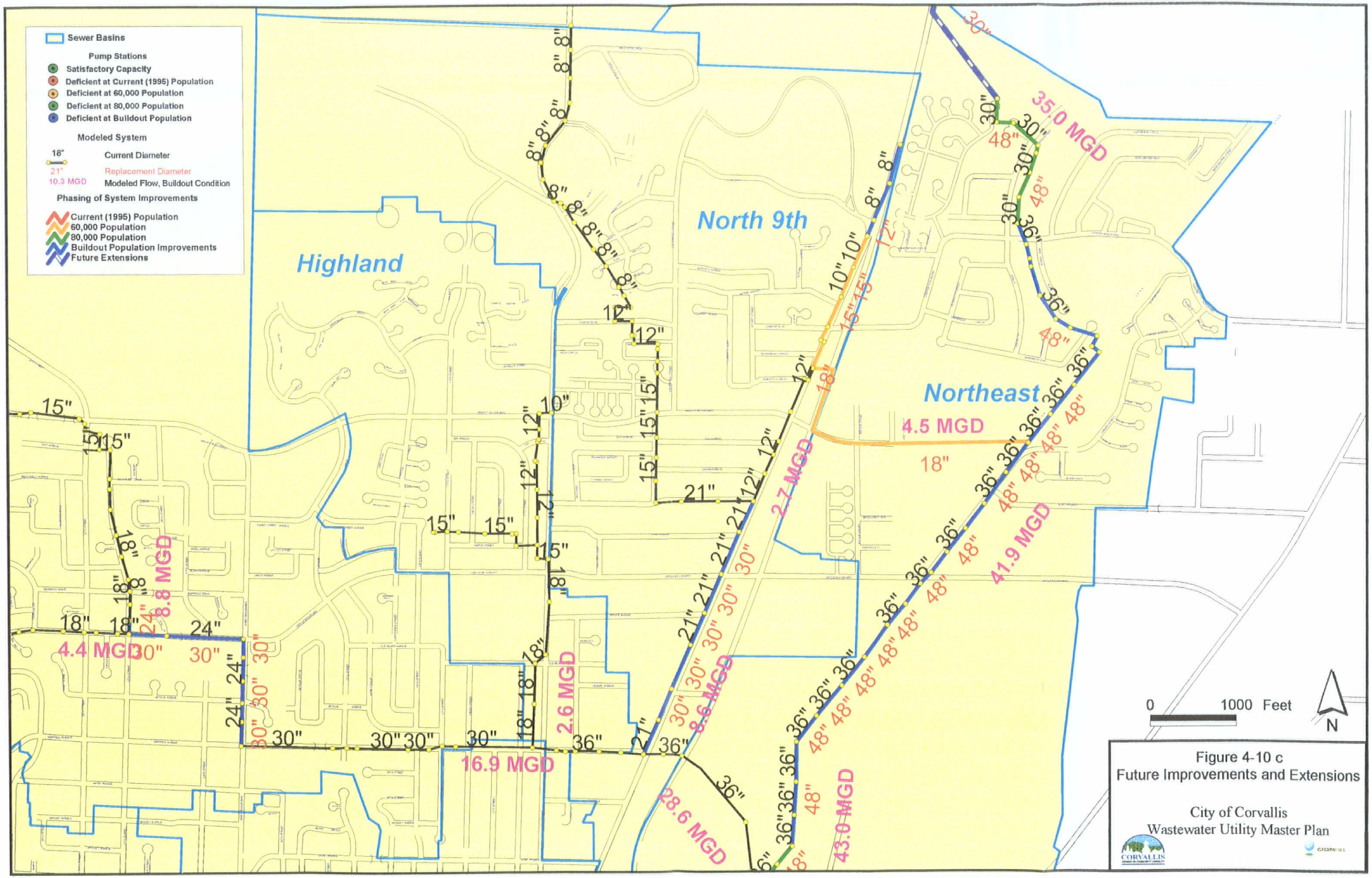


Figure 4-10 c
Future Improvements and Extensions

City of Corvallis
Wastewater Utility Master Plan

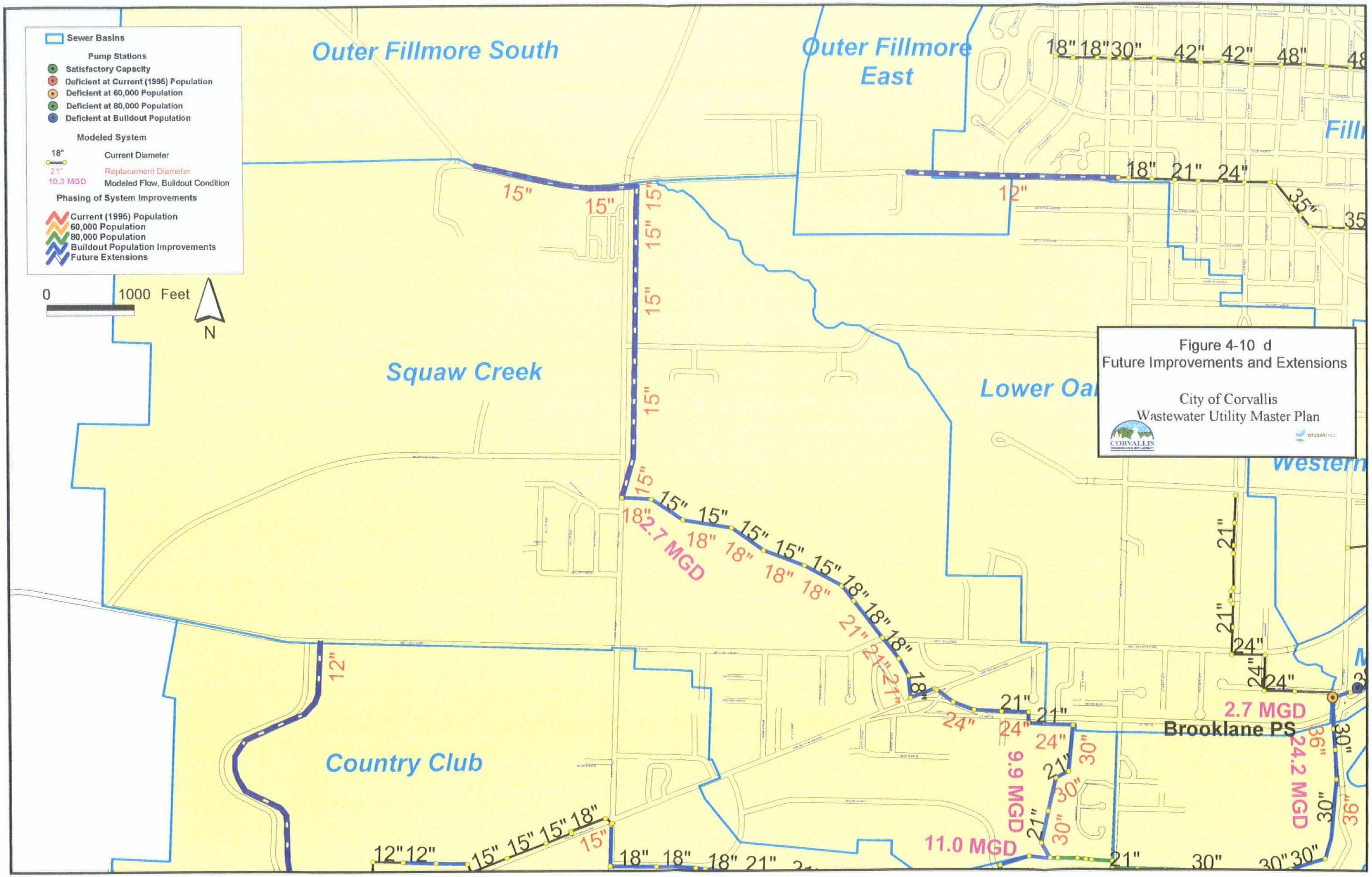


Figure 4-10 d
Future Improvements and Extensions

City of Corvallis
Wastewater Utility Master Plan

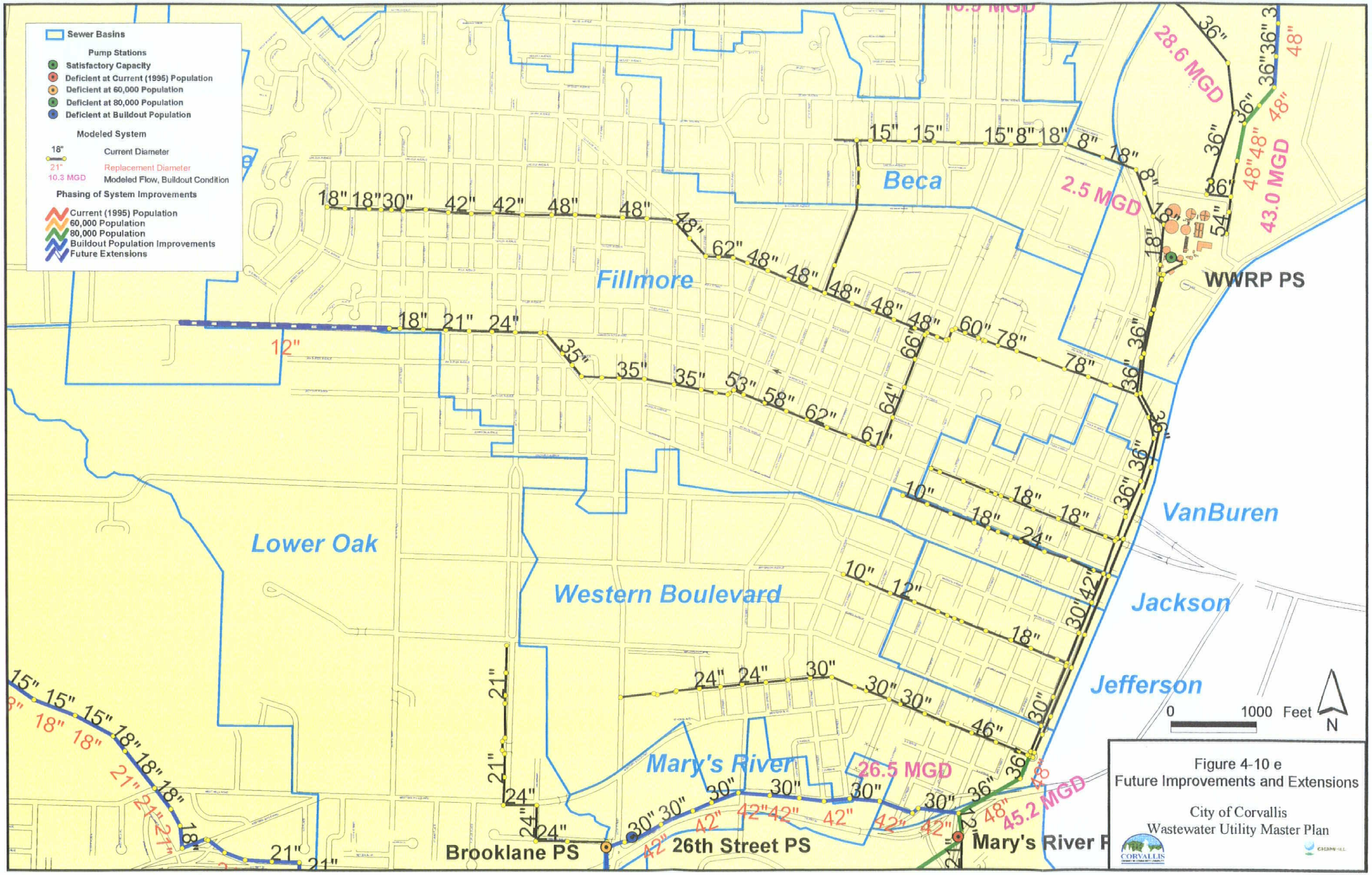


Figure 4-10 e
Future Improvements and Extensions

City of Corvallis
Wastewater Utility Master Plan

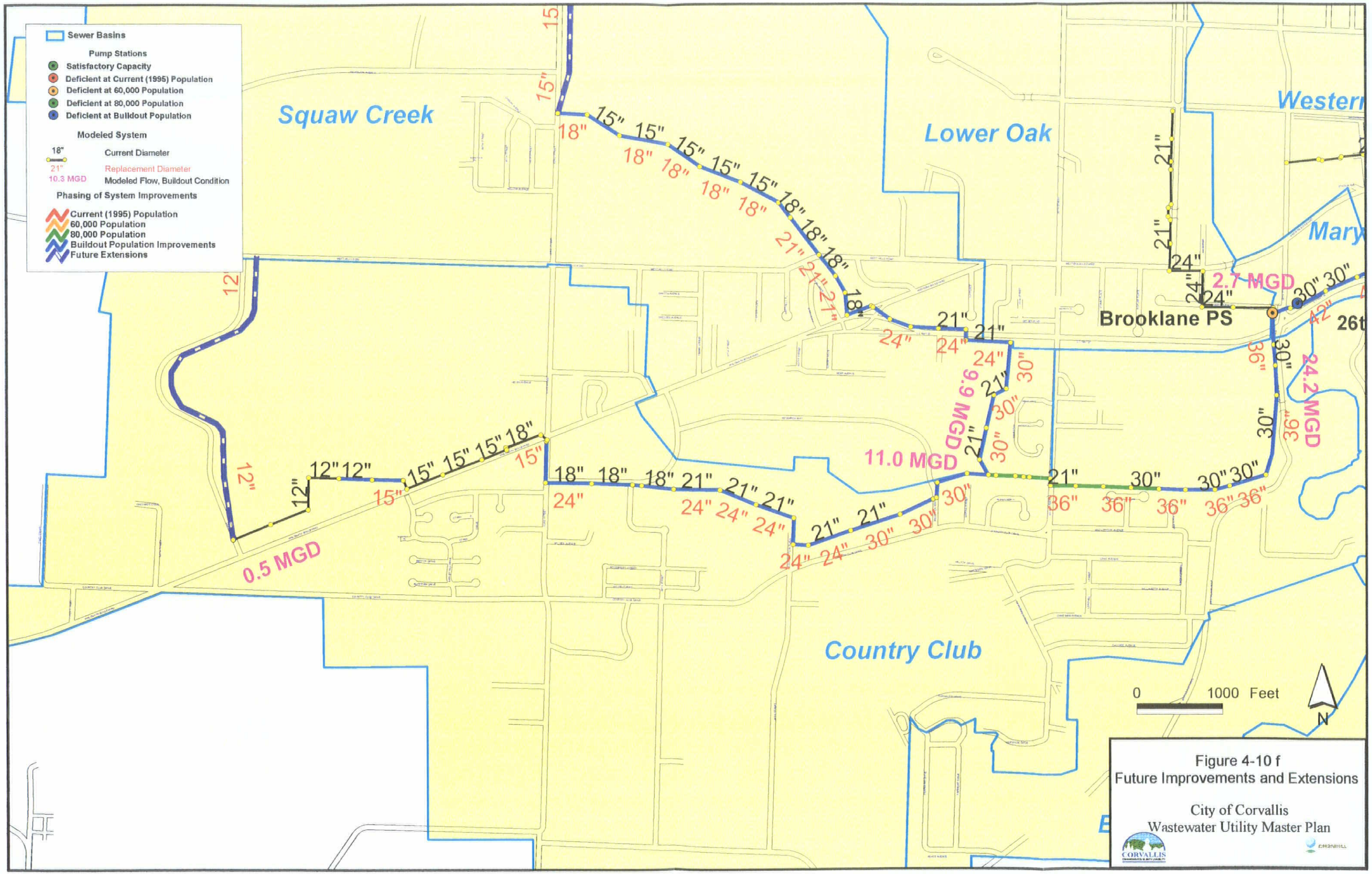


Figure 4-10 f
Future Improvements and Extensions

City of Corvallis
Wastewater Utility Master Plan

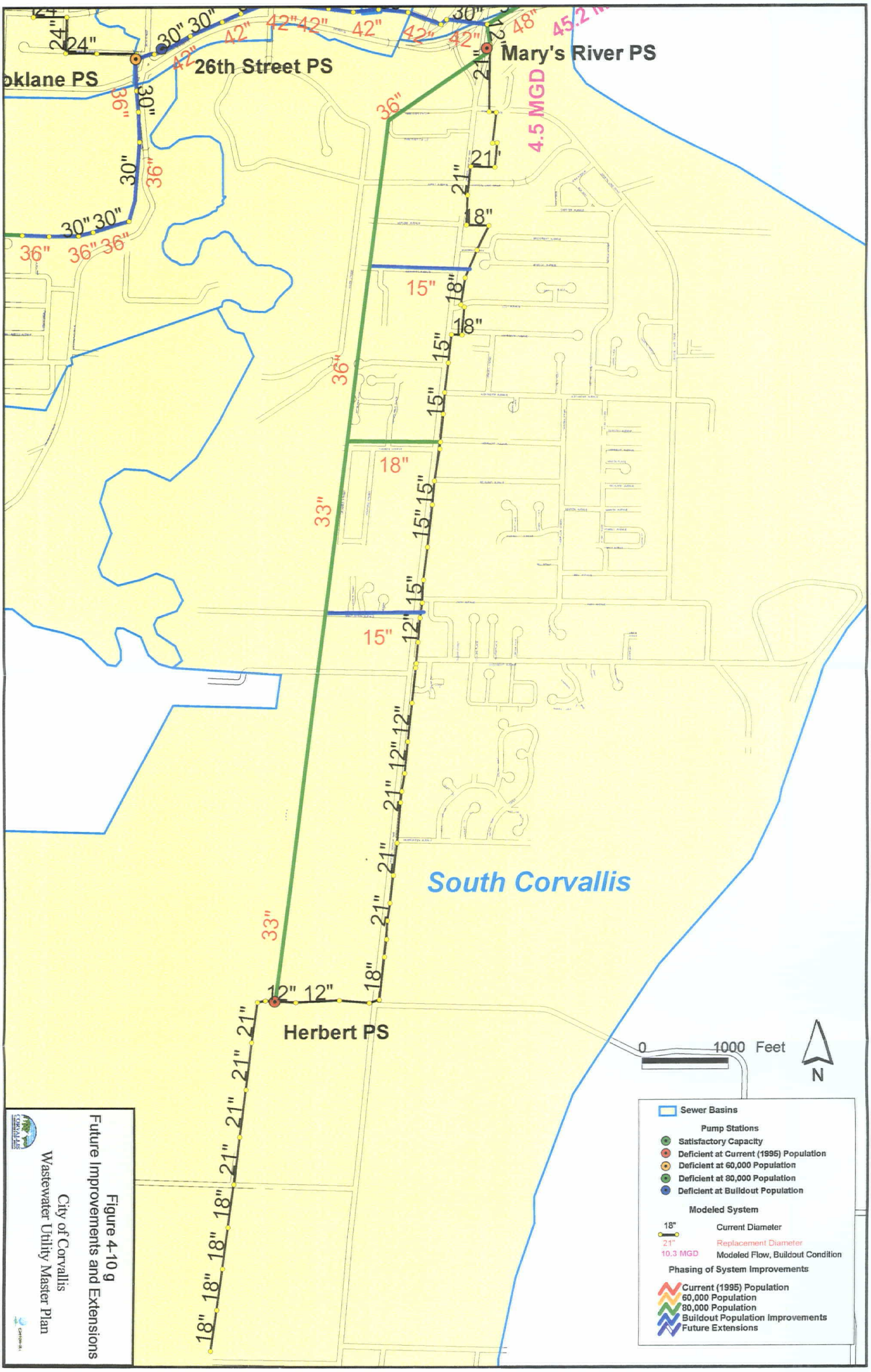
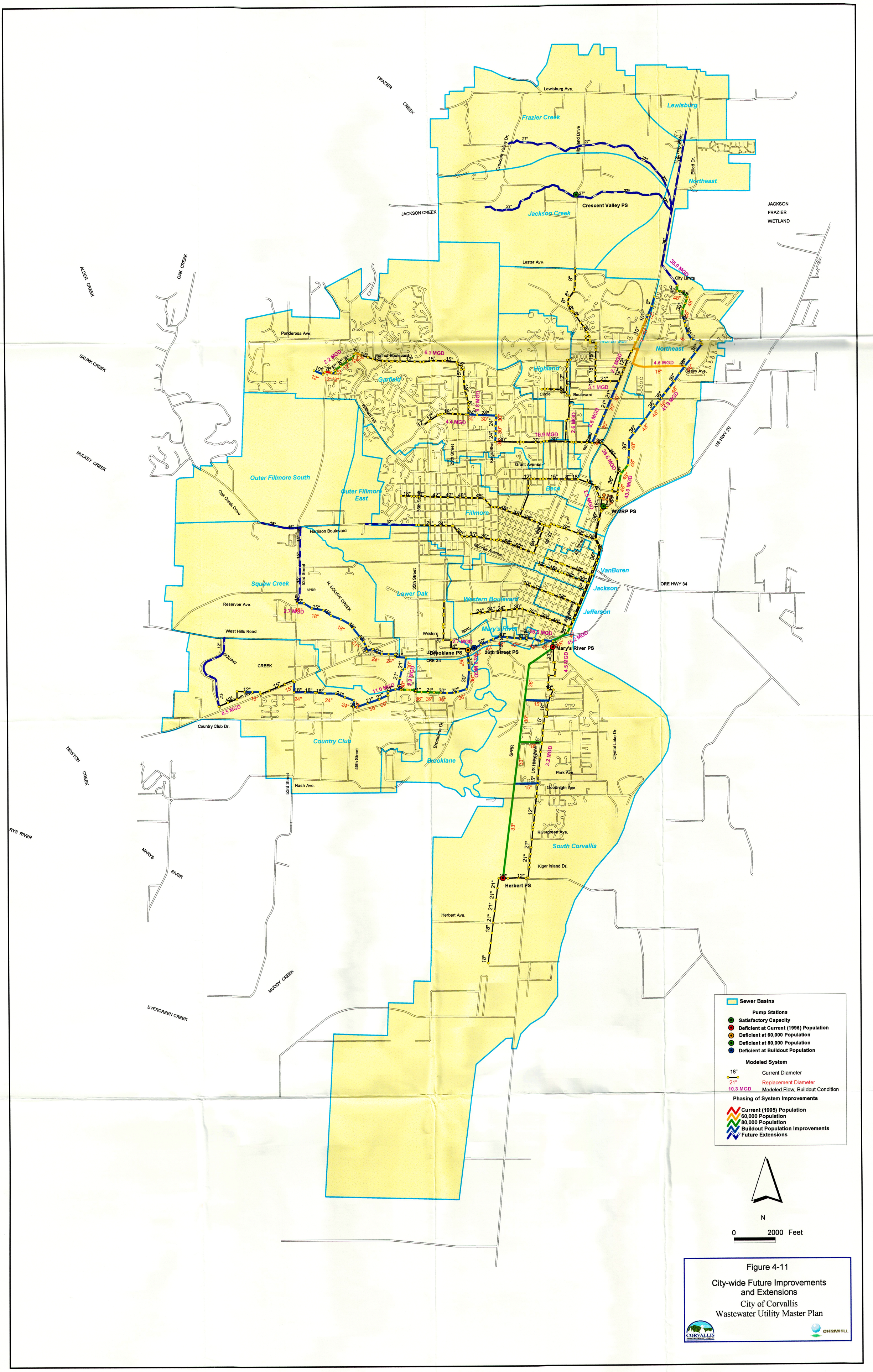


Figure 4-10 g
Future Improvements and Extensions
City of Corvallis
Wastewater Utility Master Plan


Sewer Basins
Pump Stations
Satisfactory Capacity
Deficient at Current (1995) Population
Deficient at 60,000 Population
Deficient at 80,000 Population
Deficient at Buildout Population
Modeled System
18" Current Diameter
21" Replacement Diameter
10.3 MGD Modeled Flow, Buildout Condition
Phasing of System Improvements
Current (1995) Population
60,000 Population
80,000 Population
Buildout Population Improvements
Future Extensions
N
0 2000 Feet
Crescent Valley PS
WWRP PS
Brooklane PS
26th Street PS
Mary's River PS
Herbert PS
Frazier Creek
Jackson Creek
Lewisburg
Northeast
Highland
Garfield
Outer Fillmore South
Outer Fillmore East
Fillmore
Beca
Squaw Creek
Lower Oak
Western Boulevard
Mary's River
VanBuren
Jackson
Jefferson
Country Club
Brooklane
South Corvallis
JACKSON FRAZIER WETLAND
FRAZIER CREEK
JACKSON CREEK
ALDER CREEK
OAK CREEK
SKUNK CREEK
MULKEY CREEK
NEWTON CREEK
MARYS RIVER
MUDDY CREEK
EVERGREEN CREEK
SQUAW CREEK
Lewisburg Ave.
Lester Ave.
Ponderosa Ave.
Walnut Boulevard
Harrison Boulevard
Monroe Avenue
Reservoir Ave.
West Hills Road
Country Club Dr.
Nash Ave.
Grant Avenue
Park Ave.
Goodnight Ave.
Rivergreen Ave.
Kiger Island Dr.
Herbert Ave.
Seavy Ave.
City Limits
Elliott Dr.
Crescent Valley Dr.
Highland Drive
Circle
Boulevard
Kings Blvd.
25th Street
35th Street
45th Street
53rd Street
Brooklane Dr.
Crystal Lake Dr.
US HWY 20
ORE HWY 34
SPRR
N. SQUAW CREEK
Figure 4-11
City-wide Future Improvements and Extensions
City of Corvallis
Wastewater Utility Master Plan
CORVALLIS
CH2MHILL

SECTION 5

WWRP Flow and Wasteload Analysis and Existing Capacity Analysis

This section presents the analysis of existing influent flows and wasteloads for the Corvallis Wastewater Reclamation Plant (WWRP), summarizes the future flow and wasteload projections, and provides the results of an analysis of existing system capacity.

Existing Flow and Wasteload Analysis

Daily plant influent data from January 1991 to December 1996 were analyzed to determine the historical flow and wasteload characterization for the WWRP. This analysis is summarized in Table 5-1. For each of the years analyzed, the actual population was used to establish per capita values for annual average flows and wasteloads. Wasteloads included chemical oxygen demand (COD), carbonaceous biochemical oxygen demand (CBOD), and total suspended solids (TSS). Peaking factors were calculated for other conditions based on these annual averages. The per capita and peaking factors for each of the individual years were then averaged for the entire analysis period.

Per capita values for influent flows increased from 170 gallons per day per capita (gpd/capita) to 252 gpd/capita during the analysis period. This trend is reflective of the increase in rainfall seen during this period. The per capita values for influent CBOD vary from 0.10 pound per day per capita (ppd/capita) to 0.20 ppd/capita with an average of 0.15 ppd/capita.

The data for influent TSS are influenced by two factors:

1. Waste activated sludge (WAS) flows, which are recycled back to the headworks to allow for co-thickening in the primary clarifiers, are included in the raw influent samples.
2. At high flow, wet weather conditions, these WAS flows tend to "wash out" of the primary clarifiers and create a constant recirculating stream.

As a result, analysis of the influent TSS data was "corrected" for the effects of this WAS recycle stream on the raw influent data by estimating the TSS contribution using WAS flow rates. The influent TSS per capita values that resulted from this analysis varied from 0.27 to 0.34 ppd/capita with an average of 0.31 ppd/capita.

Table 5-2 presents a comparison of the averaged per capita values and peaking factors with several other treatment plants in the Willamette Valley and with "textbook" values. As shown, the per capita values and peaking factors at the Corvallis WWRP are within the range of values observed at other treatment facilities with similar high infiltration-and-inflow (I/I) collection system characteristics. The significantly higher peaking factors for wet weather conditions and the high per capita TSS-to-CBOD ratio (0.31 to 0.15 ppd/capita) are all indicative of a combined collection system and/or high I/I.

TABLE 5-1
Historical Flows, Loadings, and Peaking Factors

Condition	Historical Flows (mgd) and Loads (lb/d)				Historical Peaking Factors			
	Flow	COD	CBOD	TSS	Flow	COD	CBOD	TSS
1991 (Population = 45,780)								
AA Per Capita (gpd/capita, lb/d/capita)	170	0.44	0.16	0.29				
AA	7.80	20,345	7,141	13,263	1.00	1.00	1.00	1.00
DWA	4.97	15,846	5,217	9,284	0.64	0.78	0.73	0.70
DWMM	10.40	26,876	9,337	17,205	1.33	1.32	1.31	1.30
DWPW	12.02	28,203	11,409	19,191	1.54	1.39	1.60	1.45
DWPD	20.18	36,004	14,611	24,997	2.59	1.77	2.05	1.88
DWPH	27.51				3.53			
WWA	10.67	25,358	9,293	17,465	1.37	1.25	1.30	1.32
WWMM	22.07	31,275	11,460	23,152	2.83	1.54	1.60	1.75
WWPW	23.48	40,204	13,581	27,205	3.01	1.98	1.90	2.05
WWPD	23.37	57,930	15,849	42,927	3.00	2.85	2.22	3.24
WWPH	31.00				3.97			
1992 (Population = 45,470)								
AA Per Capita (gpd/capita, lb/d/capita)	185	0.52	0.20	0.32				
AA	8.43	23,765	9,021	14,350	1.00	1.00	1.00	1.00
DWA	6.34	23,834	9,444	13,812	0.75	1.00	1.05	0.96
DWMM	11.18	32,260	12,565	21,949	1.33	1.36	1.39	1.53
DWPW	11.53	37,632	15,331	26,038	1.37	1.58	1.70	1.81
DWPD	13.89	60,278	15,855	76,009	1.65	2.54	1.76	5.30
DWPH	31.93				3.79			
WWA	10.55	23,699	8,615	14,860	1.25	1.00	0.95	1.04
WWMM	13.57	29,536	12,624	21,875	1.61	1.24	1.40	1.52
WWPW	20.19	37,162	15,294	24,974	2.40	1.56	1.70	1.74
WWPD	24.37	62,032	20,380	45,848	2.89	2.61	2.26	3.19
WWPH	34.26				4.06			
1993 (Population = 46,260)								
AA Per Capita (gpd/capita, lb/d/capita)	194	0.59	0.17	0.34				
AA	8.96	27,510	7,944	15,559	1.00	1.00	1.00	1.00
DWA	7.24	26,194	7,349	13,439	0.81	0.95	0.93	0.86
DWMM	13.81	33,758	9,288	21,845	1.54	1.23	1.17	1.40
DWPW	16.82	44,590	10,752	22,859	1.88	1.62	1.35	1.47
DWPD	23.15	44,590	13,591	24,875	2.58	1.62	1.71	1.60
DWPH	29.19				3.26			

TABLE 5-1
Historical Flows, Loadings, and Peaking Factors

	Historical Flows (mgd) and Loads (lb/d)				Historical Peaking Factors			
Condition	**Flow**	**COD**	**CBOD**	**TSS**	**Flow**	**COD**	**CBOD**	**TSS**
WWA	10.71	28,697	8,548	17,624	1.20	1.04	1.08	1.13
WWMM	14.43	31,559	11,111	21,589	1.61	1.15	1.40	1.39
WWPW	18.54	39,879	14,659	25,054	2.07	1.45	1.85	1.61
WWPD	24.88	51,278	20,836	33,263	2.78	1.86	2.62	2.14
WWPH	30.71				3.43			
1994 (Population = 46,195)								
AA Per Capita (gpd/capita, lb/d/capita)	191	0.62	0.15	0.32				
AA	8.81	28,820	7,064	14,754	1.00	1.00	1.00	1.00
DWA	5.98	23,996	6,561	11,243	0.68	0.83	0.93	0.76
DWMM	9.25	32,225	7,825	16,500	1.05	1.12	1.11	1.12
DWPW	11.33	32,785	11,734	20,738	1.29	1.14	1.66	1.41
DWPD	21.86	38,781	16,072	34,107	2.48	1.35	2.28	2.31
DWPH	29.92				3.40			
WWA	11.69	34,291	7,580	18,420	1.33	1.19	1.07	1.25
WWMM	15.68	43,700	9,556	24,712	1.78	1.52	1.35	1.67
WWPW	19.84	66,458	14,600	39,320	2.25	2.31	2.07	2.67
WWPD	24.61	104,889	18,513	54,207	2.79	3.64	2.62	3.67
WWPH	32.76				3.72			
1995 (Population = 47,485)								
AA Per Capita (gpd/capita, lb/d/capita)	185	0.61	0.12	0.32				
AA	10.81	29,146	5,564	15,171	1.00	1.00	1.00	1.00
DWA	7.35	29,394	5,815	14,554	0.68	1.01	1.05	0.96
DWMM	13.58	34,518	7,810	17,177	1.26	1.18	1.40	1.13
DWPW	14.68	37,478	9,636	21,902	1.36	1.29	1.73	1.44
DWPD	20.63	47,627	10,908	26,203	1.91	1.63	1.96	1.73
DWPH	32.31				2.99			
WWA	14.33	28,863	5,287	15,814	1.33	0.99	0.95	1.04
WWMM	19.29	41,930	8,500	21,664	1.78	1.44	1.53	1.43
WWPW	23.64	43,022	10,595	28,875	2.19	1.48	1.90	1.90
WWPD	24.94	63,875	13,120	57,372	2.31	2.19	2.36	3.78
WWPH	35.57				3.29			
1996 (Population = 49,275)								
AA Per Capita (gpd/capita, lb/d/capita)	252	0.51	0.10	0.27				
AA	12.42	25,305	4,788	13,192	1.00	1.00	1.00	1.00

TABLE 5-1
Historical Flows, Loadings, and Peaking Factors

	Historical Flows (mgd) and Loads (lb/d)				Historical Peaking Factors			
Condition	**Flow**	**COD**	**CBOD**	**TSS**	**Flow**	**COD**	**CBOD**	**TSS**
DWA	7.88	26,076	4,633	13,986	0.63	1.03	0.97	1.06
DWMM	11.51	41,800	7,210	26,910	0.93	1.65	1.51	2.04
DWPW	17.02	67,059	13,363	33,194	1.37	2.65	2.79	2.52
DWPD	21.31	95,066	15,697	53,831	1.72	3.76	3.28	4.08
DWPH	32.00				2.58			
WWA	17.00	24,499	4,964	12,364	1.37	0.97	1.04	0.94
WWMM	22.69	36,796	6,850	22,529	1.83	1.45	1.43	1.71
WWPW	26.07	49,476	8,320	31,682	2.10	1.96	1.74	2.40
WWPD	27.97	86,697	11,403	42,850	2.25	3.43	2.38	3.25
WWPH	32.00				2.58			
Actual Per Capita Values (for Current Population) and Peaking Factors (Average of 1991-96)								
AA Per Capita (gpd/capita, lb/d/capita)	203	0.55	0.15	0.31				
AA					1.00	1.00	1.00	1.00
DWA					0.70	0.93	0.94	0.88
DWMM					1.24	1.31	1.31	1.42
DWPW					1.47	1.61	1.81	1.68
DWPD					2.15	2.11	2.17	2.82
DWPH					3.26			
WWA					1.31	1.07	1.07	1.12
WWMM					1.91	1.39	1.45	1.58
WWPW					2.34	1.79	1.86	2.06
WWPD					2.67	2.76	2.41	3.21
WWPH					3.51			

Notes:
Calculations based on historical CBOD and TSS data "corrected" to reflect true influent characteristics by removing estimated WAS recycle contributions from the measured and reported values.

Legend:
A Average
AA Annual Average
DW Dry Weather
MM Maximum Month
PD Peak Day
PH Peak Hour
PW Peak Week
WW Wet Weather

Future Flow and Wasteload Projections

Typically, existing plant influent data are utilized to establish reasonable per capita and peaking factor values for calculating future influent flows and wasteloads to serve a projected population. At the Corvallis WWRP, this typical method of projecting future flow

Table 5-2
Comparison of Corvallis Wastewater Peaking Factors and Per Capita Flows and Wasteloads with Literature Values and Various Pacific Northwest Wastewater Treatment Plants

Parameter (peaking factor times annual average for parameter shown unless otherwise noted)	Corvallis[a]	Literature	Westside[b]	MPWRF[c]	Salmon Creek[d]	McMinnville[e]	Wilsonville[e,f]	Salem[e,g]	Tryon Creek[h]	Eugene/Springfield[e,i]	Tri-City[j]	Bremerton[k]	Average
Influent Flow													
Avg. Wet Weather	1.31	--	1.07	1.04	1.06	1.33	1.12		1.18	1.27	1.00	--	1.15
Max. Month	1.91	--	1.17	1.12	1.30	2.00	1.26	1.50	1.69	1.72	1.50	1.33	1.50
Max. Week	2.34	--	1.42	1.25	1.39	--		2.20	2.19	2.40	2.30	--	1.94
Max. Day	2.67	2.0 - 2.5	2.33	1.53	1.66	2.74	2.05	3.10	--	3.52	4.40	2.46	2.65
Peak Hour	3.51	3.0 - 4.0	3.22	2.91	2.74	3.69			3.35		5.80	3.88	3.64
Influent BOD													
Avg. Wet Weather	1.07			0.98	1.00		1.08		--	0.99	1.00		1.02
Max. Month	1.45	1.24	1.31	1.38	1.20		1.34	1.25	1.23	1.13	1.30	1.60	1.31
Max. Week	1.86		1.63	1.52	1.60			1.50	1.36	1.37	1.60	--	1.55
Max. Day	2.41	2.65	1.86	2.01	2.00			2.00	1.88	1.57	2.20	2.10	2.07
Influent TSS													
Avg. Wet Weather	1.12			0.96	1.10		1.04		--	1.03	1.00		1.04
Max. Month	1.58	1.29	1.40	1.46	1.20		1.10	1.25	1.31	1.18	1.30	2.00	1.37
Max. Week	2.06		1.84	1.56	1.50			1.40	1.39	1.47	2.50	--	1.72
Max. Day	3.21	2.91	2.39	1.86	2.20			1.80	2.58	3.12	3.50	--	2.62
Per Capita													
Flow per capita (gallons per day)	203	70-120	94	104	162		170	192		215	145		161
BOD per capita (pounds per day)	0.15	0.18-0.26	0.14	0.12	0.27		0.32	0.22		0.24	0.21		0.21
TSS per capita (pounds per day)	0.31	0.20-0.33	0.17	0.13	0.28		0.32	0.22		0.26	0.24		0.24

[a] Corvallis peaking factors are from Corvallis Master Plan Projections (1997 memo).
[b] Westside WWTP, Vancouver, Washington.
[c] Marine Park Water Reclamation Facility, Vancouver, Washington.
[d] Salmon Creek WWTP, Clark County, Washington.
[e] Oregon.
[f] Wilsonville peaking factors and per capita values are based on 2015 design numbers from April 1995 Facility Plan.
[g] Salem per capita values are based on 2027 design numbers from August 1996 Master Plan.
[h] A City of Portland WWTP located in Lake Oswego, Oregon.
[i] Eugene/Springfield per capita values are from Table 3-4 "Average Per Capita" Design Criteria values.
[j] Tri-City Water Pollution Control Plant, Clackamas County, Oregon, per capita values are based on new design numbers for 1997 updated facility plan. Peaking factors are relative to wet weather average.
[k] Washington.

and wasteloads is inappropriate because the current efforts to remediate CSOs will result in a different characterization of the influent flows and loads to the existing WWRP. Therefore, the methodology for establishing future flow and wasteload projections requires more analysis than would be required for typical growth-related projections. Because the CSO remediation plan will collect and treat CSO flows during storm conditions, average flow conditions will increase proportionally to growth in population. However, the peak conditions will be influenced by the performance of the wet weather treatment system and will be attenuated because much of the growth is in the separated portion of the collection system. Therefore, the analysis for establishing future flows consisted of two parts:

- All dry weather conditions and the averaged wet weather flow conditions (e.g., average day, maximum month, and peak week values) were determined using historical per capita and peaking factor values.
- Peak wet weather flow conditions (peak day and peak hour) were determined using the results of the collection system modeling discussed in Section 3.

In determining future wasteloads, the analysis considered that the influent flow characterization will change from that of a combined sewer system to one where most of the additional wasteload will have the characteristics of a typical separated sewer system. To accommodate this changing influent characterization, annual average wasteload projections used historical per capita values for CBOD and TSS up to the current population of 49,275, then used textbook per capita values of 0.20 ppd/capita to project the incremental increase in CBOD and TSS associated with growth over 49,275. Historical peaking factors were then applied to the resulting annual average projections to establish other wasteload conditions.

Table 5-3 shows the per capita values and peaking factors that were used in the flow and wasteload projections. These values were applied to projected populations of 60,000, 80,000, and a buildout condition of 120,000. The resulting future flow and wasteloads are summarized in Table 5-4.

TABLE 5-3
Design Per Capita Values and Peaking Factors

Condition	Flow	COD	CBOD	TSS
Annual Average Per Capita and Peaking Factors				
Per Capita (gpd/capita, lb/d/capita)	203	0.55	0.15	0.31
Peaking Factors	1.00	1.00	1.00	1.00
Dry Weather Peaking Factors				
DWA	0.70	0.93	0.94	0.88
DWMM	1.24	1.31	1.32	1.42
DWPW	1.47	1.61	1.81	1.68
DWPD	2.16	2.11	2.17	2.82
DWPH	3.26			
Wet Weather Peaking Factors				
WWA	1.31	1.07	1.07	1.12
WWMM	1.91	1.39	1.45	1.58
WWPW	2.34	1.79	1.86	2.06

TABLE 5-3
Design Per Capita Values and Peaking Factors

Condition	Flow	COD	CBOD	TSS
WWPD	See Note 3	2.76	2.41	3.21
WWPH	See Note 3			

Notes:
1. Maximum month (MM) and peak week (PW) numbers are based on running averages.
2. Calculations based on historical CBOD and TSS data "corrected" to reflect true influent characteristics by removing estimated WAS recycle contributions from the measured and reported values.
3. Calculations based on collection system modeling; therefore, peaking factors are not applicable.

Legend:
A Average
AA Annual Average
DW Dry Weather
MM Maximum Month
PD Peak Day
PH Peak Hour
PW Peak Week
WW Wet Weather

TABLE 5-4
Projection of Flows and Loadings at Future Population Conditions

Parameter/ Condition	Peaking Factor	Population 50,000	60,000	70,000	80,000	120,000 (Probable Buildout)	Remarks
Flow (mgd)							
AA	203	10.15	12.18	14.21	16.24	24.36	
DWA	0.70	7.11	8.53	9.95	11.37	17.05	
DWMM	1.24	12.59	15.10	17.62	20.14	30.21	
DWPW	1.47	14.92	17.90	20.89	23.87	35.81	
DWPD	2.16	21.92	26.31	30.69	35.08	52.62	
DWPH	3.26	33.09	28.00	46.32	50.00	60.00	
WWA	1.31	13.30	15.96	18.62	21.27	31.91	
WWMM	1.91	19.39	23.26	27.14	31.02	46.53	Clarifier overflow sizing
WWPW	2.34	23.75	28.00	33.25	38.00	57.00	
WWPD	2.67	27.10	28.00	37.94	50.00	60.00	Based on SWMM model
WWPH	3.51	35.63	28.00	49.88	50.00	64.00	Based on SWMM model pump station sizing
CBOD (lb/d)							
AA	0.15*	7,536	9,536	11,536	13,536	21,536	
DWA	0.94	7,084	8,964	10,844	12,724	20,244	
DWMM	1.32	9,948	12,588	15,228	17,868	28,428	
DWPW	1.81	13,641	17,261	20,881	24,501	38,981	Biological treatment sizing
DWPD	2.17	16,354	20,694	25,034	29,374	46,734	
WWA	1.07	8,064	10,204	12,344	14,484	23,044	
WWMM	1.45	10,928	13,828	16,728	19,628	31,228	Solids production calcs
WWPW	1.86	14,017	17,737	21,457	25,177	40,057	
WWPD	2.41	18,162	22,982	27,802	32,622	51,902	

TABLE 5-4
Projection of Flows and Loadings at Future Population Conditions

Parameter/ Condition	Peaking Factor	Population 50,000	60,000	70,000	80,000	120,000 (Probable Buildout)	Remarks
TSS (lb/d)							
AA	0.31*	15,420	17,420	19,420	21,420	29,420	
DWA	0.88	13,570	15,330	17,090	18,850	25,890	
DWMM	1.42	21,897	24,737	27,577	30,417	41,777	
DWPW	1.68	25,906	29,266	32,626	35,986	49,426	
DWPD	2.82	43,485	49,125	54,765	60,405	82,965	
WWA	1.12	17,271	19,511	21,751	23,991	32,951	
WWMM	1.58	24,364	27,524	30,684	33,844	46,484	Solids production calcs
WWPW	2.06	31,766	35,886	40,006	44,126	60,606	
WWPD	3.21	49,499	55,919	62,339	68,759	94,439	

* Quantities are historical per capita loads (lb/d/capita). For future projections, historical per capita loads were used with current population, while typical "textbook" values of 0.2 lb CBOD and TSS/d/capita were used with incremental population.

Legend: Controlling Conditions for Master Planning
A Average
AA Annual Average
DW Dry Weather
MM Maximum Month
PD Peak Day
PH Peak Hour
PW Peak Week
WW Wet Weather

Solids Production and Solids Handling System Design Criteria

As indicated above, existing plant influent wasteload data (TSS and CBOD) were used to establish reasonable per capita and peaking factor values for projecting future influent wasteloads. Typically, the influent wasteload projections produced by this method would be input into a process model of the anticipated treatment plant to establish the design criteria for the solids handling system. For the Corvallis WWRP, however, it is necessary to modify this methodology to account for the way the CSO remediation program will alter wasteload characteristics. Several thousand pounds of solids that used to be directed to the river during wet weather conditions will now be captured by the new wet weather treatment facility and conveyed to the existing WWRP for treatment. Therefore, it was necessary to do the following:

- Develop a method for characterizing the solids production from the new wet weather treatment facility
- Establish the impact these new loads will have on the existing WWRP solids handling system
- Project the solids production for various future conditions

Standard Solids Handling System Design Condition

Solids handling and treatment systems that utilize digestion processes and incorporate intermediate storage facilities (such as gravity thickening) are generally designed based on the maximum month loading conditions. In the case of Corvallis, as well as nearly all Willamette Valley treatment plants, this maximum month condition will occur during the wet weather season. Therefore, establishing the wet weather maximum month (WWMM) solids production is the key to determining the design criteria for the solids handling system.

Characterization of Solids Production from the Wet Weather Treatment Facility

In developing flow and wasteload design criteria, it is desirable to use as much site-specific data as possible. However, in the case of the Corvallis WWRP, available data regarding the quantity and character of CSO flows were insufficient. Therefore, solids production was evaluated using two different approaches, which were as follows:

- **Approach 1**: Calculate WWMM wasteload based on the historical DWMM baseload at the WWRP, modeled estimates of stormwater I/I during wet weather conditions, and typical stormwater solids concentrations.
- **Approach 2**: Calculate WWMM wasteload based on historical WWMM solids production data at the WWRP, modeled estimates of CSO flows during wet weather conditions, and existing CSO solids-concentration sampling data.

To determine influent TSS loadings to the wet weather treatment plant in either approach, the quantity of additional flow must be determined as well as the corresponding concentration of TSS in the flow stream. To establish the flow quantities, the results from the SWMM collection system model were used. For each approach, it was assumed that the maximum month condition was defined by back-to-back 5-year storm events. Since each of these events defines a 15-day period, two 5-year storms were used to establish the total flow over a 30-day (month-long) period.

The TSS concentrations were determined from representative sampling data. In the case of Approach 1, median stormwater quality data were used to represent the contribution being made by I/I. These data were taken from the values developed in the National Urban Run-off Program (NURP), conducted by EPA, and then compared with similar data developed by the Association of Clean Water Agencies (ACWA) in its stormwater monitoring program. The values are categorized by land use and tend to be highly variable. Therefore, using median values and looking at concentrations for various land uses provides a good representation of the anticipated concentrations and allows for a sensitivity analysis of the impact of concentration variability on the overall solids production rates.

For Approach 2, sampling data taken during the development of the Corvallis CSO remediation plan were used to establish the maximum month TSS concentrations for the influent to the wet weather treatment facility. Once again, these data were quite variable, so intermediate concentrations were used in the analysis.

Approach 1: DWMM Solids Plus Stormwater Solids Contribution

From the collection system modeling data, it was estimated the total stormwater (I/I) volume for the 5-year storm event will be 250 million gallons (MG). Assuming back-to-back events, this equates to 500 MG of stormwater during a WWMM condition.

The median TSS concentrations based on land use that were obtained from the NURP data are shown in Table 5-5.

TABLE 5-5
Median TSS Concentrations Based on Land Use

Land Use	TSS Concentration (mg/L)
Residential	101
Commercial	69
Mixed Residential/Industrial/Commercial	67

In general, these values, when compared with the corresponding land use, were slightly higher than similar data compiled by ACWA. Applying these TSS concentrations to the total stormwater quantity, produced the range of influent TSS loadings shown in Table 5-6.

TABLE 5-6
Increased Influent Solids Loadings Caused by WWMM I/I

Land Use	TSS Concentration (mg/L)	Monthly Solids Loading (lb)	WWMM Daily Solids Loading (lb/d)
Residential	101	421,170	14,040
Mixed	67	279,390	9,313

Next, the DWMM solids production values for both existing and future conditions were established. Using plant data for existing conditions and the results of the process modeling for future conditions, the DWMM solids production values shown in Table 5-7 were calculated.

TABLE 5-7
Existing and Projected Future DWMM Solids Production Quantities

	DWMM Solids Production (lb/d)		
Sludge Constituent	Existing	80,000 Population	Probable Buildout (120,000 population)
Primary Sludge	7,420	19,800	27,200
WAS	6,760	11,600	19,400
Total	14,180	31,400	46,600

A TSS removal efficiency of 50 percent was applied to the daily solids loading rates developed for the stormwater portion of the new CSO flows. This estimated the increased solids production associated with these new influent loadings. By adding this production to the DWMM values, the projected WWMM solids production shown in Table 5-8 was obtained.

TABLE 5-8
Approach 1 WWMM Solids Production Projections

I/I TSS Concentration (mg/L)	WWMM Solids Production (lb/d)		
	Existing	80,000 Population	Probable Buildout (120,000 population)
101	21,200	38,420	53,620
67	18,836	36,056	51,256

Approach 2: WWMM Solids Plus CSO Solids Contribution

For Approach 2, the total volume of CSO flows that will be directed to the wet weather treatment facility was determined. From the collection system modeling data, it was estimated the total CSO volume for a 5-year storm event will be 119 MG. Assuming back-to-back 5-year storms, a total CSO volume of 238 MG during a WWMM condition was estimated.

The CSO sampling data compiled during the CSO remediation plan development showed a wide range of TSS concentrations: from 44 to 118 mg/L. It was assumed that a worst-case condition (from the solids production perspective) would occur during an early season storm that would scour the sewers of any accumulated solids. Therefore, selecting a TSS concentration toward the upper end of the sampled range would be the most indicative of a design condition. Using 100 mg/L as the TSS concentration, the values shown in Table 5-9 were calculated.

TABLE 5-9
Influent Solids Loadings to the Wet Weather Treatment Facility

TSS Concentration (mg/L)	Monthly Solids Loading (lb)	WWMM Daily Solids Loading (lb/d)
100	198,492	6,616

Next, the WWRP's WWMM solids production was determined. Using plant data for existing conditions and the results of the process modeling for future conditions, the WWMM solids production shown in Table 5-10 were calculated.

TABLE 5-10
Existing and Projected Future WWMM Solids Production Quantities

Sludge Constituent	WWMM Solids Production (lb/d)		
	Existing	80,000 Population	Probable Buildout (120,000 population)
Primary Sludge	13,426	22,000	30,200
WAS	4,658	13,000	20,200
Total	18,084	35,000	50,400

A TSS removal efficiency of 50 percent was applied to the daily solids production rates for the new wet weather treatment facility. This estimated the increased solids production associated with these new influent loadings. By adding this production to the current WWMM values, the projected WWMM solids production shown in Table 5-11 were obtained.

TABLE 5-11
Approach 2 WWMM Solids Production Projections

	Existing	80,000 Population	Probable Buildout (120,000 population)
Total TSS (lb/day)	21,390	38,310	53,710

Results

The results of this analysis indicate very close correlation between the two approaches, particularly when an I/I TSS concentration of 101 mg/L is used for Approach 1. However, review of the results generated a concern that the assumed TSS concentrations may have an effect on the overall solids design criteria. Therefore, a sensitivity analysis was conducted to look at the impact of these assumptions.

Table 5-12 provides a matrix that summarizes the impact of varying the assumptions for CSO TSS concentrations on both the existing primary digester and future digester capacity requirements. Approach 1 was selected to calculate the WWMM solids production, because existing plant data for dry weather conditions are much more reliable than the data available for wet weather conditions that were used in Approach 2. In addition, a projection of the WWRP solids production at startup of the CSO facilities (year 2000) was used for the Loading Condition "with CSO Loads" in Table 5-12. This is not intended to make the projections more conservative, but reflects the loading condition that ultimately must be addressed.

To assess the impacts of this solids production on the existing digester, the resulting mean cell residence time (MCRT) was computed for a variety of conditions. These values should be compared with a typically recommended 16-day MCRT standard as a means of evaluating the impacts on the digestion system capacity.

The impacts to future digester sizing are shown for two sizing increments: populations of 60,000 and 80,000. For each loading condition, the larger number is the total capacity requirement, in gallons, whereas the parenthetical number is the capacity required in addition to the existing 795,000-gallon primary digester. Note that for the "existing" loading condition in Table 5-12, the future digester capacity requirements shown do not have a practical value because they do not account for any quantity of CSO solids. This information has been presented however for comparative purposes.

The solids production rates shown in Table 5-13 are recommended for evaluation and planning of the solids handling facilities.

TABLE 5-12
Impacts of CSO Solids Loadings on Current and Future WWRP Digestion System

Loading Condition	CSO TSS Concentration (mg/L)	WWMM Solids Production[a] (lb/d)	MCRT of Existing Digester[b] (days)	Digester Capacity Needed for 60,000 Population [c,d] (gallons)	Digester Capacity Needed for 80,000 Population [c,e] (gallons)
Existing	Not applicable	14,180	21.0	900,000 (105,000)	1,492,000 (697,000)
With CSO Loads	100	21,000	14.2	1,137,000 (342,000)	1,620,000 (825,000)
	60	18,350	16.2	1,047,000 (252,000)	1,576,000 (781,000)
	40	16,960	17.6	998,000 (203,000)	1,549,000 (754,000)

[a] Indicates projected solids production in year 2000 under various CSO influent loading conditions.
[b] MCRT (mean cell residence time) is calculated assuming a 4.5 percent feed concentration. Minimum MCRT is considered to be 16 days based on EPA PSRP stabilization criteria.
[c] Values in parentheses indicate additional volume above and beyond existing 795,000 gallon digester.
[d] Population corresponds to 2015 planning year.
[e] Population corresponds to 2030 planning year.

TABLE 5-13
Recommended Wet Weather Maximum Month Solids Production Design Criteria

	Existing	60,000 Population	80,000 Population
Total TSS (lb/day)	18,000	25,000	37,000

Condition Assessment and Capacity Analysis of Existing Treatment Facilities

The Corvallis WWRP is a 9.7-mgd trickling filter/activated sludge (TF/AS) plant with anaerobic digestion for sludge stabilization. A new wet weather treatment facility is currently under construction on the existing plant site to treat CSO flows. Figure 5-1 shows a site layout of the existing WWRP and the new wet weather treatment facilities.

Liquid Treatment Process

System Overview

Figure 5-2 shows a flow diagram for the existing treatment processes. The plant currently operates in two separate liquid treatment modes: one for winter conditions and one for summer conditions. During the winter, the two trickling filters are operated in parallel followed by aerobic treatment in the aeration basins (generally using only one of the basins). In this mode, the plant is non-nitrifying and is setup for carboneous CBOD removal only.

In the summer operating mode, the two trickling filters are operated in series allowing for near-complete nitrification at current flows. Flows are then directed to an aeration basin where the basin is configured to provide an anoxic zone in the first half of the basin and an aerobic zone in the second half of the basin. This allows the plant to get approximately

50 percent denitrification at current conditions. The advantage to this summer operating mode is that the process is extremely stable with very little operational change required. It also improves the performance of the secondary clarifiers by reducing pin floc problems that can occur using other operating modes.

Capacity Analysis

Table 5-14 summarizes the capacity analysis for the existing liquid treatment facilities. This table shows the number and size of units, the estimated existing capacity with all units in service and one unit out of service, and the current loadings to the facility. The analysis has been developed to incorporate the impact of CSO improvements to existing facilities.

From this analysis, it was determined that the primary areas of concern are those related to hydraulic capacity. The influent pump station, the primary and secondary clarifiers, and the chlorine contact basin are all operating at or above their current capacities.

There is still considerable biological treatment capacity remaining in the existing plant.

Solids Handling System

System Overview

Figure 5-2 shows a process flow schematic of the existing solids handling system. This system consists of gravity thickening of both primary and waste activated sludge in the rectangular primary clarifier, anaerobic digestion for sludge stabilization, storage of the digested biosolids in facultative sludge lagoons, and liquid land application of the biosolids on cooperative farmlands.

Capacity Analysis of Existing Solids Handling System

A summary of the capacity analysis for the existing solids handling system is presented in Table 5-15. The analysis must also consider the impact to the solids handling system when the new wet weather treatment facility is brought online in year 2000.

Gravity Thickening

Currently, primary sludge is gravity thickened in the primary clarifiers, and WAS is either co-thickened in the primary clarifiers (dry weather) or gravity thickened in the existing secondary digesters (wet weather) by using the decant mode for these tanks. This approach could continue for the WWRP, but concern exists about utilizing the new wet weather primary clarifiers to thicken primary sludge. The high hydraulic loading rates and the variability of the incoming solids concentrations could result in times when low-solids primary sludge will be pumped, resulting in high hydraulic loading of the digester. In addition, the high hydraulic loading during wet weather conditions makes reliable thickening of primary sludge in the primary clarifiers less attainable as flows continue to increase.

Evaluation of the solids loading rate (SLR) to the secondary digesters that would be created if both primary sludge and WAS were co-thickened indicates SLRs of 9 and 15 pounds per day per square foot (lb/d/ft^2) for existing and 80,000 population levels, respectively. These

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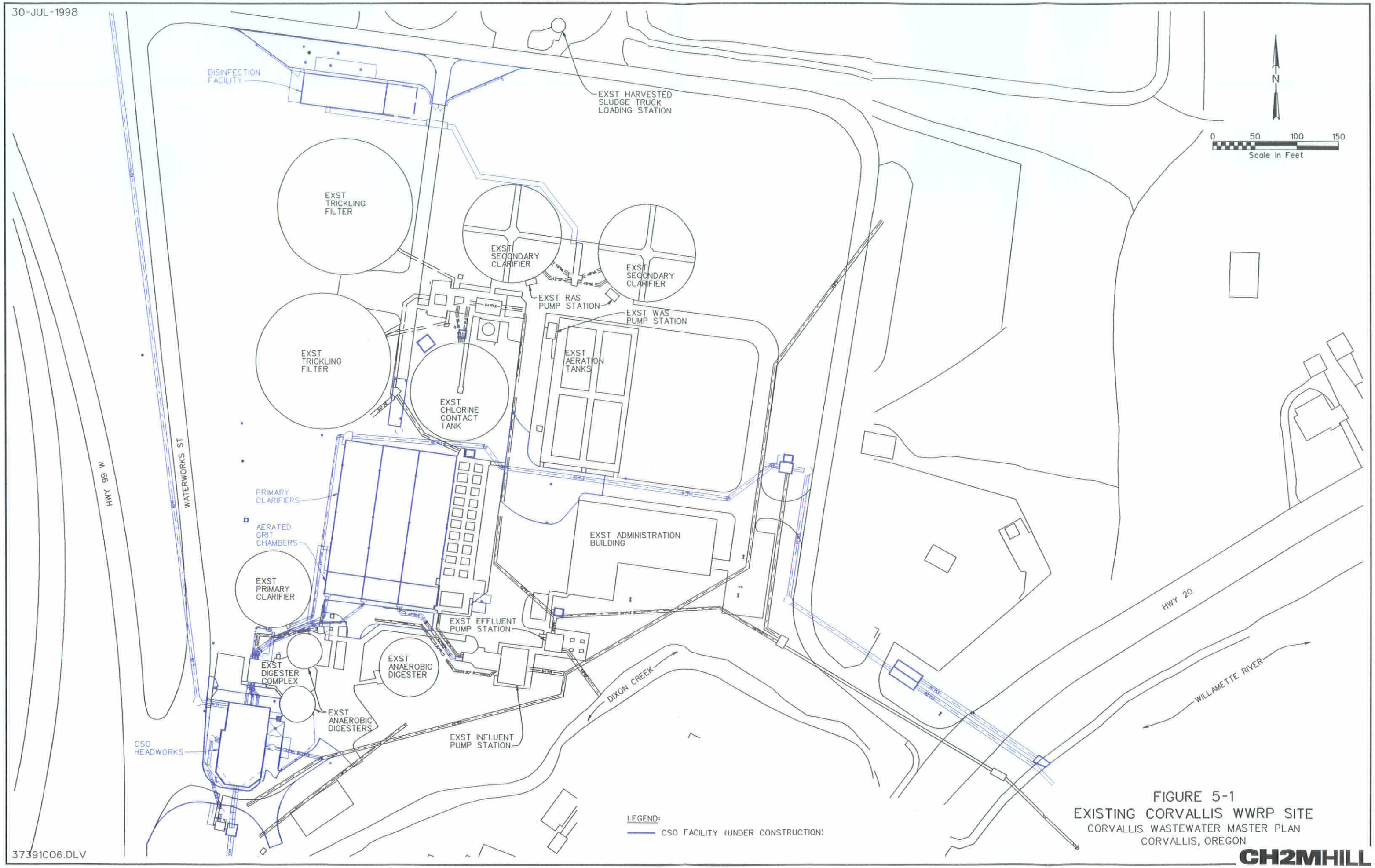

FIGURE 5-1
EXISTING CORVALLIS WWRP SITE
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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TABLE 5-14
Capacity Analysis of Existing Liquids Treatment Facilities

Component	Number and Size	Estimated Current Capacity, All Units	Estimated Current Capacity, Redundant Unit Out	Current Loadings (including CSO for Digestion)	Capacity Units
Influent Pumping					
Influent Pumps	One 7 mgd; three 14 mgd	49	35	35.3	mgd (WWPH)
Grit Removal					
Aerated Grit Tanks	One 8,460 ft^3	11.92	5.06	13.3	mgd (WWPD)
Grit Pumps	Two 180 gpm Two 250 gpm	860	430	0.18	gpm (WWPD)
Primary Sedimentation					
Rectangular tanks	One 136 ft x 31 ft x 9.5 feet, 4,216 ft^2 area One 136 ft x 42 ft x 9.5 ft, 5,712 ft^2 area	11.92	5.06	22.69	mgd (WWMM)
Primary Sludge Pumps	Two 200 gpm	400	200	20	gpm (WWPD)
Primary Scum Pumps	One 100 gpm	100	0	3.7	gpm (WWPD)
Trickling Filters					
Trickling Filters	Two 160 ft diameter 8 ft deep	18,343	9,172	13,500	lb CBOD/d (WWMM)
Circulation Pumps	Two 11.5 mgd	23	11.5	35.2	mgd (WWPH)
Aeration Tanks					
Aeration Tanks	Two 156 ft x 40 ft x 15 ft	33.6	16.8	15.5	mgd (WWMM)
Aeration Blowers	Four 5,900 cfm	23,600	17,700	9,333	cfm
Secondary Clarifiers					
Secondary Clarifiers	Two 115 ft diameter 18 ft side water depth	28.0	14.0	15.5	mgd (WWMM)
Return Activated Sludge (RAS) Pumping	Four 2,000 gpm	8,000	6,000	5,382	gpm (WWMM)
Waste Activated Sludge (WAS) Pumping	One 100 gpm; one 160 gpm	260	100	133	gpm (WWMM)
Disinfection					
Chlorine Contact Tank	One 110 ft diameter, 10.5 ft side water depth	17.9	0	35.2	mgd (WWPH)
Chlorinators	Two 2,000 lb/d	32.0	16.0	35.2	mgd (WWPH)

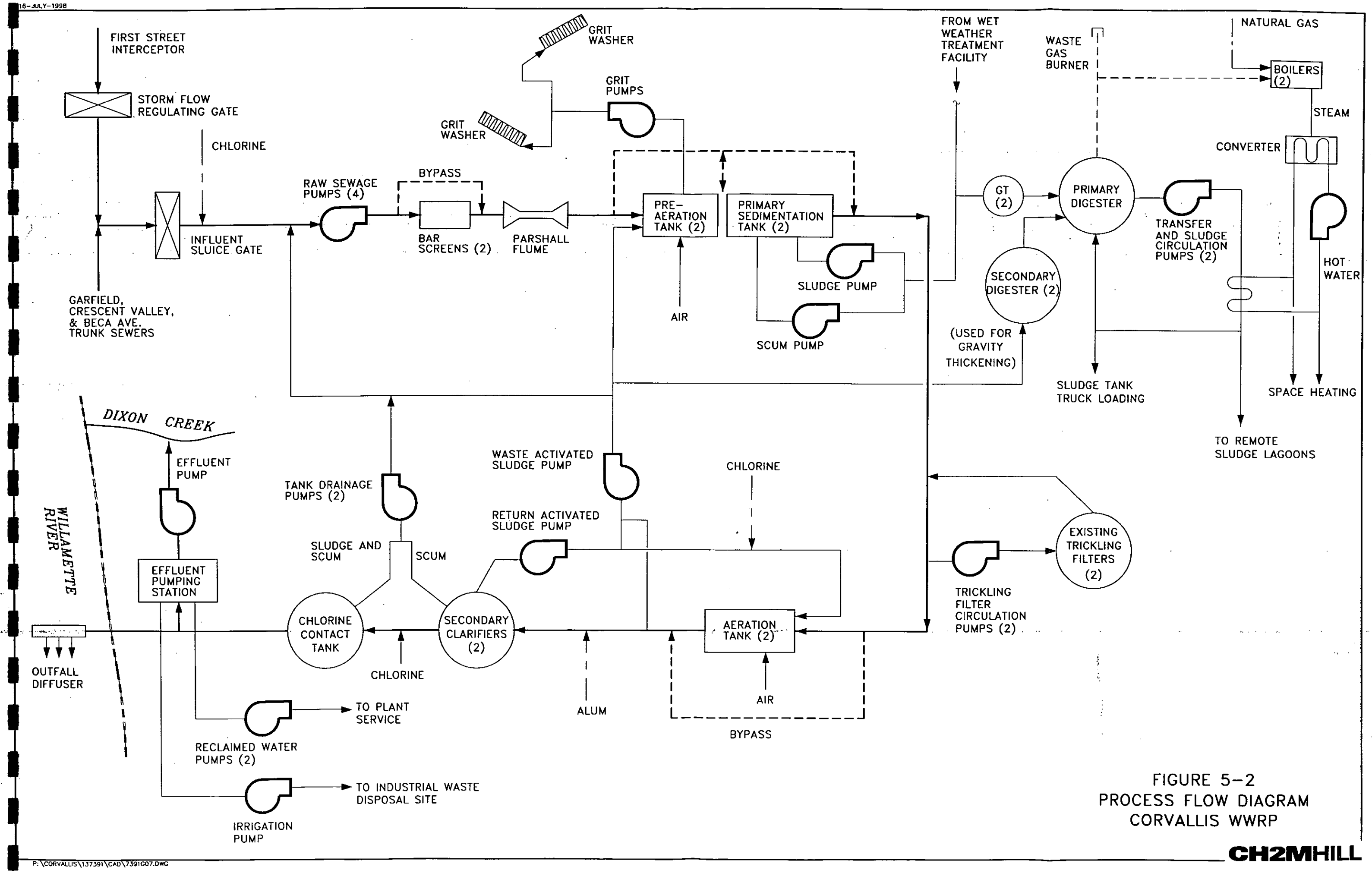

FIGURE 5-2
PROCESS FLOW DIAGRAM
CORVALLIS WWRP

TABLE 5-15
Capacity Analysis of Existing Solids Processing Facilities

Component	Number and Size	Estimated Current Capacity, All Units	Estimated Current Capacity, Redundant Unit Out of Service	Current Loadings (including CSO for Digestion)	Capacity Units
Sludge Digestion					
Anaerobic Digester	One 66 ft diameter, 31 ft side water depth	0.038	0	0.048	mgd (PSL+WAS WWPW)
Facultative Sludge Lagoon	Two 2.2 acres each	25	12.5	14.8	lb VSS/ 1,000 sq. ft./ day
Land Application Program					
Land		3000	3000	360	acres
Sludge storage Tanks	One 50,000 gal	50,000	0	50,000	gallons
Sludge Pumps	One 750 gpm at 50 ft TDH, 50 hp	750	0	750	gpm

values are well below the maximum design SLR of 25 lb/d/ft^2. However, reliable performance could not be assured without converting these tanks to gravity thickeners.

Anaerobic Digestion

In an initial assessment of the anaerobic digestion system, the following was found:

- Existing system is providing adequate volatile solids destruction
- Existing mixing system performance results in considerable solids deposition in the digester.
- There is cracking visible on the outside of the digester roof that could suggest internal structural problems.
- The system provides no redundancy for mechanical failures, maintenance, or cleaning.
- Heating system does not maintain digester temperature at feed rates above approximately 25,000 gallons per day and winter design conditions.
- Winter volatile solids loadings, and associated methane gas production, are not adequate to meet the winter heating demand, requiring the use of supplemental natural gas.

An MCRT of 16 days is required for an anaerobic digestion process that is operated at 35°C. Therefore, in evaluating the impacts of CSO solids to the digestion system, the detention time that will result with the new solids production values was examined. Assuming an average influent solids concentration of 4.5 percent, the existing 795,000-gallon anaerobic digester will have an MCRT of between 14.2 and 17.6 days at the projected WWMM solids

production. Therefore, the existing digester has marginal, if even adequate, capacity to process the increased solids quantities.

The minimum MCRT presented in this analysis is based on optimum operating conditions in the digester. This means a well-heated, well-mixed digester with the capabilities for a consistent sludge feed. As the existing digester nears capacity and the need to feed sludge from the wet weather treatment system is incorporated, control of digester temperature will become even more problematic without some facility upgrades. Also, the current grit buildup suggests that the existing mixing system is less than optimal. The result will eventually cause a decrease in digester solids stabilization performance. The potential down-side risks of decreased digester performance are the following:

- Regulatory compliance problems.
- Odor generation at the facultative sludge lagoons and/or substantial chlorine usage (100 pounds per day) to attempt to control odors.
- Odor generation at the cooperative land application sites which could jeopardize the use of some sites.
- Increases in the quantity of solids to be hauled and disposed of as a result of the reduction in volatile solids destruction.

In addition, operational considerations are making the use of the existing secondary digesters as gravity thickeners more important. This not only reduces the availability of online anaerobic digester capacity, it eliminates the available redundancy for maintenance activities on the primary digester. The 30-year-old primary digester is potentially in need of structural repairs and has historically required cleaning on 5- to 7-year intervals. The last digester cleaning cycle resulted in significant odor generation at the digester complex, several months to bring the digester back into stable operation, and increased odor generation at the facultative sludge lagoon.

Facultative Sludge Lagoons

Evaluation of the facultative sludge lagoons (FSLs) resulted in the following findings:

- FSL capacity is highly stressed with current biosolids loading (which includes Albany biosolids). One FSL is at 180 percent of design solids capacity.
- Dredge pumping is the primary limitation/constraint in the biosolids withdrawal and disposal operations.
- After the addition of Albany sludge to the FSLs, ammonia odors became a source of complaints. The addition of a chlorine solution feed has eliminated this problem.

Albany biosolids were no longer accepted at the Corvallis WWRP after 1997. Therefore, much of the stress to the facultative lagoon capacity should be eliminated. Given the substantial quantity of sludge that was being imported from Albany (ranged from 350 to 500 dry tons per year) relative to the increased solids production caused by CSOs, there is no anticipated near-term impact to the facultative sludge lagoons.

Land Application Program

Evaluation of the land application program for the disposal of biosolids revealed the following:

- Permitted acreage for land application is more than adequate to meet current requirements. In addition, the primary large application sites are located within a short haul distance (< 10 miles) from the plant.
- Currently, plant staff has very good relations with the farming community.
- A new lease agreement for the City's existing dedicated land application will provide a small, but flexible alternative for disposal at other than cooperative sites.

Recommended Near-Term Improvements

As a result of these condition assessments and capacity analyses several near-term improvements have been identified to improve system performance and address capacity shortfalls. These improvements should be addressed in the next 1 to 5 years. The improvements have been broken down into liquids and solids treatment systems and are discussed below.

Liquids Treatment Process

Table 5-16 shows a summary of the recommended near-term improvements for the liquid treatment facilities. These improvements are grouped into two categories: base sanitary flow improvements and peak hydraulic flow improvements. Each improvement is given a priority as follows:

- High—needs to be addressed to eliminate potential for equipment or process failure
- Medium—will result in reduced operating cost and/or improvements to plant regulatory compliance
- Low—would be beneficial to plant operation but not essential

Solids Handling Processes

The portion of the solids handling system that will be the most significantly stressed by the increased solids loadings resulting from the new wet weather treatment facility is anaerobic digestion. As can be seen above, projected WWMM solids loadings will significantly impact the digestion process ability to achieve adequate stabilization. This impact will be further compounded if the feed solids concentration cannot be maintained at 4.5 percent or higher. Therefore, several near-term improvements are recommended to match the capacity and reliability of the solids handling system with the liquids side of the plant. These improvements are:

- Convert the existing secondary digesters to gravity thickeners by adding mechanisms, weirs and launders, and providing a domed aluminum cover for odor control.
- Construct an additional 800,000-gallon anaerobic digester and control building with associated heating, pumping, and gas handling systems.

TABLE 5-16
Recommended Near-Term Improvements to the WWRP

Facility	Near-Term Need/Improvement	Alternative	Improvement Benefit	Priority	Cost	Remarks
Base Sanitary Flow Improvements						
Influent Pump Station	Improved wet well ventilation	Increase ventilation rate to min. of 12 a.c./hr.; raise duct exhaust; provide multiple outlets; inc. ventilation rate	Corrosion control, prolonged facility life	H	$5,000	
	Improved pump suction isolation	Provide aluminum volute bolt on flanges	Easier maintenance; increased safety	H	$1,000	
Grit Removal	Replace 3100 aerated grit diffusers	Coarse bubble, fixed Sanitaire or equal	Improved performance	M	$16,000	
	Improve grit chamber baffling/flow control	Longitudinal flow baffles inducing rolling pattern	Improved performance	L	$10,000	
Primary Clarifiers	Improve inlet well and effluent baffling in 6700	New baffles	Improved performance	H	$50,000	Implemented in CSO WWTF Project
	Structurally recondition concrete	Sandblast, spot patch, grout, and epoxy coat	Prolonged facility life	H	$9,000	Implemented in CSO WWTF Project
Trickling Filters	Remove effluent end baffle wall	Saw cut baffle off	Improved performance	L	$20,000	
Trickling Filters Recycle Pump Station	Add slide gates for positive flow and snail control; elevated gate operating platform matching existing	Replace directional vanes with slide gates	Reduced maintenance costs; increased capacity; improved operational flexibility	M	$160,000	
Aeration Basins	Provide aeration and anoxic zone modifications to allow improved anoxic/aerobic performance of both basins	Fine bubble diffuser system in aeration basin 2, anoxic mixer, and mixed liquor recycle pump in both aeration basins	Improved secondary clarifier performance, better effluent pH control	M	$306,000	
WAS Pump Station	Increase/improve WAS pumping/metering	Add parallel 3-inch magmeter and isolation valves	Improved performance/process control	M	$20,000	
Chlorine Contact Basin	Add compound loop control	New control, residual analyzer	Reduced chemical costs/improved compliance	H	$10,000	Implemented in CSO WWTF project

TABLE 5-16
Recommended Near-Term Improvements to the WWRP

Facility	Near-Term Need/Improvement	Alternative	Improvement Benefit	Priority	Cost	Remarks
Effluent Pump Station	Add second effluent pump	Second VFD, pump, valving	Provides redundancy	M	$150,000	
Anaerobic Digester	Add heat exchanger to 3rd. draft tube	Replicate existing heat transfer draft tubes	Increased reliability	L	$50,000	
	Repair digester roof	Pressure grout cracks (possibly coal tar epoxy interior)	Prolonged life	M	$50,000	
Plant Water System	Expand WR system capacity	Expand CSO washwater system/wetwell	Increased capacity	L	$50,000	
Administration Building	Repair roofing system	Refurbish build up roofing	Prolonged life	L	$25,000	
	Replace HVAC system (part of lab expansion into chlorine area)	Replace/refurbish as part of lab expansion and bldg. reconfiguration	Improved occupancy conditions	M	$50,000	
Subtotal					$745,000	
WWRP Peak Flow Improvement (28 mgd)						
Influent Pump Station	Improved hydraulic control	Hydraulic scale model to I.D. cause of problem/improvements necessary	Improved storm and non-storm performance	M	$75,000	
Primary Clarifiers	Construct 2nd. rectangular clarifier (4 total)	New aerated grit/clarifier matching existing unit	Improved performance/capacity	M	$1,250,000	Partially addressed
RAS Pump Station	Increase RAS capacity	Replace existing 2000 gpm pumps with same type but 3000 gpm capacity and higher head	Improved performance	L	$75,000	
	Provide new RAS VFD/controls	Replace magcoupling w/ VFD and magmeters	Lower maintenance costs; energy savings	L	$50,000	
Subtotal					$1,450,000	
Subtotal (all categories)					$2,501,000	
Contingency at 40 Percent					$1,000,000	
Total					$3,501,000	

- Provide digester feed pumping to allow thickened sludge to be transferred from the gravity thickeners to either the existing digester or the new digester.
- Provide odor control for the gravity thickeners to allow year-round co-thickening of primary sludge and WAS.

Figure 5-3 shows the proposed solids handling process flow diagram that would result from these improvements.

Gravity Thickeners

The proposed conversion of the secondary digesters to gravity thickeners would consist of the following:

- Add a rotating gravity thickener mechanism with sludge scraper and pickets. A column-supported mechanism would be recommended because it tends to provide a more rigid installation. Scum removal provisions are an optional feature.
- Add launders, weirs, and baffles to the perimeter of the tank for supernatant overflow.
- Remove the existing floating covers and replace them with aluminum, dome-type covers.

Existing sludge withdrawal piping could be used for the converted application.

Anaerobic Digester

Based on the projected future solids production for the 80,000 population level, 1.6 million gallons of anaerobic digester capacity will be required to maintain a minimum MCRT of 16 days. It is recommended that this capacity be provided in two separate digesters to improve redundancy and reliability. Therefore, each digester would be sized to provide 800,000 gallons of capacity.

For near-term improvements, it is recommended that one of these 0.80 million gallons digesters be added along with a control building to house the ancillary equipment. Because of the difficulty and overall higher cost of accommodating a future building expansion, it is recommended that the control building be sized to accommodate the equipment for two digesters with expansion provisions for at least one more digester. The new complex would be located adjacent to the facultative sludge lagoons.

Equipment would be provided for only one digester at this time. Future digester equipment would be provided when that digester is constructed. This equipment would include:

- Mechanical draft-tube mixing system with hot water heat exchangers for digester heating
- Gas collection and handling system
- Hot water boiler and heating hot water pumping system
- Digested sludge pumps
- Digester gas flare

Digester Feed Pumping

Since the new digester complex will be located remote from the existing digester and gravity thickeners, consideration must be given to feeding the new digester. As can be seen from the proposed solids handling process flow diagram, it is recommended that all sludge flow be routed through the gravity thickeners to a point in the system where digester feed solids concentration can be controlled and maintained. This also provides a convenient point from which to control digester feed distribution and pumping rates. To accommodate this, a new set of digester feed pumps must be provided. These pumps would be required to deliver sludge over a wide range of head conditions while maintaining a relatively consistent pumping rate. This is best accomplished with some type of positive displacement pump. Given the application and sludge types, the following pumps would be recommended for further evaluation:

- Progressing cavity pumps
- Air-operated diaphragm pumps
- Piston pumps
- Membrane pumps

Odor Control Provisions

With gravity co-thickening of primary and biological sludges, it is usually desirable to have odorous air collection and treatment of the headspace over the gravity thickeners. This is particularly true during the dry, warmer months when hydrogen sulfide formation is at a maximum. Alternatives for providing odor control include:

- Biofiltration
- Carbon adsorption scrubbing
- Wet chemical scrubbing
- Liquid-phase treatment using iron salts (e.g., ferric chloride, ferrous sulfate)

In these applications, carbon adsorption has tended to be the most often used approach; however, detailed alternative evaluations can be conducted as part of a preliminary design effort if deemed appropriate.

Solids Handling System Cost Estimates

The cost estimates are summarized in Table 5-17.

TABLE 5-17
Solids Handling System Improvements Cost Estimates

Facility	Estimated Cost
Gravity Thickener Modifications	$780,000
New Anaerobic Digester	$3,320,000
Total	**$4,100,000**

Alternative selection and additional design detail will impact these estimates. However, these order-of-magnitude costs should allow the impacts to project budgets and funding issues to be assessed.

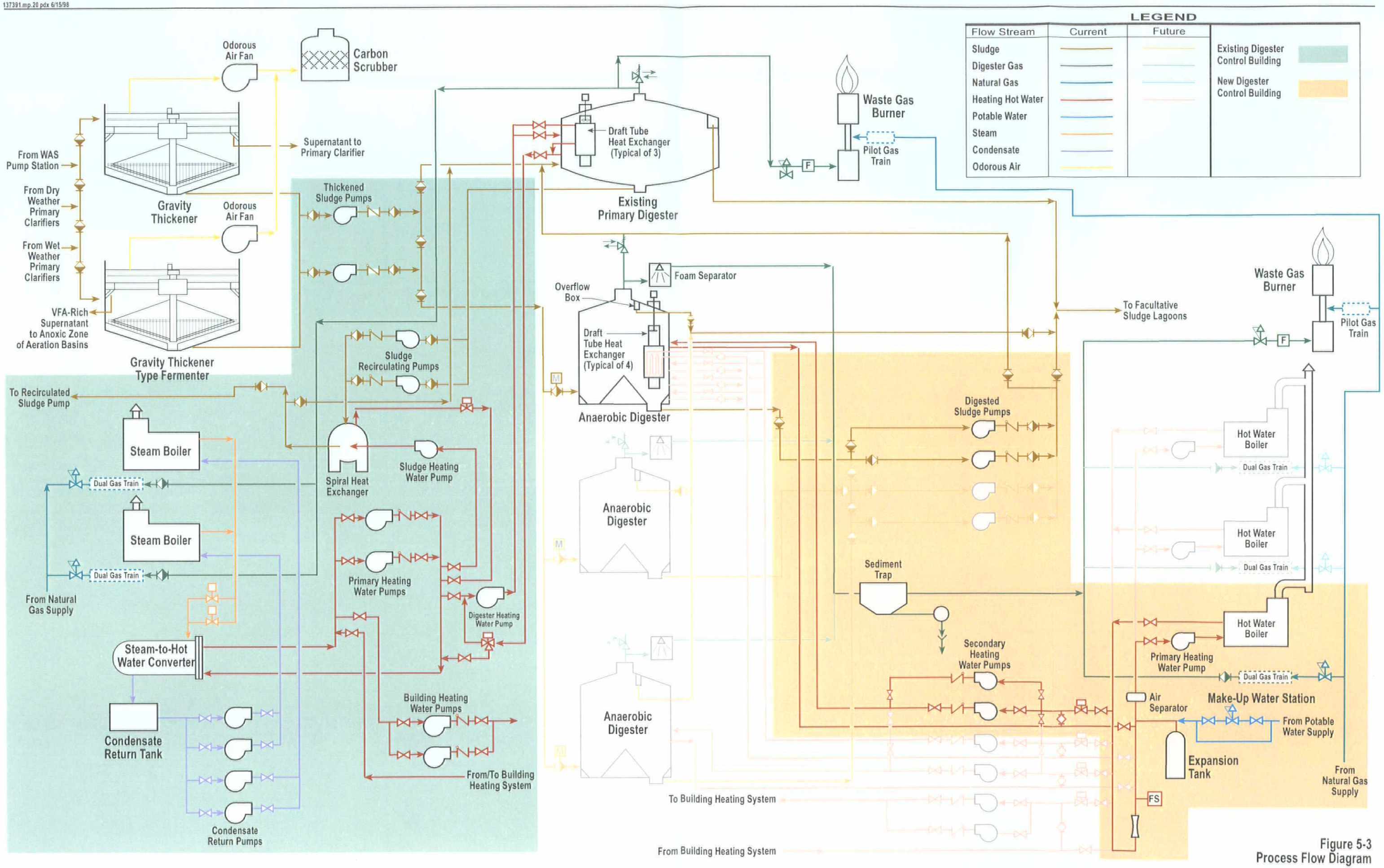

Figure 5-3
Process Flow Diagram

SECTION 6

Wastewater Treatment, Effluent Disposal, and Biosolids Management Improvements

On the basis of the Corvallis WWRP flow and wasteload projections (discussed in Section 5) and the regulatory planning scenarios (developed in Section 2), alternatives were developed for each of the liquids and solids unit processes. Treatment alternatives consisting of various unit processes were then configured to meet the various regulatory planning scenarios and evaluated for selection of a treatment alternative. This section outlines the development, evaluation, and selection of the recommended alternatives for liquids treatment, effluent disposal, and biosolids management.

Methodology

The methodology for selecting the recommended liquids and solids improvements consisted of the following steps:

1. A full range of alternatives capable of achieving the regulatory or environmental adjacency planning criteria was developed for each unit process.
2. These alternatives were arranged into treatment alternatives capable of achieving the criteria. Alternatives that did not fit within the existing process were eliminated.
3. An initial screening analysis was conducted to reduce the number of treatment alternatives to only those that are the most viable.
4. The remaining alternatives were then subjected to a detailed alternative analysis.
5. From this detailed alternative analysis, the preferred treatment alternatives were selected for each of the regulatory planning scenarios.

The majority of the alternative analyses were conducted using a population of 80,000 to meet the most probable regulatory scenario. This allowed for a common basis on which to evaluate and select treatment alternatives. The exception to that was where a buildout analysis was deemed more appropriate (e.g., headworks) or where the treatment alternatives would only be required for more stringent regulatory planning criteria (e.g., tertiary treatment). The description of each detailed alternative analysis specifies the design and evaluation criteria for each treatment process.

Screening Analysis

The initial screening analysis consisted of monetary and non-monetary evaluations. The basis for these evaluations is outlined below.

Monetary Evaluation

- Capital costs only
- Costs based on cost curves, simplified cost models, and similar sized facilities constructed on previous projects

Non-Monetary Evaluation

- Ranking of each alternative based on the criteria shown in Table 6-1
- No weighting of the criteria; i.e., each criterion was considered of equal importance
- Elimination of those alternatives that are deemed infeasible because of site constraints or could not otherwise be implemented

Detailed Alternative Evaluations

The detailed alternative evaluations were also based on both monetary and non-monetary evaluations. The attributes of each of portion of these evaluations are outlined below.

Monetary Evaluation

- Capital costs and annual operations and maintenance (O&M) costs; these costs were then rolled up to a present worth life cycle cost
- Capital costs based on detailed costing of major equipment, systems, and structures; actual construction costs for similar sized facilities on previous projects; and allowances for mechanical, electrical, instrumentation and control (I&C), and sitework
- O&M costs based on detailed analysis of annual costs for labor, equipment maintenance, electricity, fuel, chemicals, and miscellaneous costs (e.g., truck hauling)
- Contingency and engineering costs were included at 25 percent for each

Non-Monetary Evaluation

The non-monetary evaluation for detailed alternative analysis consisted of a specific discussion of the relative advantages and disadvantages of each alternative. No specific non-monetary ranking system was used in the detailed alternative analysis.

Liquids Treatment Process and Effluent Disposal

The liquids treatment analysis was developed to address the three regulatory planning scenarios. These planning scenarios and the basic treatment requirements are summarized in Table 6-2. Regulatory requirements are discussed in Section 2 of this master plan.

TABLE 6-1
Non-Monetary Screening Analysis Criteria and Ranking Score Explanations

Item	Description
Criteria	
Capital Cost	The order-of-magnitude capital cost based on a population of 80,000 and most probable scenarios.
Operating Cost	The relative operations and maintenance cost of the alternative.
System Reliability	The mechanical and physical reliability of the systems, equipment, and components that make up the alternative.
Impacts to Facility Operation	The impact that implementation of the alternative will have on facility operations.
Impacts to Facility Maintenance	The impact that implementation of the alternative will have on facility maintenance.
Operational Complexity	The relative complexity of operation of the alternative including maintenance, monitoring, and management requirements.
Staffing Requirements	The impact to facility staffing; '5' for no staffing impacts, '3' for little or no staff impacts, '1' for a resulting staff increase.
Ease of Construction	The ease with which the alternative could be incorporated into the current facility's configuration, both in terms of disruption during construction activities and consistency with current facility layouts.
Adaptability to Changing Regulations	The alternative's overall ability to adapt to more stringent regulatory requirements including add-on or modified facility configurations with minimal re-work.
Adaptability to Existing Site	The alternative's overall ability to fit on the existing site footprint (i.e., no additional land requirements)
Mitigation of Adjacency Issues	The alternative's overall contribution to or mitigation of adjacency issues (e.g., odor, noise, traffic, public acceptance, and safety) including the ability to add-on or modify the facility configuration to address these issues.
Ranking Scores	
1	The alternative has a very negative impact on or does not address the stated criterion.
2	The alternative has a somewhat negative impact on the stated criterion.
3	The alternative has a neutral impact on the stated criterion.
4	The alternative favorably addresses the stated criterion.
5	The alternative very favorably addresses the stated criterion.

TABLE 6-2
Liquid Process Treatment Requirements for Various Regulatory Planning Scenarios

Regulatory Planning Scenario	Basic Treatment Requirements
Current Conditions	Carbonaceous BOD removal Chlorination with residual limits
Most Probable	Summer nitrification Filtration Dechlorination
Most Stringent	Summer nitrification/denitrification Filtration Tertiary treatment (phosphorus removal) Summer temperature mitigation Eliminate by-products of chlorination/dechlorination

The alternatives development, evaluation, and process selections to meet these treatment criteria are discussed below.

Liquid Process Alternatives Development

The analysis of the liquids treatment system looked at the following unit processes:

- Influent pumping and screening
- Grit removal
- Primary clarification
- Secondary treatment
- Secondary clarification
- Tertiary treatment
- Filtration
- Metals control
- Temperature management
- Disinfection/dechlorination
- Effluent pumping

Table 6-3 presents a summary of the alternatives that were developed for each unit process for each regulatory scenario. The basic alternatives for increasing capacity to maintain current regulatory conditions are listed first along with the capacity driver for each unit process. Alternatives to address additional incremental improvements with gradually more stringent regulatory criteria are then listed for the Most Probable and Most Stringent planning scenarios. Table 6-4 provides the sizing criteria for each of these alternatives to serve a population of 80,000. Table 6-5 provides the sizing criteria for buildout conditions.

After determining the various unit process alternatives for each of the regulatory planning scenarios, these alternatives were assembled into treatment alternatives. This made it possible to ascertain how the various treatment alternatives could be adapted to address increasingly more stringent treatment requirements, how well the various unit processes complemented each other, and what the site impacts would be of certain treatment

Table 6-3
Liquids Unit Process Alternatives for Various Regulatory Scenarios

	Incremental Improvement Alternatives									
	Current Conditions			Most Probable			Most Stringent			
Process	Capacity Driver	Alternative 1	Alternative 2	Incremental Driver	Alternative 1	Alternative 2	Incremental Driver	Alternative 1	Alternative 2	Alternative 3
Influent pumping & screenings	WWPH flow	Wet pit/dry pit (expanded)	Wet pit/dry pit (new)							
Grit removal	WWPD flow	Aerated	Vortex							
Primary clarification	WWMM flow	Additional rectangular clarifiers					BOD/TSS, P	Chemical addition		
Secondary treatment	BOD/TSS loading	BFAS: Additional TFs and/or media replacement. No new ABs.	RFAS: No TF modifications. Add ABs.	Ammonia-N	RFAS with additional AB volume for nitrification.	Post-secondary nitrifying TF w/ "solids control" (solids control options to include clarifiers, chlorine contact basin, filters)	TN, P	RFAS with additional AB volume for nitrification, denitrification, and P removal.		
Secondary clarifiers	WWMM flow	Circular		BOD/TSS, P	Additional clarifier area	Source control of P				
Tertiary treatment							P, Metals	Chemical addition with reactor/ flocculator clarifier	Irrigation reuse (agricultural and golf course)	Constructed wetlands treatment
Filtration				BOD/TSS, TN, P, Metals	Conventional deep bed					
Metals removal							Metals	Source control/ diffuser		
Effluent Cooling							Temperature	Stream mitigation	Covered TF	
Disinfection/ dechlorination	WWPH flow	Liquid chlorine		Chlorine toxicity	Sulfur dioxide dechlorination w/ new CCB		Disinfection byproducts, safety	UV only (dechlorination is not an alternative).	Hypochlorite and bisulfite	Ozonation
Effluent Pump Station	WWPH flow	Expand existing. (Add flow measurement and increase reliability/ redundancy)								

alternatives. Figures 6-1 through 6-3 show the treatment alternatives for the Current Conditions Scenario, Most Probable Scenario, and the Most Stringent Scenario, respectively.

Initial Screening of Liquid Process Alternatives

After developing the alternative unit processes, the various treatment alternatives were screened to determine which alternatives were viable. The remaining alternatives were retained for more detailed analysis. Included below is a summary of the results of the screening analysis.

Influent Pump Station/Headworks

Table 6-6 shows the results of the capital cost and qualitative analysis comparison of the two alternatives for the influent pump station and headworks: expand existing or construct new. Since Alternative No. 1 has a significant advantage in cost and Alternative No. 2 was ranked much higher in the qualitative comparison, it was decided to evaluate both alternatives in more detail. Therefore, the following alternatives were selected for detailed alternative analysis:

Alternative No. 1—Expand existing headworks facility

Alternative No. 2—Provide a new influent pump station and new screenings facility

Primary Treatment

The design of the CSO treatment facilities established the expansion scenario for grit removal and primary clarification. This CSO design includes the addition of a rectangular primary clarifier for use in the existing treatment plant. Therefore, no alternatives to rectangular clarifiers were evaluated. The wet weather treatment facility includes the conversion of the existing round primary clarifier (6700 clarifier) into a wet weather clarifier and the addition of two new rectangular wet weather primary clarifiers. Because of its location, the future addition of primary clarifiers will require that the existing round primary clarifier be demolished and a rectangular primary clarifier be added to replace it in the wet weather plant.

Expansion of the primary treatment facilities will require that the wet weather primary clarifiers be shifted to the west with each subsequent addition of non-CSO primary clarifier capacity. In this way, the wet weather primaries are always the most westerly bank of three primary clarifiers.

Secondary Treatment

Table 6-7 shows a summary of the screening analysis that was conducted for secondary treatment. Due to the high cost and relatively low qualitative ranking, Alternative No. 4, post-secondary trickling filter with some form of solids control, was eliminated. In addition, Alternative No. 3, conversion of the existing trickling filter tanks to suspended-growth activated sludge tanks, was also eliminated because of uncertainty regarding the trickling filter tanks' structural soundness and the inefficient use of the plant site afforded by two round tanks.

Table 6-4
Liquids Unit Process Alternative Sizing Under Various Regulatory Scenarios: 80,000 Population

	Incremental Improvement Alternative Sizing								
	Current Conditions			Most Probable			Most Stringent		
Process	**Alternative 1**	**Alternative 2**	**Alternative 3**	**Alternative 1**	**Alternative 2**	**Alternative 3**	**Alternative 1**	**Alternative 2**	**Alternative 3**
DWMM flow (mgd)	20.14	20.14	20.14	20.14	20.14	20.14	20.14	20.14	20.14
WWMM flow (mgd)	31.02	31.02	31.02	31.02	31.02	31.02	31.02	31.02	31.02
WWPH flow (mgd)[1]	57	57	57	57	57	57	57	57	57
Influent pumping	**Wet pit/dry pit (expanded)**	**Wet pit/dry pit (new)**							
Pumps	Existing: 1 @ 7 mgd, 3 @ 14 mgd. Expansion: 2 @ 14 mgd.	Discard existing. New: 5 @ 14 mgd.							
Wet pit	Expansion: 10'lx6'w	New: 30'lx6'w							
Dry pit	Expansion: 20'lx15'w	New: 50'lx15'w							
Screening	**Coarse Bar (expanded)**	**Coarse Bar (New)**							
WWPH approach velocity (ft/s)	3	3							
Total width of expansion/new channels (inches)	0	73							
Grit removal	**Aerated (expanded)**								
WWPH HRT (min)	3								
Expansion volume (ft^3)	7410								
Primary clarification	**Additional rectangular clarifiers**						**Chemical addition**		
WWMM SOR (gpd/ft^2)	1000								
Additional area (ft^2)	21129								
Additional number	4								
Secondary treatment	**TFAS: Additional TFs. No new ABs.**	**TFAS: Raise height of existing TFs and replace media. No new Abs.**	**RFAS: No TF modifications. Add ABs.**	**TFAS. Expand TF capacity by raising height, media change, and/or additional units.**	**Post-secondary nitrifying TF w/ "solids control" (solids control options to include clarifiers, chlorine contact basin, filters)**	**RFAS with additional AB volume for nitrification.**	**RFAS with additional AB volume for BNR (Nutrified sludge-type process)**	**Convert TFs to activated sludge tanks (Wilson, NC-type BNR)**	
TF BOD loading ($lb/d/1000\ ft^2$)	30 to 60	30 to 60	< 300	<30	N/A	<300	<300	N/A	
Minimum TF recycle ratio	2:1	2:1	2:1	2:1	2:1	2:1	2:1	N/A	
Aeration SRT (d)	1	1	3	1	3	10	10 to 15	15	
Additional TFs	0	0	0	1	0	0	0	0	
Additional ABs	0	0	0	0	0	2	1 or 2	2	
Secondary clarifiers	**Additional circular clarifiers**								
WWMM SOR (gpd/ft^2)	600								
Additional area (ft^2)	30926								
Additional number	3								
Tertiary treatment							**Chemical addition with reactor/ flocculator clarifier**	**Irrigation reuse (agricultural and golf course)**	**Constructed wetlands treatment**
WWMM SOR (gpd/ft^2)							800		
Clarifier diameter (ft)							111		
Total clarifier area (ft^2)							38775		
Number of clarifiers							4		
Reuse storage (acre-feet)								11913	
Average storage depth (ft)								8	

Table 6-4
Liquids Unit Process Alternative Sizing Under Various Regulatory Scenarios: 80,000 Population

	Incremental Improvement Alternative Sizing								
	Current Conditions			Most Probable			Most Stringent		
Process	**Alternative 1**	**Alternative 2**	**Alternative 3**	**Alternative 1**	**Alternative 2**	**Alternative 3**	**Alternative 1**	**Alternative 2**	**Alternative 3**
Storage area (acres)								1500	
Irrigation pumps								3 @ 16 mgd	
Pump station footprint								25'lx15'w	
Filtration				**Conventional deep bed**					
DWMM Filtration rate (gpm/ft^2)				6					
Filter area (ft^2)				2331					
Cell length (ft)				30					
Cell width (ft)				20					
Number of cells				5					
Footprint length (ft)				90					
Footprint width (ft)				80					
Backwash pumps and air scour blower building				25'x25'					
Metals removal							**Source control/ diffuser**		
Effluent Cooling							**Stream mitigation**	**Covered TF**	
Disinfection/ dechlorination	**Sodium hypochlorite disinfection**			**Sodium bisulfite dechlorination w/ new CCB**			**UV only (dechlorination is not an alternative). Assumed Trojan System 4000**	**Ozonation**	
WWPH HRT (min)	30			No contact time needed.					
CCT volume (ft^3)	158735								
Number of basins	2								
Passes per basin	3								
Pass width (ft)	15								
Pass depth(ft)	15								
Pass length (ft)	118								
Overall length (ft)	128								
Overall width (ft)	103								
UV Dose (mW-s/cm^2)							40	Ozone dose: 6 mg/l	
Approx length (ft)							40	120	
Approx width (ft)							20	120	
Effluent Pump Station	**Expand existing. (Add flow measurement and increase reliability/redundancy)**								
Pumps	5 @ 14 mgd.								
Structure	50'lx15'w								

Table 6-5
Liquids Unit Process Alternative Sizing Under Various Regulatory Scenarios: Probable Buildout Population (120,000)

Process	Incremental Improvement Alternative Sizing								
	Current Conditions			Most Probable			Most Stringent		
	Alternative 1	Alternative 2	Alternative 3	Alternative 1	Alternative 2	Alternative 3	Alternative 1	Alternative 2	Alternative 3
DWMM flow (mgd)	30.21	30.21	30.21	30.21	30.21	30.21	30.21	30.21	30.21
WWMM flow (mgd)	46.53	46.53	46.53	46.53	46.53	46.53	46.53	46.53	46.53
WWPH flow (mgd)[1]	85.5	85.5	85.5	85.5	85.5	85.5	85.5	85.5	85.5
Influent pumping	**Wet pit/dry pit (expanded).**	**Wet pit/dry pit (new)**							
Pumps	Existing: 1 @ 7 mgd, 3 @ 14 mgd. Expansion: 4 @ 14 mgd.	Discard existing. New: 7 @ 14 mgd.							
Wet pit	Expansion: 20'lx6'w	New: 40'lx6'w							
Dry pit	Expansion: 30'lx15'w	New: 65'lx15'w							
Screening	**Coarse Bar (expanded)**	**Coarse Bar (New)**							
WWPH approach velocity (ft/s)	3	3							
Total width of expansion/new channels (inches)	17	89							
Grit removal	**Aerated (expanded)**								
WWPH HRT (min)	3								
Expansion volume (ft^3)	15350								
Primary clarification	**Additional rectangular clarifiers**						**Chemical addition**		
WWMM SOR (gpd/ft^2)	1000								
Additional area (ft^2)	36639								
Additional number	9								
Secondary treatment	**TFAS: Additional TFs. No new ABs.**	**TFAS: Raise height of existing TFs and replace media. No new ABs.**	**RFAS: No TF modifications. Add ABs.**	**TFAS. Expand TF capacity by raising height, media change, and/or additional units.**	**Post-secondary nitrifying TF w/ "solids control" (solids control options to include clarifiers, chlorine contact basin, filters)**	**RFAS with additional AB volume for nitrification.**	**RFAS with additional AB volume for BNR (Nutrified sludge-type process)**	**Convert TFs to activated sludge tanks (Wilson, NC-type BNR)**	
TF BOD loading ($lb/d/ft^2$)	30 to 60	30 to 60	< 300	<30	N/A	<300	<300	N/A	
Minimum TF recycle ratio	2:1	2:1	2:1	2:1	2:1	2:1	2:1	N/A	
Aeration SRT (d)	1	1	3	1	3	10	10 to 15	15	
Additional TFs	1	0	0	4	0	0	0	0	
Additional ABs	0	0	1	0	1	5	4 to 6	6	
Secondary clarifiers	**Additional circular clarifiers**								
WWMM SOR (gpd/ft^2)	600								
Additional area (ft^2)	56776								
Additional number	6								
Tertiary treatment							**Chemical addition with reactor/ flocculator clarifier**	**Irrigation reuse (agricultural and golf course)**	**Constructed wetlands treatment**
WWMM SOR (gpd/ft^2)							800		
Clarifier diameter (ft)							111		
Total clarifier area (ft^2)							58163		
Number of clarifiers							6		
Reuse storage (acre-feet)								17872	

Table 6-5
Liquids Unit Process Alternative Sizing Under Various Regulatory Scenarios: Probable Buildout Population (120,000)

Process	Incremental Improvement Alternative Sizing								
	Current Conditions			Most Probable			Most Stringent		
	Alternative 1	Alternative 2	Alternative 3	Alternative 1	Alternative 2	Alternative 3	Alternative 1	Alternative 2	Alternative 3
Average storage depth (ft)								8	
Storage area (acres)								2200	
Irrigation pumps								3 @ 25 mgd	
Pump station footprint								30'lx15'w	
Filtration				**Conventional deep bed**					
DWMM Filtration rate (gpm/ft^2)				6					
Filter area (ft^2)				3497					
Cell length (ft)				30					
Cell width (ft)				20					
Number of cells				7					
Footprint length (ft)				120					
Footprint width (ft)				80					
Backwash pumps and air scour blower building				25'x25'					
Metals removal							**Source control/ diffuser**		
Cooling							**Stream mitigation**	**Covered TF**	
Disinfection/ dechlorination	**Sodium hypochlorite disinfection**			**Sodium bisulfite dechlorination w/ new CCB**			**UV only (dechlorination is not an alternative). Assumed Trojan System 4000**	**Ozonation**	
WWPH HRT (min)	30			No contact time needed.					
CCT volume (ft^3)	238103								
Number of basins	3								
Passes per basin	3								
Pass width (ft)	15								
Pass depth(ft)	15								
Pass length (ft)	118								
Overall length (ft)	128								
Overall width (ft)	151								
UV Dose (mW-s/cm^2)							40	Ozone dose: 6 mg/l	
Approx length (ft)							40	160	
Approx width (ft)							30	120	
Effluent Pump Station	**Expand existing. (Add flow measurement and increase reliability/redundancy)**								
Pumps	7 @ 14 mgd.								
Structure	65'lx15'w								

TABLE 6-6
Headworks Screening Evaluation

	Alternative Ranking	
	No. 1 Influent Pumping/Screening—Expanded Existing	**No. 2 Influent Pumping/Screening—New**
Capital Cost Comparison	**$4,580,000**	**$6,720,000**
Qualitative Comparison		
Operating Cost	3	4
System Reliability	3	4
Impacts to Facility Operation	3	4
Impacts to Facility Maintenance	3	4
Operational Complexity	3	3
Staffing Requirements	5	5
Ease of Construction	2	5
Adaptability to Changing Regulations	3	5
Adaptability to Existing Site	5	3
Mitigation of Adjacency Issues	3	5
Overall Ranking	**33**	**42**

TABLE 6-7
Secondary Treatment Screening Evaluation

Criteria	Alternative Ranking			
	No. 1 TFAS: Add TF Capacity; No new AB's.	**No. 2 RFAS: No TF Modifications; Add AB's.**	**No. 3 Convert TFs to AS tanks (i.e., Wilson, NC-type BNR)**	**No. 4 Post-secondary nitrifying TF w/ solids control**
Capital Cost Comparison	**$4,970,000**	**$4,520,000**	**$4,200,000**	**$7,310,000**
Qualitative Comparison				
Operating Cost	5	3	2	4
System Reliability	5	4	4	3
Impacts to Facility Operation	4	3	3	2
Impacts to Facility Maintenance	4	4	3	3
Operational Complexity	4	3	3	2
Staffing Requirements	3	3	3	3
Ease of Construction	4	5	1	2
Adaptability to Changing Regs.	3	4	4	1
Adaptability to Existing Site	4	4	5	2
Mitigation of Adjacency Issues	3	4	5	4
Overall Ranking	**39**	**37**	**33**	**26**

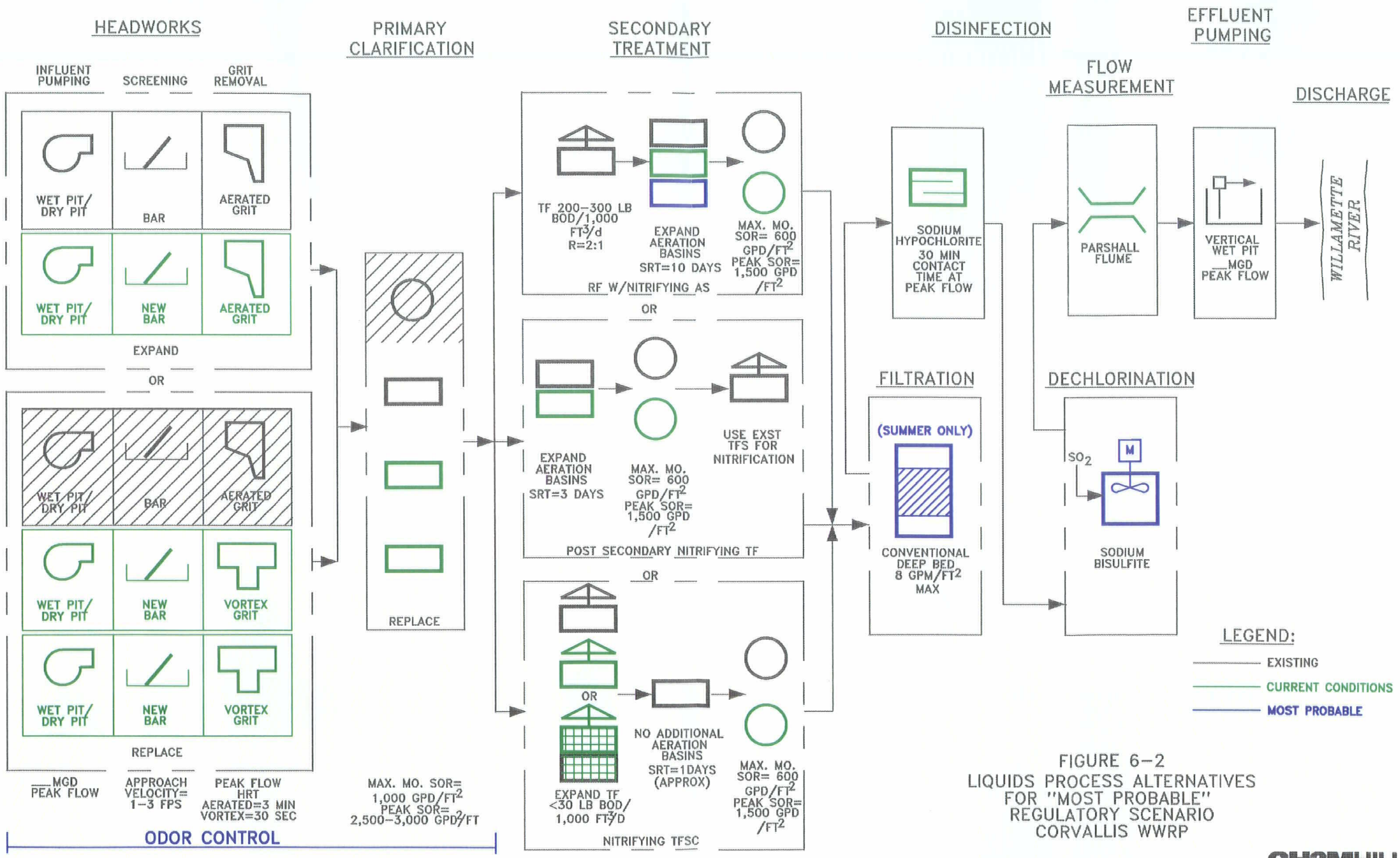

FIGURE 6-2
LIQUIDS PROCESS ALTERNATIVES
FOR "MOST PROBABLE"
REGULATORY SCENARIO
CORVALLIS WWRP

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16-JULY-1998

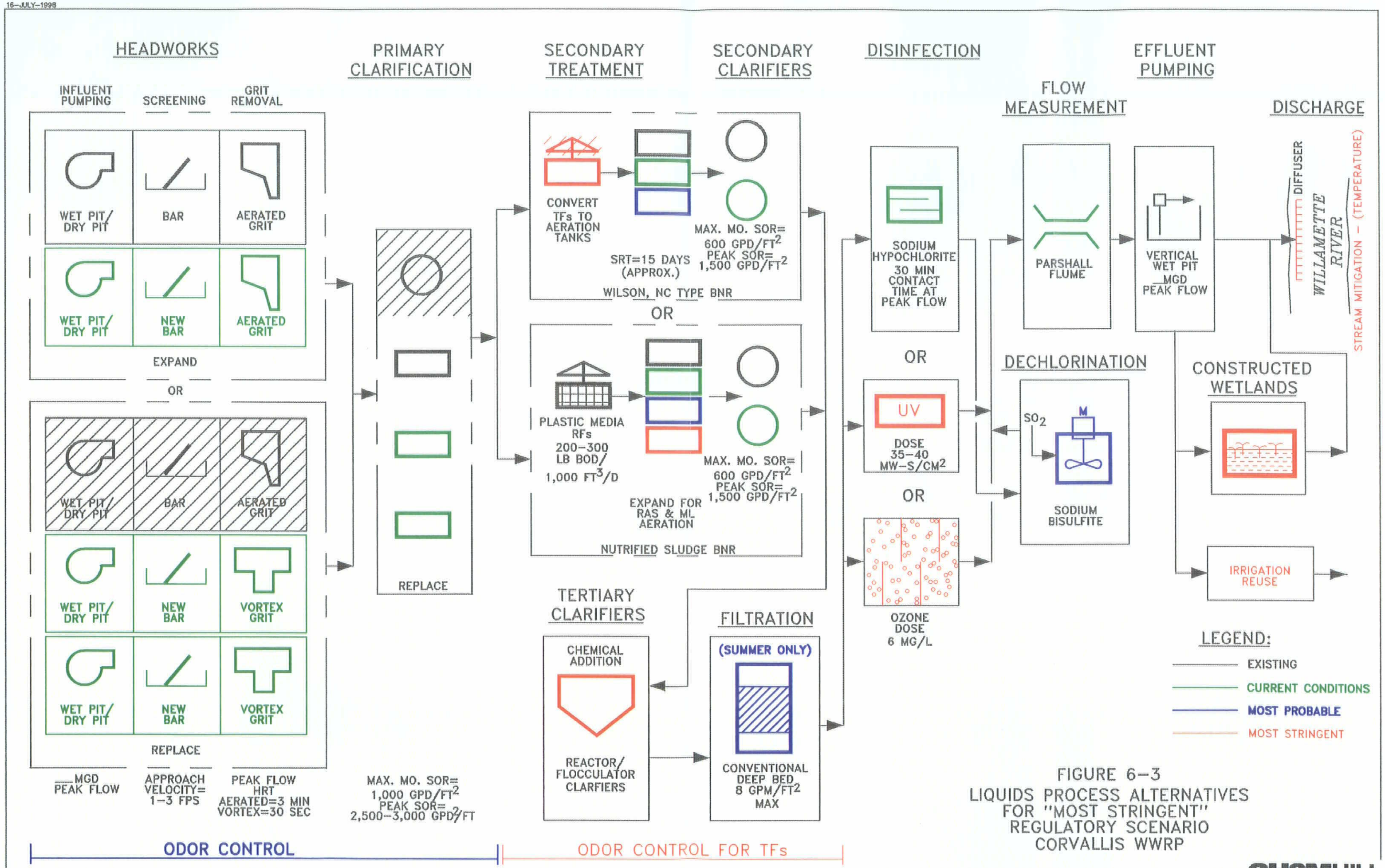

FIGURE 6-3
LIQUIDS PROCESS ALTERNATIVES FOR "MOST STRINGENT" REGULATORY SCENARIO
CORVALLIS WWRP

7391903.dwg

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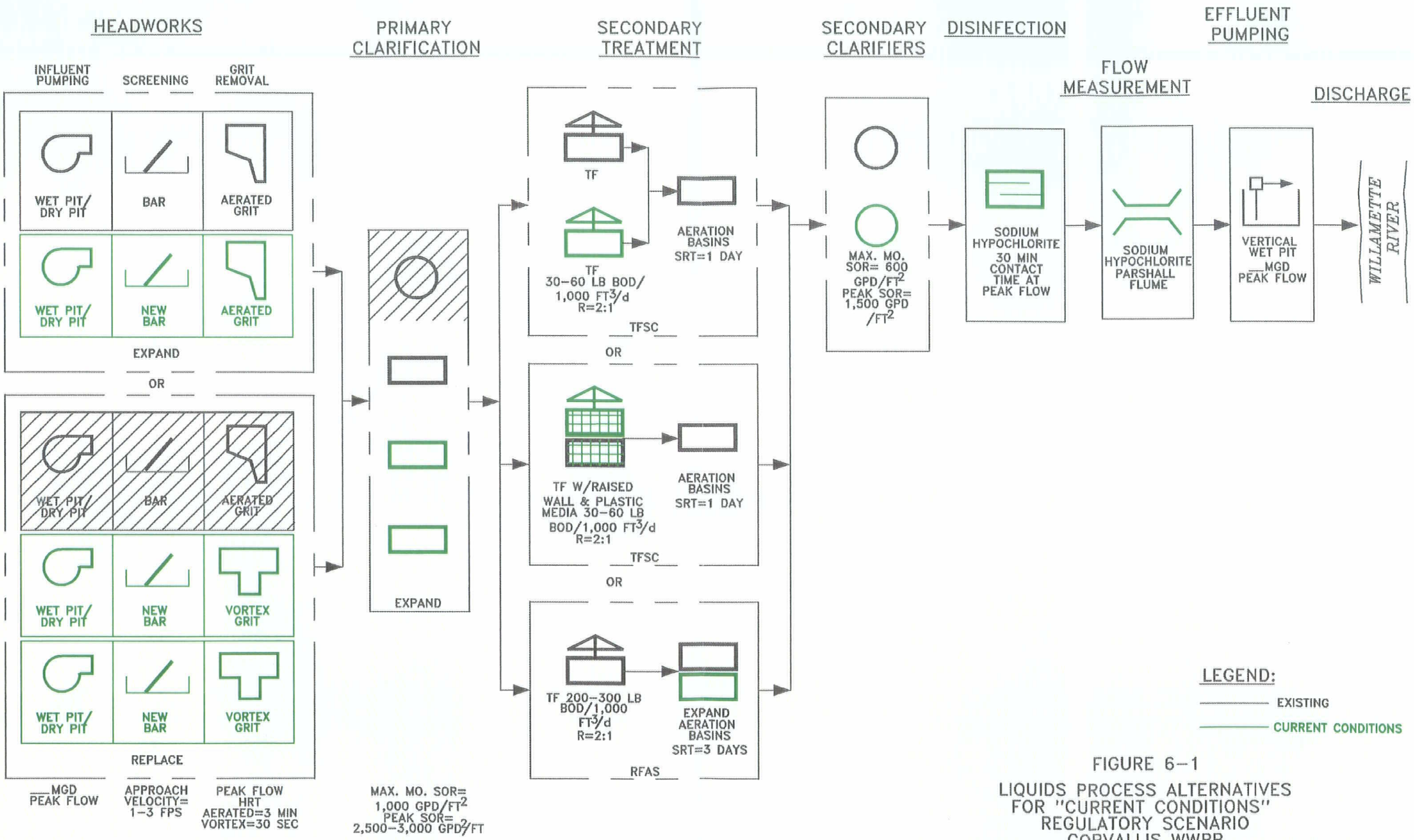

FIGURE 6-1

LIQUIDS PROCESS ALTERNATIVES FOR "CURRENT CONDITIONS" REGULATORY SCENARIO CORVALLIS WWRP

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7391G01.dwg

The alternatives that were selected for further detailed evaluations were:

Alternative No. 1—Trickling Filter/Activated Sludge—Add trickling filter capacity; no new aeration basins.

Alternative No. 2—Roughing Filter/Activated Sludge—No trickling filter modifications; add aeration basin capacity.

Tertiary Treatment

Three alternatives were evaluated for providing tertiary treatment to meet potential future low instream water quality standards for phosphorus. Note that this level of treatment would only be required for the most stringent regulatory scenario. These alternatives were:

Alternative No. 1—Chemical phosphorus removal using reactor/flocculator clarifiers.

Alternative No. 2—Irrigation reuse during the summer to eliminate all or partial discharge to the Willamette River during periods when nutrient limits are in force.

Alternative No. 3—Constructed wetlands treatment system.

Table 6-8 provides a summary of the screening evaluation. Alternative Nos. 2 and 3 were both eliminated because of the extremely high costs. However, depending on the nutrient limits that might be established, either of these alternatives might be used in concert with advanced secondary treatment (biological phosphorous removal) to reduce the nutrient load to the river to acceptable levels. For example, irrigation reuse might be used, along with biological nutrient removal, during the summer months to maintain the total phosphorus mass load to below regulatory requirements.

Therefore, Alternative No. 1 will be used for development of the Most Stringent Scenario. However, final determination on the treatment approach to nutrient removal should be made based on information available at the time the requirements are imposed.

Disinfection/Dechlorination

Table 6-9 shows the results of the screening analysis for disinfection/dechlorination. As can be seen from this analysis, Alternative No. 1, chlorine gas disinfection/sulfur dioxide dechlorination, is the lowest cost alternative and scored higher in the qualitative evaluation. Even so, this alternative was eliminated because of policy decisions made as a part of the CSO remediation project to eliminate the use of gaseous chlorine for safety reasons.

Alternative No. 4, Ozonation, was eliminated from further consideration because of its extremely high capital and operating costs. Capital costs for this alternative were nearly twice that estimated for UV disinfection and almost ten times the cost of a sodium hypochlorite/sodium bisulfite system. In alternative disinfection studies conducted for the East Bay Municipal Utility District (EBMUD) and the Upper Occoquan Sewage Authority (UOSA), annual operating costs for ozone were from 70 to 90 percent higher than those estimated for ultraviolet (UV) disinfection and 20 to 40 percent higher than for hypochlorite/bisulfite systems. In addition, the relatively few wastewater ozone applications (25 in the United States; most being less than 5 years old) provides little data to assess

TABLE 6-8
Tertiary Treatment Screening Evaluation

	Alternative Ranking		
Criteria	**No. 1 Chemical Addition w/ Reactor/Flocculator Clarifier**	**No. 2 Irrigation Reuse**	**No. 3 Constructed Wetlands Treatment**
Capital Cost Comparison	**$6,780,000**	**$48,000,000**	**$31,000,000**
Qualitative Comparison			
Operating Cost	2	1	4
System Reliability	3	4	5
Impacts to Facility Operation	2	3	3
Impacts to Facility Maintenance	2	4	5
Operational Complexity	2	4	3
Staffing Requirements	3	3	1
Ease of Construction	3	1	1
Adaptability to Changing Regs.	3	4	3
Adapability to Existing Site	4	1	1
Mitigation of Adjacency Issues	4	2	2
Overall Ranking	**28**	**27**	**28**

TABLE 6-9
Disinfection Screening Evaluation

Criteria	Alternative Ranking			
	No. 1 Chlorine Gas/ Sulfur Dioxide Dechlorination	**No. 2 UV Disinfection**	**No. 3 Sodium Hypochlorite/ Sodium Bisulfite**	**No. 4 Ozonation**
Capital Cost Comparison	**$0**	**$3,400,000**	**$750,000**	**$6,400,000**
Qualitative Comparison				
Operating Cost	5	3	1	1
System Reliability	4	3	3	2
Impacts to Facility Operation	5	3	2	3
Impacts to Facility Maintenance	5	1	3	1
Operational Complexity	5	3	3	2
Staffing Requirements	5	3	5	3
Ease of Construction	5	3	4	3
Adaptability to Changing Regs.	3	4	3	3
Adapability to Existing Site	5	4	4	3
Mitigation of Adjacency Issues	0	5	3	5
Overall Ranking	**42**	**32**	**31**	26

the long-term operations and maintenance implications of this technology. Thus, ozonation has typically been used for wastewater disinfection only in very small facilities or where site-specific conditions preclude the use of other technologies (e.g., extremely low UV transmittance because of industrial or food-processing discharge).

Based on this screening analysis, the following alternatives were selected for detailed evaluations:

Alternative No. 2—UV disinfection

Alternative No. 3—Sodium hypochlorite disinfection/sodium bisulfite dechlorination

Detailed Evaluation of Liquids Unit Process Alternatives

Following the screening evaluation, three liquid unit processes had alternatives that warranted detailed alternative analysis:

- Influent Pump Station/Headworks
- Secondary Treatment
- Disinfection/Dechlorination

These detailed evaluations are outlined below.

Influent Pump Station/Headworks

The detailed analysis of the influent pump station/headworks area looked at two alternatives:

Alternative No. 1—Expand existing headworks facility

Alternative No. 2—Provide a new influent pump station and new screenings facility

Criteria. The sizing and performance criteria used in the alternative analysis consisted of the following:

Design Flowrate:	34,700 gpm (50 mgd WWPHF)
Design Head:	39 feet
Redundancy Requirements:	Firm pumping capacity with largest unit out of service
Maximum Turndown:	50 percent of rated speed

Assumptions. The following were the assumptions that were made in configuring the facilities and analyzing the alternatives:

- All pumping modifications to be made within the existing pump station can be accommodated by the existing geometry.
- Existing piping connections and mounting base are in serviceable condition.
- All influent pumps will be provided with variable speed drives (adjustable frequency drive).
- The design of the existing pump station is adequate to meet future needs.

- Because the maximum capacity of the existing pump station appears to be approximately 50 mgd, the alternative comparison will be made over a 50-year life to account for the impact of adding a small "peak flow" pump station as the City reaches the 80,000-population level.

Alternative No. 1—Expand the Existing Headworks Facilities. The proposed improvements to the existing influent pump are shown in Figure 6-4. They are as follows:

- Replace the existing pumps in the existing influent pump station (one 7-mgd, three 14-mgd) with four new 16.7 mgd (11,600 gpm) influent pumps with 39 feet total dynamic head (TDH), 200 hp, and 705 rpm (variable).
- Remove the grating and concrete channeling in the wet well; install a floating baffle over the inlet drop pipes.
- Remove the 24-inch-by-20-inch reducer in the floor slab and replace with a 24-inch spool piece.

The proposed improvements to the headworks (screening and influent flow measurement) are shown in Figure 6-5. They are as follows:

- Remove the existing Parshall flumes and drop boxes downstream of the screenings channel.
- Install one new mechanically-cleaned bar screen in the existing bypass channel.
- Install three new 24-inch Parshall flumes, flow splitting channel, and five flow splitting weirs and drop boxes for distributing flow to primary clarifiers. Raise the Parshall flumes approximately 24 inches to eliminate submergence and provide head for downstream flow splits.

The relative advantages and disadvantages of this alternative are outlined below.

Advantages. The advantages are as follows:

- Maximizes use of existing installed equipment and facilities
- Lowest capital cost alternative
- Improves the Parshall flume approach conditions and submergence problems

Disadvantages. The disadvantages are as follows:

- No improvement in screening capability
- Does not allow for significant improvement to the influent wet well geometry (i.e., pump approach conditions)
- Does not provide future expansion capability beyond 80,000 population; inadequate space for future primary influent piping and flow splitting structure
- Construction of improvements must be made during 3-month dry weather period to allow use of the wet weather pump station.
- Existing configuration does not allow for individual pump isolation provisions.

Alternative No. 2—Provide a New Influent Pump Station and New Screening Facility. Construction of a new influent pump station and headworks was based on the preliminary design concepts developed for the wet weather treatment facility as part of the CSO remediation program. This pump station was originally developed as a wet well/dry pit facility with coarse screening similar to the existing headworks. Some specific features are outlined below:

- Construct a new headworks to include a new influent pump station, screenings facility, influent flow measurement, and primary flow splitting structure.
- Provide the new influent pump station with four influent pumps sized as indicated above.
- Construct a new coarse screening facility, with screenings handling, downstream of the influent pump station.
- Construct the new facilities for "hot" tie-ins to the existing yard piping.

This alternative's relative advantages and disadvantages are summarized below.

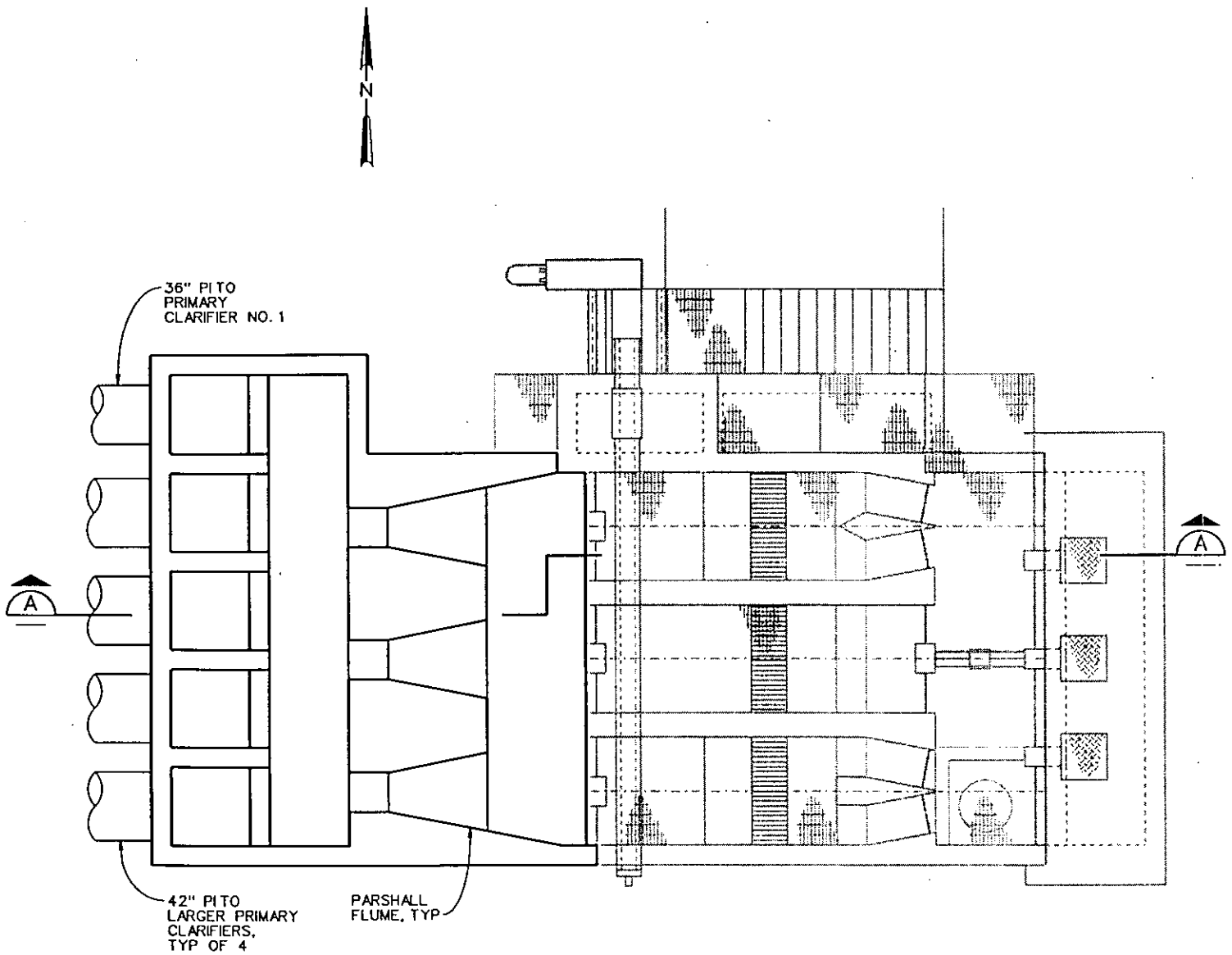
Advantages. The advantages are as follows:

- Provides improved screening capability
- Improves influent wet well hydraulics
- Eliminates concerns for influent flow measurement conditions
- Allows the greatest flexibility for improving upstream hydraulic constraints
- Provides significant facility performance improvements with the least number of constructibility problems
- Provides the most flexibility for future expansion
- Allows for improved odor containment in the screenings area

Disadvantages. The disadvantages are as follows:

- Most costly alternative; reuses very little of the existing headworks
- Requires the construction of additional anaerobic digestion capacity to replace the capacity demolished to construct the new headworks

Evaluation Results. A comparison of the cost for the two alternatives is given in Table 6-10. This summary shows a comparison of capital, annual O&M, and life cycle present worth costs for each alternative. From this analysis, Alternative No. 1 is the lowest cost alternative both in terms of capital expenditures and overall life cycle costs.



TOP PLAN
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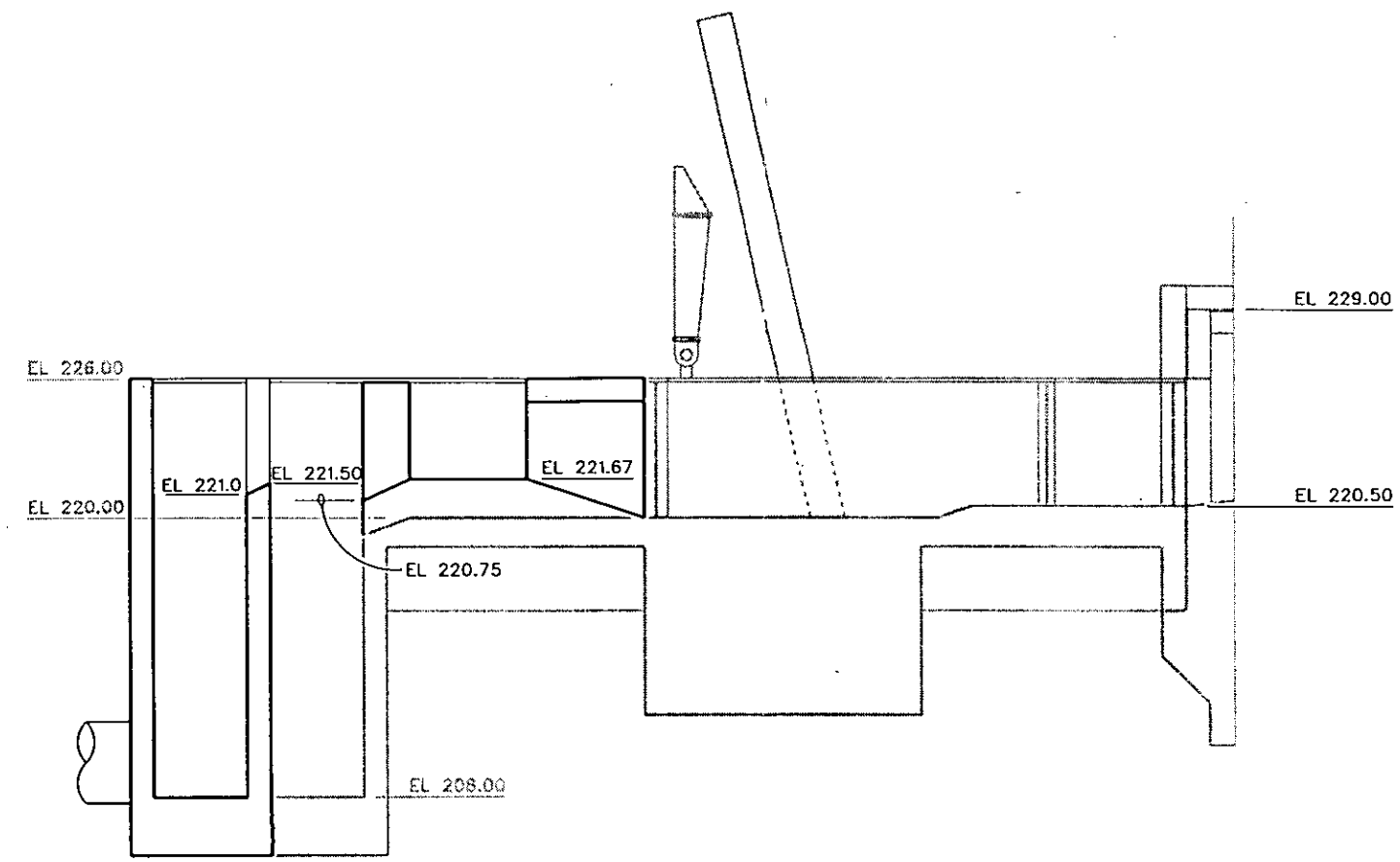

SECTION A
1/4"=1'-0"

FIGURE 6-5
PROPOSED HEADWORKS EXPANSION LAYOUT
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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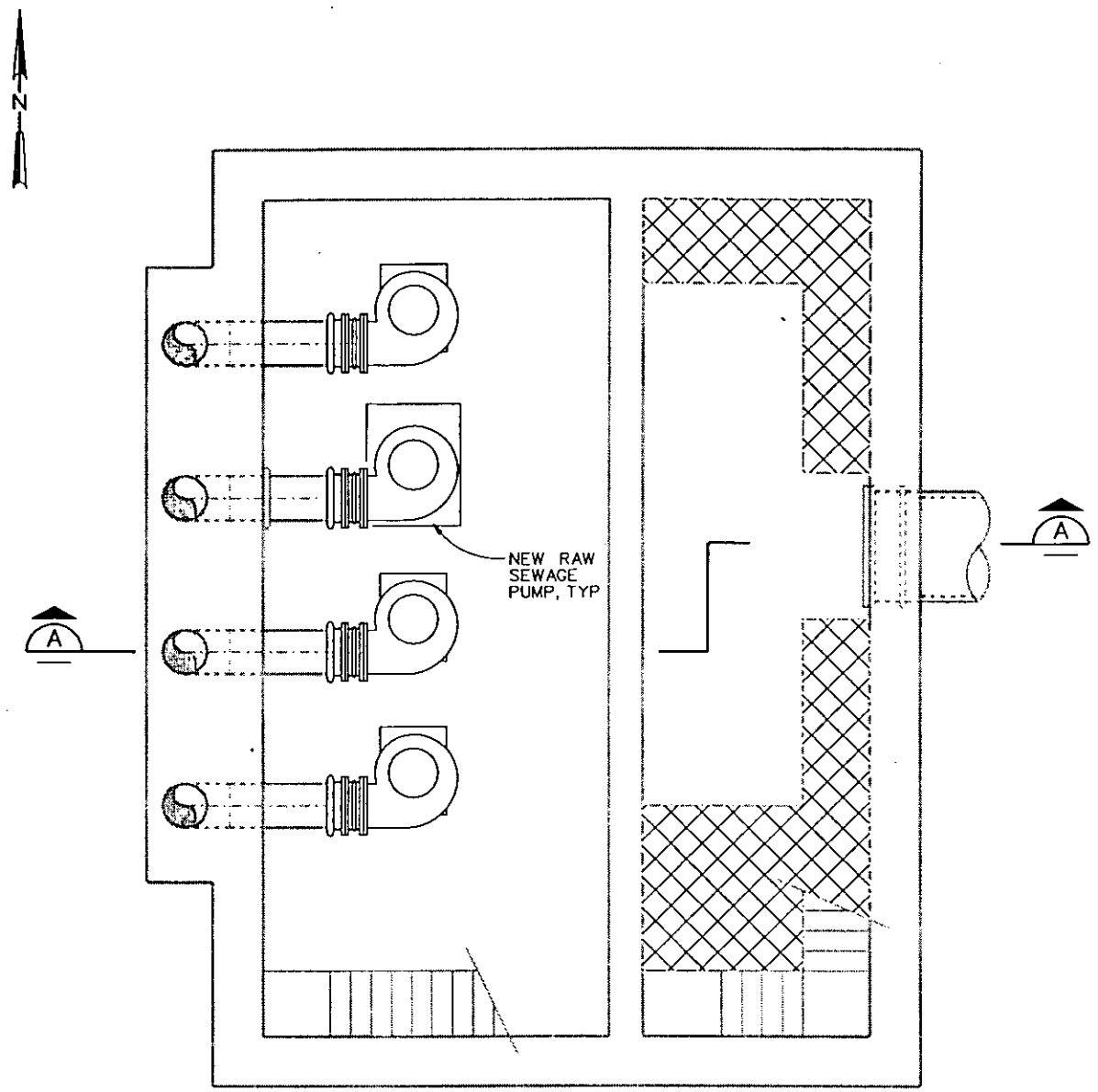

PLAN AT EL 200.00
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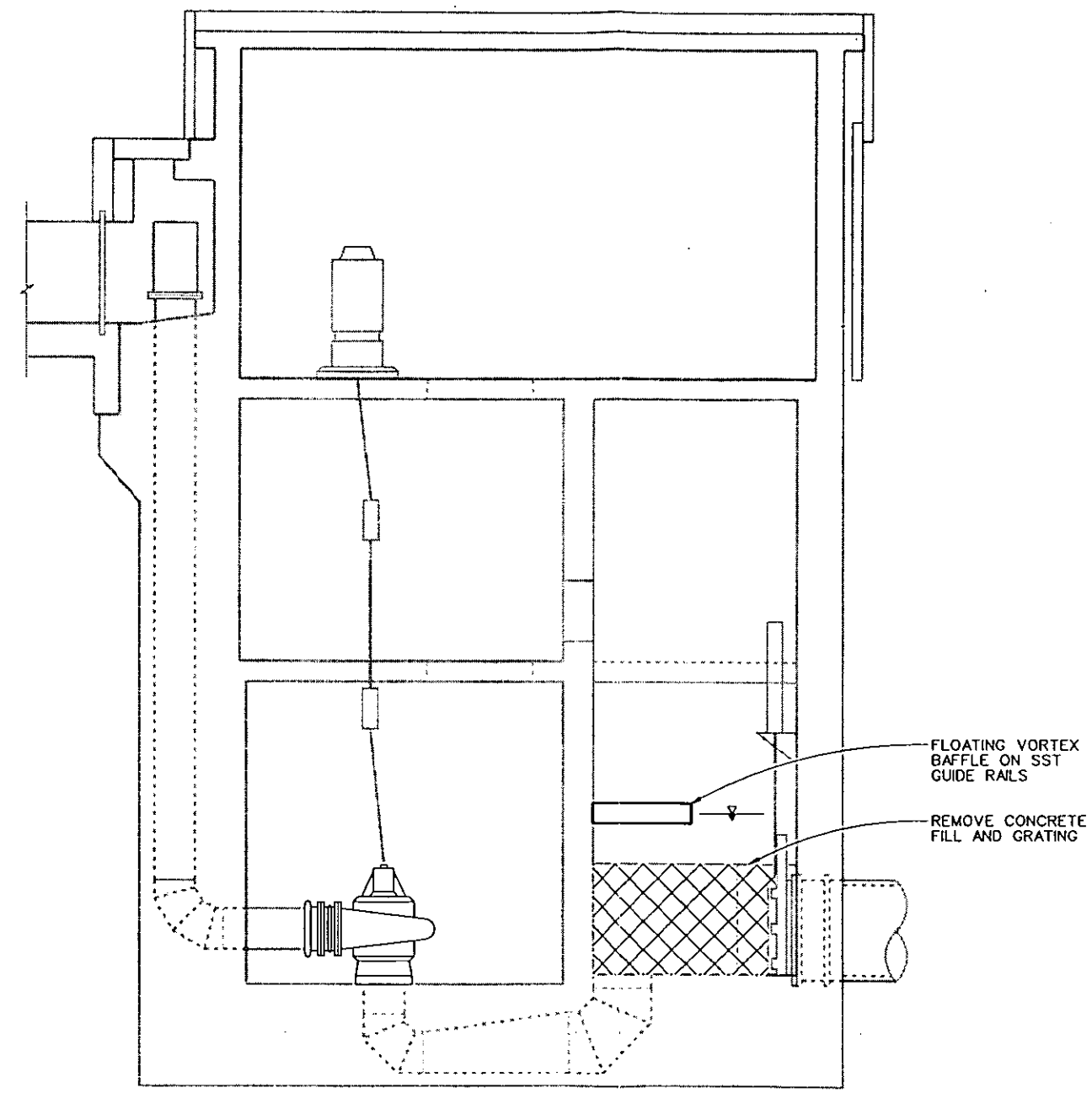

SECTION A
1/4"-1'-0"

FIGURE 6-4
PROPOSED INFLUENT PUMP STATION
EXPANSION LAYOUT
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

CS20M142.DLV

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In addition, the following observations were made:

1. The analysis of the existing influent pump station indicates that the original design intent was to expand this facility by increasing the capacity of the influent pumps. Evaluation of the existing pipe sizing (much of which is encased in the structural concrete) indicates that flow velocities, although high, are within an acceptable range. Approach conditions to the pumps are not optimal, but would be improved by the modifications shown.

2. The expansion provisions of the existing headworks do not adequately consider future primary flow splitting capabilities or the future plant hydraulic grade line (e.g., submerged flume conditions). Therefore, the proposed modifications to the existing headworks would need to address these issues.

3. A 14-mgd peak flow pump station would need to be added to expand beyond the 80,000-population level. The costs for this expansion are included in the life cycle cost analysis as a "future cost." This cost was then discounted to 1998 dollars.

TABLE 6-10
Headworks Detailed Alternative Evaluation Cost Summary

No.	Alternative	Initial Capital Cost	Annual Cost	20-Year Present Worth Cost	50-Year Present Worth Cost
1	Influent pump station—expand existing	$2,605,674	$127,259	$4,126,463	$4,760,356
2	New influent pump station	7,272,500	110,166	8,589,026	9,137,777

Note: See Appendix E for the details of the present worth cost analysis.

Alternative Selection. Based on this evaluation, Alternative No. 1 was selected for development during implementation planning of the future WWRP improvements. However, because of the uncertainty about the hydraulic performance of the proposed wet well modifications, it is recommended that physical modeling of the influent pump station be conducted before incorporating any improvements. This will allow different wet well configurations (e.g., baffling, pump suction modifications, etc.) to be tested and refined before making a major investment in the existing facility.

Secondary Treatment System

The detailed analysis of the secondary treatment system looked at two alternatives:

Alternative No. 1—Trickling Filter/Activated Sludge—Add trickling filter capacity; no new aeration basins.

Alternative No. 2—Roughing Filter/Activated Sludge—No trickling filter modifications; add aeration basin capacity.

Criteria. Since the most probable regulatory scenario is being used for the detailed alternative evaluation, the basic configuration of the secondary treatment alternatives was

established to provide summer nitrification. In addition, the following specific sizing and performance criteria were used in the alternative development and analysis:

Design condition:	Dry weather maximum month
Design flowrate:	20.14 mgd
Influent total Kjeldahl nitrogen (TKN):	35 mg/L
BOD loading:	9440 pounds per day
Wastewater temperature:	16 degrees C
Effluent ammonia:	<1 mg/L

Assumptions. The following were the assumptions made in configuring and analyzing the alternatives:

1. The existing trickling filters and aeration basins are in serviceable condition.
2. No additional aeration blower capacity will be required in either alternative.
3. Recently compiled bid results from the Salmon Creek WWTP in Clark County, Washington, and the Westside WWTP in Vancouver, Washington, were used to develop the construction costs for the aeration basin improvements.
4. The existing recycle pump station would be expanded to serve all of the trickling filters.

Alternative No. 1—Trickling Filter/Activated Sludge. The improvements to the existing trickling filters to provide a nitrifying TFAS system would consist of the following:

- Add one additional trickling filter with the following attributes:

Size:	160-ft diameter w/ 8-foot media depth
TF BOD loading:	30 lb/day/100 ft^2
Media:	Plastic
Recycle rate:	2-to-1
Aeration basin SRT:	1 day

- Add one trickling filter recycle pump with the following design criteria:

Capacity:	8,000 gpm (11.5 mgd)
Head:	25 feet TDH
Type:	Vertical, mixed flow
Horsepower:	75 hp

The relative advantages and disadvantages for this alternative are summarized below.

Advantages. The advantages are as follows:

- As an extension of the current operating scheme, it maintains the same operational simplicity that plant staff are accustomed to.
- This alternative will result in the lowest annual O&M cost because less aeration blower power consumption is required and less operator attention is necessary.

- Lowest capital cost alternative
- Treatment alternative will result in the most settleable solids; meaning that gravity thickening can continue to be used for WAS thickening.

Disadvantages. The disadvantages are as follows:

- Least efficient use of the plant site because it requires a relatively large footprint.
- Fixed film treatment is not easily adapted to provide biological nutrient removal (denitrification and biological phosphorous removal) should future regulatory requirements dictate.
- Greater potentials to create adjacency concerns such as odor generation, aesthetic complaints, and/or concerns over the birds that typically frequent the trickling filters.
- Snail growth in the trickling filters can cause operational problems with downstream solids handling processes.

Alternative No. 2—Roughing Filter/Activated Sludge. The improvements to provide a RFAS system would consist of the following:

- Trickling filters operated using the following criteria:

New TFs:	None
TF BOD loading:	300 lb/day/100 ft^2
Media:	Plastic
Recycle rate:	2-to-1

- Two additional aeration basins downstream of the trickling filters with the following attributes:

Size:	156-ft long by 40-ft wide, each basin
Sidewater depth:	16 feet
Diffused air system:	Fine bubble
Aeration basin SRT:	10 day

A discussion of the relative advantages and disadvantages is provided below.

Advantages. The advantages are as follows:

- Most efficient use of the plant site because it requires a smaller footprint than Alternative No. 1. Could have a long-term impact on site buildout.
- Suspended growth treatment is easily adapted to provide biological nutrient removal should future regulatory requirements dictate.

Disadvantages. The disadvantages are as follows:

- Higher solids production increasing the load on the digestion system.

- This alternative will result in the highest annual O&M cost because of the greater aeration blower power consumption and more day-to-day operational adjustment is necessary.
- Although less visually objectionable than Alternative No. 1, the higher BOD loading rates to the trickling filters could create greater odor generation potential.
- Highest capital cost for improvements.
- Would probably dictate the need for mechanical thickening of the WAS stream at some point in the future to maintain solids concentrations for thickened WAS.

Evaluation Results. Table 6-11 shows a summary of the cost comparison of the two alternatives. It includes a comparison of the capital, annual O&M, and the life cycle present worth costs. Given the low capital and O&M costs and the operational simplicity, Alternative No. 1 is the more desirable alternative if not constrained by the site or future discharge limits on nutrients. The following observations were made:

1. Considerable secondary treatment capacity still exists in the existing plant. Therefore, final selection of the secondary treatment alternative to be used for future expansion can be deferred to reflect more up-to-date knowledge on regulatory trends.
2. If nutrient limits were imposed, then suspended growth treatment would be preferred to fixed film. This is because of the greater flexibility to adapt suspended growth systems to provide biological nutrient removal.
3. If appropriate for future regulatory requirements, Alternative No. 1 is the preferred alternative of the plant operations staff. Particularly if higher recycle rates can be achieved and provisions can be made for discrete series operation of the trickling filters, this process provides very steady operation with little operator intervention. By denitrifying in the aeration basins, operators can further improve the process operation and resulting effluent quality.

TABLE 6-11
Secondary Treatment Detailed Alternative Evaluation Cost Summary

No.	Alternative	Capital Cost	Annual Cost	Present Worth Cost
1	Trickling filter/activated sludge	$3,660,696	$146,599	$5,412,605
2	Roughing filter/activated sludge	5,074,457	259,396	8,174,337

Note: See Appendix E for the details of the present worth cost analysis.

Alternative Selection. This evaluation suggests that a secondary treatment alternative selection be made on a conditional basis. For the most probable regulatory scenario, Alternative No. 1 was selected as the preferred alternative. However, should the most stringent regulatory scenario appear to be likely in the future, then Alternative No. 2 would be the preferred approach because it focuses capital investment on a gradual shift to suspended growth secondary treatment. This will be necessary to meet strict nutrient limits.

Current implementation planning should be based on Alternative No. 1, expansion of the plant's fixed film treatment capability. However, this selection should be revisited when capacity and/or regulatory requirements dictate that additional secondary treatment capacity is needed. If, at that time, the regulatory trends do not indicate the need for nutrient removal, then Alternative No. 1 should be the basis for plant expansion. However, if the regulatory environment is moving toward nutrient limits on the Willamette River dischargers, then strong consideration should be given to expansion based on Alternative No. 2.

Disinfection/Dechlorination System

A previous determination had been made to utilize gaseous chlorine for disinfection. However, policy decisions were made as part of the CSO Remediation Project to eliminate gaseous chlorine for safety reasons. To ensure that the master planning approach is consistent with these policies and the investments being made as a part of the CSO project, there was a need to re-visit the disinfection/dechlorination alternatives in the WWRP. The alternatives to be evaluated are:

Alternative No. 1—Sodium hypochlorite/sodium bisulfite for disinfection/dechlorination

Alternative No. 2—Ultraviolet (UV) disinfection

Criteria. The following specific sizing and performance criteria were used in the alternative development and analysis:

Design Population:	80,000 people
Peak Hour Flow:	58 mgd (for hydraulic capacity)
Annual Average Flow:	16.2 mgd (for O&M costs)
Chlorine Dose:	3 mg/L

Alternative No. 1—Sodium Hypochlorite/Sodium Bisulfite for Disinfection/Dechlorination. This alternative would consist of liquid chemical feed to supply sodium hypochlorite for disinfection and sodium bisulfite for dechlorination of the effluent stream. The chemical storage tanks and chemical feed systems would be located in space provided in the new disinfection facility being constructed as part of the CSO project. This allows the majority of the cost for the chemical storage and feed facilities to be defrayed as a part of the CSO facility requirements. Thus, the only new facilities provided with this alternative are a new chlorine contact basin to replace the existing circular chlorine contact basin.

Alternative No. 2—Ultraviolet (UV) Disinfection. In this alternative, UV disinfection would be provided for the WWRP effluent and sodium hypochlorite/sodium bisulfite would be used for disinfection/dechlorination of treated CSO flows. The new facilities would consist of a new UV disinfection facility designed to treat the peak hour WWRP flows. This facility was conceived as an outdoor installation with a small building to house electrical and control system equipment. The basic design criteria of this new facility are as follows:

UV System	Low-pressure, medium intensity
No. of lamps	4,000
No. of channels	4

UV design dose	32,000 uW-sec/cm^2
Design transmittance	65 percent
Level control	Weighted discharge gate
System accessories	Gantry crane, lamp module cleaning tank, standby power system

Evaluation Results. Table 6-12 shows a summary of the results of the alternative evaluation.

TABLE 6-12
Disinfection System Detailed Alternative Evaluation Summary

Criterion	Alternative No. 1 Sodium Hypochlorite/Sodium Bisulfite	Alternative No. 2 UV Disinfection
Capital Cost	$2,400,000	$4,300,000
O&M Cost	$169,000	$118,000
Present Worth Cost	$4,400,000	$5,700,000
Effluent Toxicity	Low	None
Safety Concerns	Low	Very Low
Impacts to Plant Hydraulics	~ 1 foot	~ 5 feet
O&M Labor	Low	High

Note: See Appendix E for the details of the present worth cost analysis.

From this evaluation, it can be seen that Alternative No. 1 has the lower capital cost, but the highest annual O&M cost. In addition, the following observations can be made:

1. Even though Alternative No. 1 is higher in annual O&M costs, primarily because of chemical costs, Alternative No. 2 requires a much higher level of O&M labor. This is due to lamp cleaning and replacement associated with a UV system.
2. The primary benefit of a UV disinfection system is that it eliminates the discharge of chemicals and their reactive by-products from the effluent stream. Therefore, implementation of this alternative would most likely be based on a regulatory requirement or policy decision to eliminate these byproducts from the effluent.

Alternative Selection. Based on the results of the evaluation, Alternative No. 1 was selected for future WWRP disinfection. Because sodium hypochlorite will be required for disinfection of CSO flows for the foreseeable future, selection of this alternative capitalizes on the investment being made as a part of the CSO remediation project by utilizing common facilities and operating strategies. However, it is recognized that UV disinfection may be evaluated again in the future to address changing regulations or a policy decision to eliminate chemical discharges. Therefore, the new chlorine contact basin should be designed with provisions to accommodate future conversion to UV disinfection channels. This would allow UV to be re-considered for the WWRP or for the entire effluent stream (both secondary effluent and treated CSO flows) based on technological advancements.

Solids Processes and Biosolids Management

The solids handling system evaluation was focused on meeting the three regulatory and environmental adjacency planning scenarios. These planning scenarios and the basic requirements associated with each are summarized in Table 6-13.

TABLE 6-13
Solids Handling Process Requirements for Various Planning Scenarios

Planning Scenario	Basic Process Requirements
Current Conditions	Class B stabilization Liquid land application
Most Probable	Improved thickening performance Thickener odor control Class B stabilization Liquid land application
Most Stringent	Mechanical thickening of WAS stream to address changes in the secondary treatment process Class A stabilization Eliminate facultative sludge lagoons to address odor concerns and/or space limitations Land application of dewatered biosolids

Class B stabilization criteria for land application of biosolids (per 40 CFR Part 503) permits the presence of low levels of pathogenic organisms (salmonella sp, enteric viruses, helminth ova). Therefore, restrictions are placed on the sites and types of crops where land application may occur. Distribution to the public or application to sites where public contact is possible is not permitted.

Class A stabilization criteria for land application of biosolids require that the biosolids be essentially pathogen-free. Under these criteria, no restrictions are placed on the sites and types of crops where land application may occur. Unrestricted distribution to the public or application to sites where public contact is possible is permitted.

The alternatives development, evaluation, and process selections to meet these criteria are discussed below.

Solids Handling Alternatives Development

The following solids handling system unit processes were evaluated:

- Sludge thickening
- Sludge stabilization
- Biosolids storage
- Biosolids dewatering

Table 6-14 presents a summary of the alternatives that were developed for each unit process for each regulatory scenario. The basic alternatives for increasing capacity to maintain current regulatory conditions are listed first along with the capacity driver for each unit process. Alternatives to address additional incremental improvements with gradually more stringent criteria are then listed for the most probable and most stringent planning

scenarios. Table 6-14 provides the sizing criteria for each of these alternatives to serve a population of 80,000. Table 6-15 provides the sizing criteria for buildout conditions.

Similar to the analysis that was done for the liquids treatment processes, after determining the various solids handling unit process alternatives for each planning scenario, the solids handling treatment alternatives were developed. This made it possible to ascertain how the various treatment alternatives could be adapted to address increasingly more stringent treatment requirements, how well the various unit process alternatives complemented each other, and what the site impacts would be for treatment alternatives. Figures 6-6 through 6-8 show the solids handling treatment alternatives for the Current Conditions Scenario, Most Probable Scenario, and the Most Stringent Scenario, respectively.

The solids production and solids handling design criteria were developed in Section 5. The basic sludge characteristics and mass balance for the solids handling system evaluations are outlined in Table 6-16.

Initial Screening of Solids Handling Alternatives

The various treatment alternatives were screened to determine which were either functionally or economically viable. The remaining alternatives were retained for more detailed analysis. Included below is a summary of the results of the screening analysis.

Sludge Thickening

Table 6-17 shows a summary of the screening analysis for sludge thickening. The screening analysis for sludge thickening indicated that gravity co-thickening of primary and waste activated sludge is the least costly alternative and also ranks highest in the qualitative comparison. Mechanical thickening of WAS would be advantageous only if the secondary treatment process were converted to all suspended growth. In this case, the poor settling characteristics of WAS leaving a suspended growth system would make mechanical thickening a more viable alternative.

Therefore, Alternative No. 1 was selected for development of the overall master plan. Alternative No. 2 should be considered in the event that nutrient limits drive a conversion to a suspended growth secondary process.

Sludge Stabilization

The continued use of anaerobic digestion for stabilization of sludge was selected for incorporation into the solids handling system based on the following:

- Its proven track record for providing cost-effective, reliable solids stabilization
- The significant investment in the existing digestion facilities
- The familiarity that the existing plant staff has with this process
- Process is easily adapted to provide Class A stabilization capability

As indicated, one of the advantages of the anaerobic digestion process is that it can be readily adapted to meet Class A pathogen reduction criteria. This can be done by two different methods.

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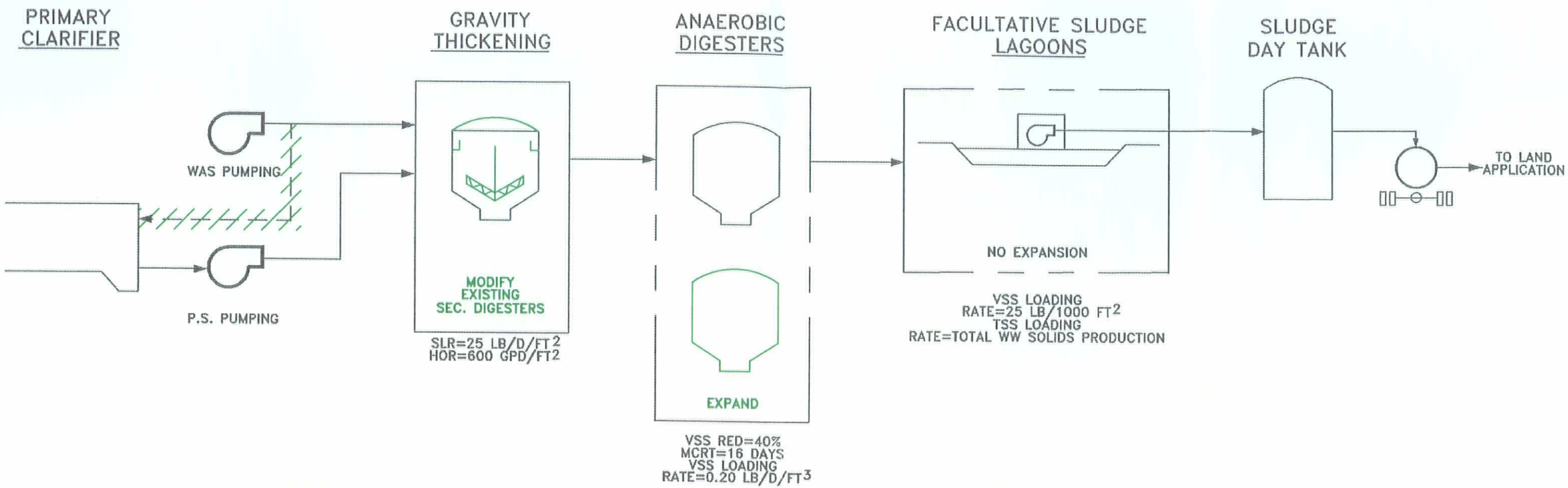

LEGEND:

EXISTING

CURRENT CONDITIONS

FIGURE 6-6
SOLIDS PROCESS ALTERNATIVES
FOR "CURRENT CONDITIONS"
REGULATORY SCENARIO
CORVALLIS WWRP

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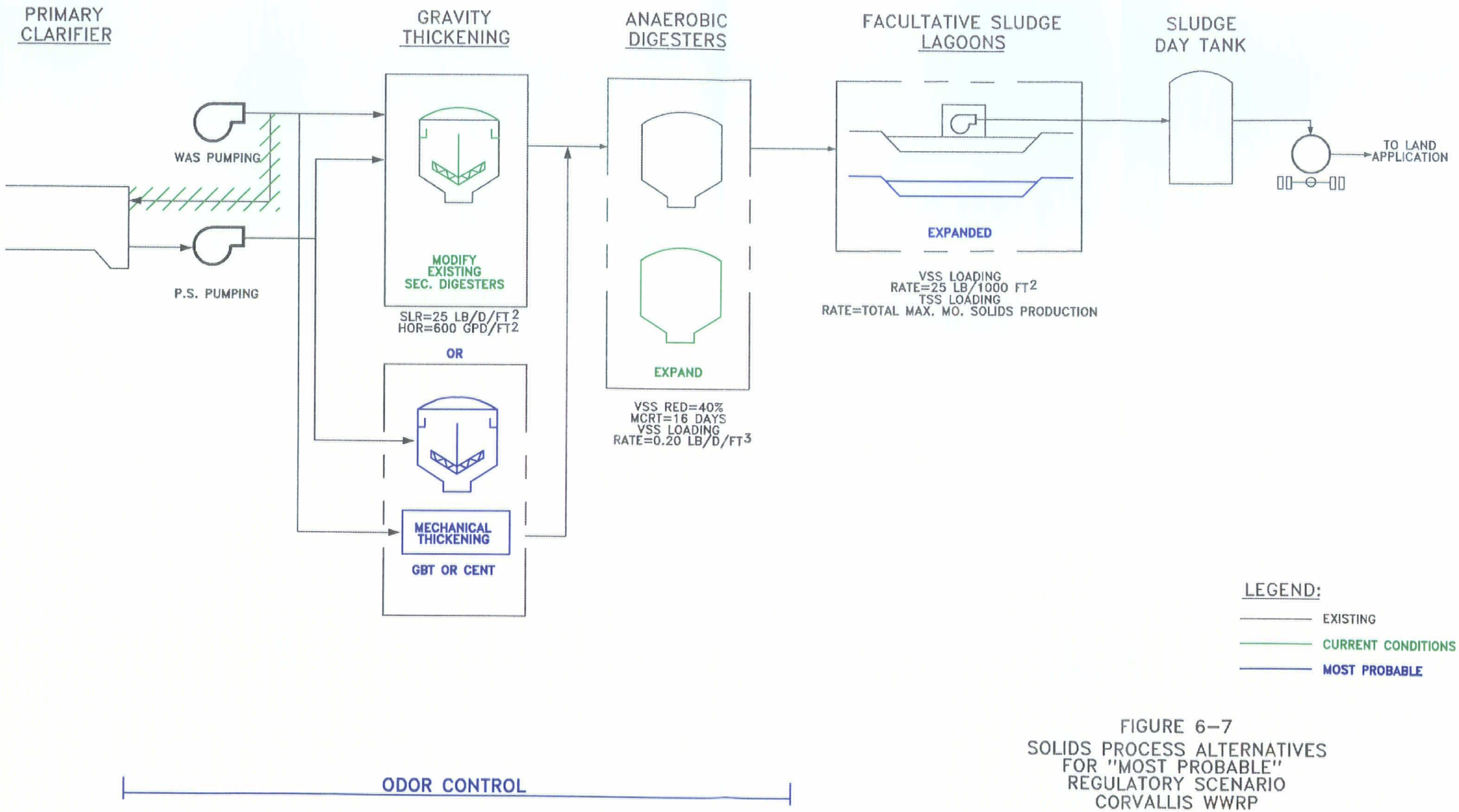

FIGURE 6-7
SOLIDS PROCESS ALTERNATIVES
FOR "MOST PROBABLE"
REGULATORY SCENARIO
CORVALLIS WWRP

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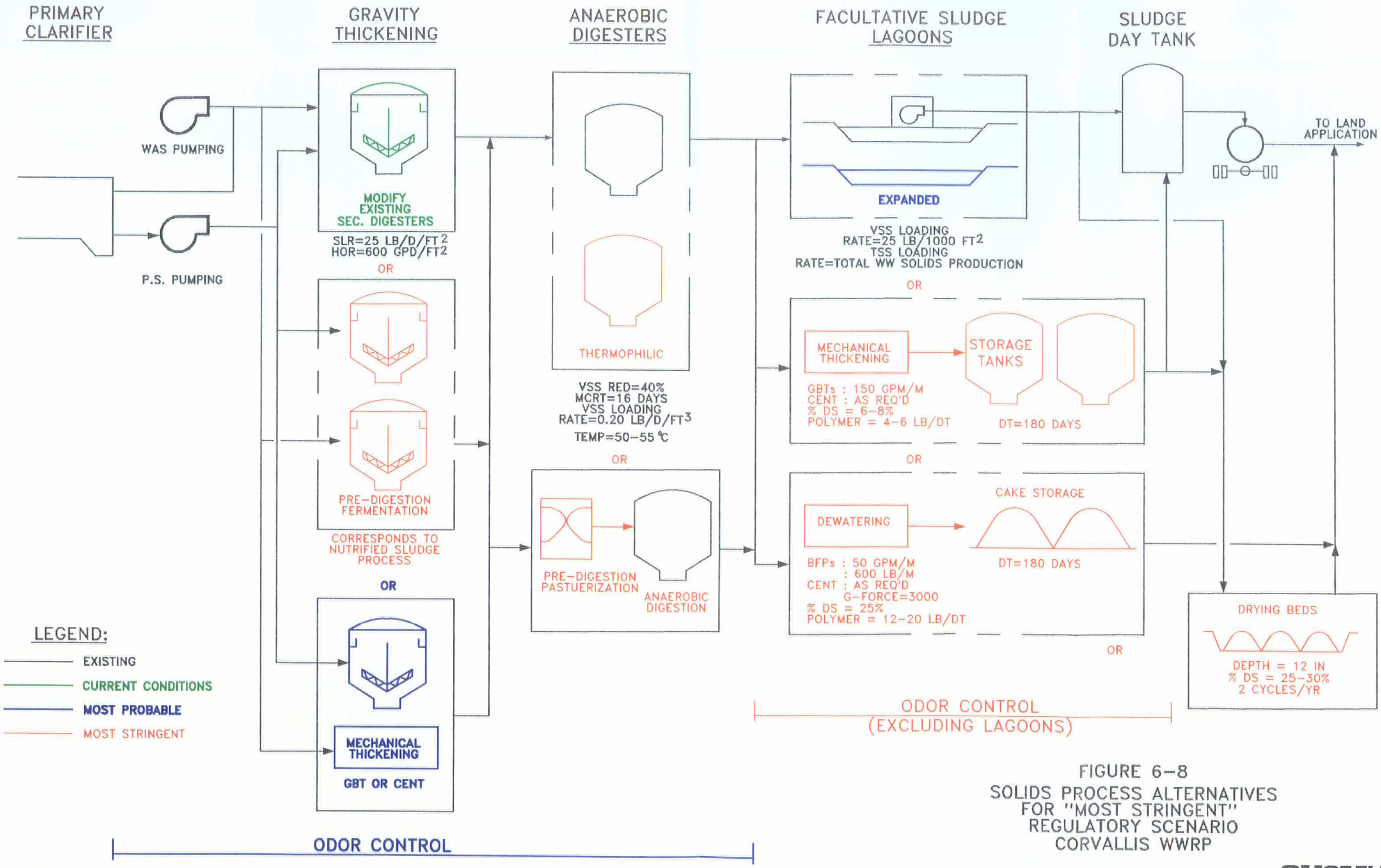


FIGURE 6-8
SOLIDS PROCESS ALTERNATIVES FOR "MOST STRINGENT" REGULATORY SCENARIO
CORVALLIS WWRP

CH2MHILL

TABLE 6-16
Sludge Characteristics and Mass Balance

Item	Entering Digester	Leaving Digester	Leaving FSL
Sludge Characteristics			
Total solids (TS) concentration	4 to 5 percent TS	2 to 2.5 percent TS	4 to 6 percent TS
Volatile solids concentration	78 percent of TS	58 percent of TS	49 percent of TS
Nitrogen content of the solids	NA	NA	5.4 percent of TS
Sludge Quantities			
Current daily average	9,270 lb/day	6,390 lb/day	5,920 lb/day
Current maximum month	14,180 lb/day	7,907 lb/day	32,862 lb/day
Design daily average	12,100 lb/day	7,000 lb/day	NA
Design maximum month	21,600 lb/day	13,000 lb/day	NA

TABLE 6-17
Sludge Thickening Screening Evaluation

Criteria	Alternative Ranking	
	No. 1	No. 2
Description	Gravity Co-Thickening	Gravity thicken primary sludge/mechanically thicken WAS
Capital Cost Comparison	**$2,120,000**	**$3,530,000**
Qualitative Comparison		
Operating Cost	5	3
System Reliability	5	4
Impacts to Facility Operation	5	3
Impacts to Facility Maintenance	4	3
Operational Complexity	5	3
Staffing Requirements	5	3
Ease of Construction	4	4
Adaptability to Changing Regs.	3	5
Adapability to Existing Site	3	3
Mitigation of Adjacency Issues	2	4
Overall Ranking	41	35

Thermophilic Anaerobic Digestion. In this treatment process the anaerobic digesters are operated at thermophilic temperatures (i.e., greater than 50 degree C). The basic modifications would include:

- Sludge-to-sludge heat exchangers for the recovery of heat in the digester effluent stream to preheat the digester feed sludge.
- Heating system modifications to provide adequate capacity to achieve the elevated operating temperatures (the existing digesters typically operate at about 35 degrees C).
- Digester feed modifications to allow for batch operation of two (or more) digesters.

Pre-digestion Pasteurization. In this Class A treatment process, an initial pasteurization step would be provided prior to standard mesophilic anaerobic digestion. Typically, this would consist of a reactor vessel(s) upstream of the existing anaerobic digesters. This reactor would be operated at temperatures greater than 60 degrees C and would have a detention time corresponding to the time-temperature relationship given in 40 CFR Part 503 (sludge) regulations. The initial reactor vessel can be purely a pasteurization step (1 hour at 70 degrees C), an aerobic treatment step to begin hydrolysis (aerobic pretreatment—2 to 4 days at 60 degrees C), or it can be an anaerobic treatment phase (temperature phased digestion—2 days at 55 to 60 degrees C).

Therefore, anaerobic digestion was selected as the sludge stabilization process for master planning purposes. Should Class A biosolids be required in the future to address regulatory requirements or a policy decision, it is anticipated that one of the two Class A processes would be adapted to the anaerobic digestion system.

A Class A process selection has not been made as a part of this analysis because both of these process approaches are relatively new in this country (less than ten total operating facilities). It is felt that selection of the specific process would benefit from deferring this decision. A longer development period will provide a greater base of knowledge evaluating specific process performance and operating requirements.

Biosolids Storage

Alternative No. 2, Post-digestion Thickening with Enclosed Concrete Storage Tanks, was eliminated from further consideration because of the high cost. Thus, the following alternatives were selected for detailed evaluations:

Alternative No. 1—Facultative sludge lagoons

Alternative No. 3—Covered storage pad for dewatered cake

Table 6-18 summarizes the screening analysis.

Biosolids Dewatering

The biosolids dewatering alternatives were similar in cost and in scoring on the qualitative comparison. However, the most significant differentiation was the significant land requirement of the drying beds in Alternative No. 1. It was felt that the 13 acres required for siting a drying bed system, the poor drying conditions that exist in the Willamette Valley, and the significant odor generation potential associated with a large air drying facility made this

alternative unfeasible. Therefore, Alternative No. 2—Mechanical Dewatering was selected for further detailed alternative analysis. Table 6-19 summarizes the screening analysis.

TABLE 6-18
Sludge Storage Screening Evaluation

Criteria	Alternative Ranking		
	No. 1	No. 2	No. 3
Description	Facultative Lagoons	Post-digestion thickening w/ enclosed storage tanks	Covered storage pad for dewatered cake
Capital Cost Comparison	**$3,000,000**	**$13,200,000**	**$1,260,000**
Qualitative Comparison			
Operating Cost	3	1	4
System Reliability	4	2	5
Impacts to Facility Operation	5	2	3
Impacts to Facility Maintenance	4	1	5
Operational Complexity	4	2	3
Staffing Requirements	3	1	3
Ease of Construction	3	1	2
Adaptability to Changing Regs.	3	4	4
Adapability to Existing Site	2	3	4
Mitigation of Adjacency Issues	4	3	4
Overall Ranking	35	20	37

TABLE 6-19
Sludge Dewatering Screening Evaluation

Criteria	Alternative Ranking	
	No. 1	No. 2
Description	Drying Beds	Mechanical dewatering
Capital Cost Comparison	**$6,310,000**	**$5,860,000**
Qualitative Comparison		
Operating Cost	4	2
System Reliability	5	3
Impacts to Facility Operation	4	3
Impacts to Facility Maintenance	4	2
Operational Complexity	5	3
Staffing Requirements	1	3
Ease of Construction	2	4
Adaptability to Changing Regs.	3	4
Adapability to Existing Site	1	4
Mitigation of Adjacency Issues	1	4
Overall Ranking	30	32

Detailed Evaluation of Biosolids Management Alternatives

Following the screening evaluation, two treatment alternatives warranted detailed alternative analysis. However, these individual processes were more a function of the overall approach to biosolids management. Therefore, the following biosolids management alternatives were subjected to detailed analysis:

Alternative No. 1—Liquid Biosolids Hauling and Land Application

Alternative No. 2—Hauling and Land Application of Mechanically-Dewatered Biosolids

Each of these alternatives assumed that anaerobic digestion would precede the specified biosolids management alternative.

Criteria

The sizing and performance criteria used in the alternative analysis consisted of the following:

- Annual Solids Production
  - For sizing of facilities: 2580 dry tons/year
  - For calculation of O&M costs: 1930 dry tons/year
- Haul distance: 16 miles round-trip
- Solids concentration of trucked biosolids
  - Liquid land application: 5 percent dry solids
  - Dewatered land application: 22 percent dry solids

Alternative No. 1—Liquid Biosolids Hauling and Land Application

The proposed improvements to the existing liquid hauling and land application program were as follows:

- Provide two new facultative sludge lagoons totaling 4.4 acres or 2.6 million gallons. A new mudcat dredge would be provided to increase the withdrawal rate to 1000 gpm.
- Replace the existing truck loading pump and day tank with two new truck loading pumps and day tanks. These pieces of equipment would have the following design characteristics:

Truck Loading Pumps

Type	Screw-induced flow centrifugal
Capacity	1,000 gpm
Head	20 ft TDH
Horsepower	20 hp

Truck Loading Day Tanks

Type	Vertical steel tank
Capacity	50,000 gallons

Advantages. The advantages are as follows:

- Maximizes use of existing installed equipment and facilities
- Lowest capital cost alternative
- Continues a successful program that is familiar to operations staff and the farming community

Disadvantages. The disadvantages are as follows:

- Relatively high annual O&M costs caused by trucking the high-volume liquid biosolids
- Large land area required for facultative sludge lagoons; expansion beyond 60,000-population level will create a significant site impact.
- Higher volume of truck traffic to haul liquid biosolids
- Land application of liquid biosolids is somewhat inhibited in early season because of limitations on getting tanker trucks on wet fields; use of a big gun sprinkler for application will mitigate this inhibition.

Alternative No. 2—Hauling and Land Application of Mechanically-Dewatered Biosolids

This alternative would consist of a new dewatering facility to mechanically-dewater biosolids prior to land application, a storage pad for storing dewatered biosolids in lieu of the facultative lagoons, and the appropriate hauling and land application equipment. Development of this alternative would include the following improvements:

- Provide a new building to house the dewatering equipment, polymer feed system, sludge feed pumping, and dewatered biosolids conveyance system. The equipment to be provided will include the following:
  - Belt Filter Presses

Quantity	2
Size	2 meter units
Capacity	1200 dry lb/hr; 100 gpm

  - BFP Feed Pumps

Quantity	2
Type	Progressing cavity
Capacity	10 to 100 gpm

  - Polymer Preparation System

Quantity	1
Type	Liquid Mix/Dilution System
Capacity	50 lb/hr

- Polymer Feed/Transfer Pumps

  Quantity 4 (2 transfer, 2 feed)
  Type Progressing cavity w/ AFD
  Capacity 2 to 12 gpm

- Polymer Storage Tank

  Quantity 1
  Type Vertical FRP
  Capacity 6,000 gallons

- Provide a 35,300 square foot asphalt storage slab capable of providing 180 days of dewatered biosolids storage. The pad would be covered with a pre-engineered metal roof structure (no walls) to eliminate stormwater collection concerns and provide wet weather protection.

- The following equipment would be required for handling, hauling and land application of the dewatered material:

  - One additional front-end loader
  - Two 20-cubic-yard dump trailers to be used with the existing trucks
  - One manure spreader and tracker

This alternative's relative advantages and disadvantages are summarized below.

Advantages. The advantages are as follows:

- Lower annual O&M cost because of the reduction in biosolids to be hauled and land applied.

- Dewatered biosolids provide a longer land application window because application equipment typically can get onto the fields earlier than is possible with liquid biosolids.

- Much smaller site required for biosolids storage during non-application season.

- Lower potential for environmental adjacency impacts because of the reduction in truck traffic and lower odor generation potential.

Disadvantages. The disadvantages are as follows:

- Most costly alternative; higher in both capital and life cycle costs.

- Increased complexity in the solids handling system requiring greater operator training and sophistication.

- Creates a plant recycle stream that is high in soluble ammonia; a potentially negative impact if the plant has a future ammonia discharge limit.

Evaluation Results

A comparison of the costs for the two alternatives is given in Table 6-20. This summary shows a comparison of capital, annual O&M, and life cycle present worth costs for each

alternative. From this analysis, Alternative No. 1 is the lowest cost alternative both in terms of capital expenditures and overall life cycle costs.

TABLE 6-20
Solids Handling Detailed Alternatives Evaluation Cost Summary

No.	Alternative	Capital Cost	Annual Cost	Present Worth Cost
1	Liquid sludge hauling to cooperative application sites	$3,644,196	$428,020	$8,759,194
2	Mechanical dewatering w/ hauling to cooperative application sites	7,226,667	233,693	10,019,388

Note: See Appendix E for the details of the present worth cost analysis.

In addition, the following observations were made:

1. There is a significant amount of land available for application of biosolids within the current haul distance, in particular the land adjacent to the airport, making it unlikely that an increase in haul distance will occur in the future that would change the results of the cost analysis.
2. The current biosolids management program is operated at a very low cost (approximately $80 per dry ton), is familiar to the operations staff and the farmers, and is easily managed. Therefore, a driver that would significantly increase costs or the management complexity would need to occur to justify a change in this program.
3. Site restrictions will significantly impact the ability to expand the facultative sludge lagoons. This could mean relocation of these lagoons to an off-site location or moving the Public Works facilities to a new location to provide adequate land area for the facultative sludge lagoons. The cost of additional land or relocation of Public Works has not been included in this analysis.
4. Odor generation and/or visual impacts from the facultative sludge lagoons, concerns over the volume of truck traffic, run-off or aerosol concerns with liquid application, and the use of off-site storage facilities are among the factors that could support the need to move away from liquid biosolids hauling and application.

Alternative Selection

Based on this evaluation, Alternative No. 1 was selected as the biosolids management alternative for development during implementation planning of the future WWRP improvements. However, Alternative No. 2 was retained as a biosolids management approach to address the most stringent planning criteria because of the potential for factors other than regulatory compliance to impact the viability of liquid biosolids application.

Digester Gas Utilization

Introduction

The Corvallis WWRP currently produces methane gas as a by-product of the anaerobic digestion process used for sludge stabilization. This digester gas is then utilized as fuel for the boilers that provide heating for the anaerobic digester and non-process buildings on the WWRP site. Any excess digester gas that is produced and not utilized is burned in a waste gas burner.

A detailed analysis of digester gas utilization options was completed in January 1994 as part of the City of Corvallis Methane Utilization Evaluation. Therefore, this work was not repeated as a part of this master planning effort. However, a review of this study was conducted to ensure that its analyses were updated to current conditions and to incorporate the results of this report into the overall master plan.

Existing Digester Gas Production and Utilization

The existing digester gas utilization system consists of the following components:

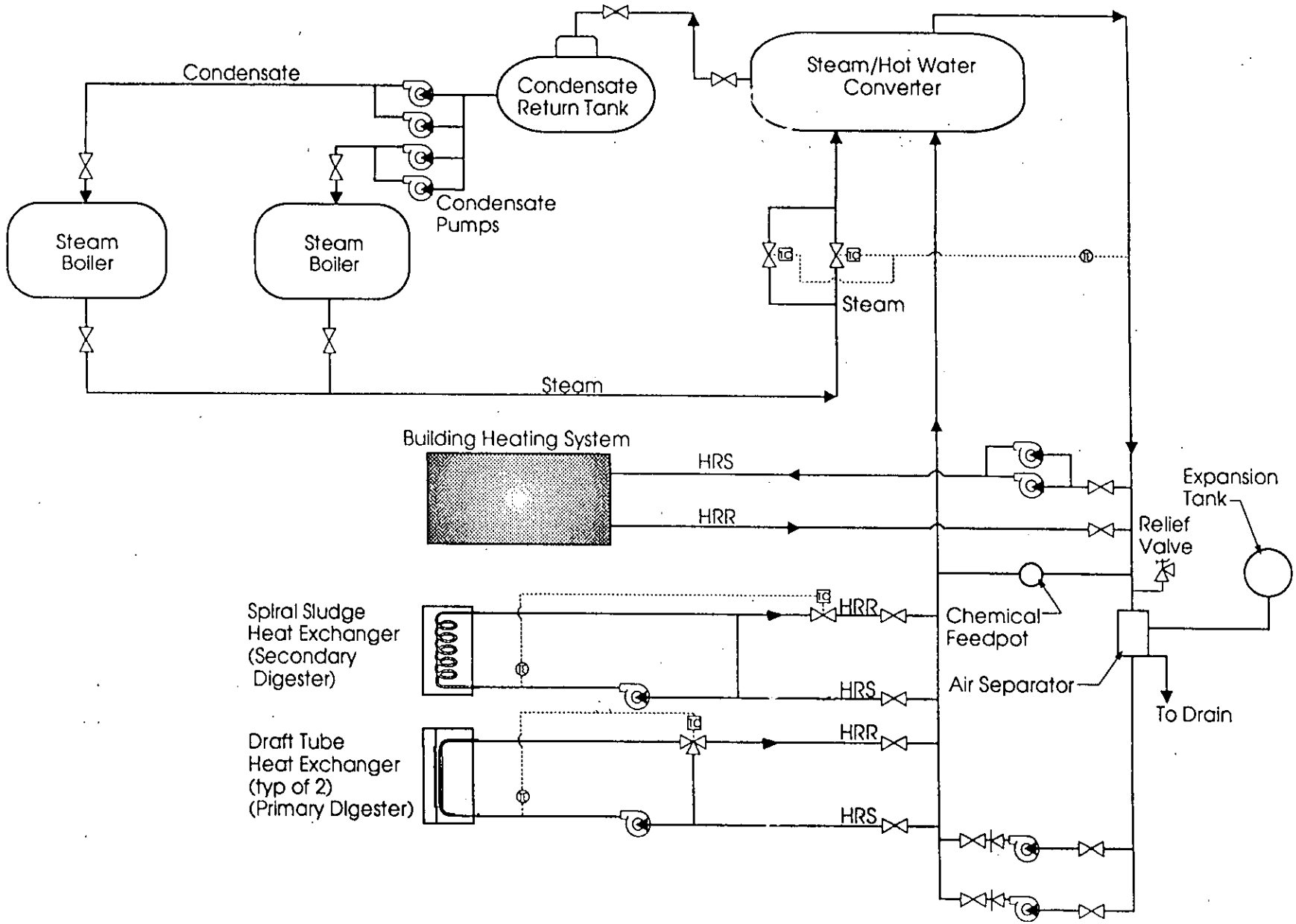
- A fixed digester cover that provides system pressurization control and limited gas storage.
- Two steam, fire-box boilers provided with dual-fuel (natural gas/digester gas) forced draft burners.
- A back-pressure-controlled waste gas burner for flaring the excess digester gas.

Other than gas metering equipment and moisture removal traps, the digester gas utilization system has no significant gas conditioning equipment.

The digestion process produces an average of about 65,100 cubic feet of digester gas per day. At a higher heating value of approximately 600 Btu per cubic foot, this equates to an average of 39 (10) 6 Btu per day of available digester gas energy. Looking at a 24-month time span for digester performance, this digester gas production rate represents an average of 14.8 cubic feet of gas for each pound of volatile solids destroyed (ft^3/lb VS). Typical values are from 13 to 18 ft^3/lb VS, indicating fairly typical digester performance.

All of the digester gas that is currently utilized provides heat to the anaerobic digester and the non-process buildings on the WWRP site. The existing heating system consists of two steam boilers that feed a steam-to-hot water converter. The hot water leaving this heat exchanger is circulated in a primary heating water loop by in-line centrifugal pumps. Heating hot water is drawn off this primary loop, as needed, by secondary heating water pumps that feed the draft tube heat exchangers in the digester, an external spiral heat exchanger, and the building heating system. A schematic of this heating water system is shown in Figure 6-9.

Currently, the heating system for the WWRP utilizes about half of the available digester gas. The remainder is flared in the waste gas burner. Reports from plant staff indicate that in colder periods and wet weather conditions, the digester gas production is inadequate to



BLACK & VEATCH
PROGRESS BY DESIGN

CORVALLIS, OREGON WWRP PLANT HEATING SCHEMATIC

FIGURE 6-9

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keep up with the instantaneous heating demands of the system. Thus, the use of supplemental natural gas is required for fueling the boilers.

Projected Digester Gas Production and Utilization

Figure 6-10 shows an average monthly comparison of the WWRP heating requirements and digester gas production that is anticipated at the 60,000-population level. This comparison incorporates the increases in gas production as a result of higher volatile solids loading, as well as an increase in the WWRP's heating requirements. The increase in the overall WWRP heating requirement includes both an increase in digester heating requirements, primarily due to higher feed rates, and non-process facility heating requirements, as a result of anticipated increases in the heated, non-process facility square footage.

Evaluation of Digester Gas Utilization Alternatives

Analysis of Previous Digester Gas Study

The *Methane Utilization Evaluation* looked at a total of seven options for utilizing surplus digester gas and performed detailed analysis of three potentially viable alternatives:

1. Cogeneration using internal combustion engine-generators
2. Gas engine-driven process equipment
3. Supply digester gas to the Public Works Department (PWD) boiler

The results of the economic analysis of these alternatives are summarized in Table 6-21.

TABLE 6-21
Summary of Gas Utilization Present Worth Costs

Costs	Option 1 Engine Generator	Option 2 Engine Driven Equipment	Option 3 Supply Gas to PWD Boiler
Capital Costs	$1,334,300	$703,100	$67,700
Annual Costs	187,400	124,500	8,600
Annual Savings	(699,500)	(320,700)	(47,100)
Total Present Worth Costs	822,200	506,900	29,200

A review of the alternatives evaluated and the analysis methodology suggests a sound analysis that looked at most of the viable digester gas utilization options currently being employed. The study recommended that Option 3 be implemented even though it did not provide a positive financial benefit. This recommendation was based on a strong desire by City staff to provide some beneficial use of a reusable resource. Figure 6-11 shows that utilization of digester gas in the PWD boiler will have a nominal impact on the amount of excess gas burned in the waste gas burner. In addition, this is a seasonal load that will utilize the most gas during parts of the year when digester heating requirements are the greatest. This increases the likelihood that supplemental natural gas will be required on the WWRP site or that digester gas will not be available for PWD heating. Note, however, that the proposed improvements to the thickening and digester feed systems should significantly reduce the occurrence of supplemental natural gas usage.

Figure 6-10
Comparison of Digester Gas Production to Current Utilization
(60,000 Population)

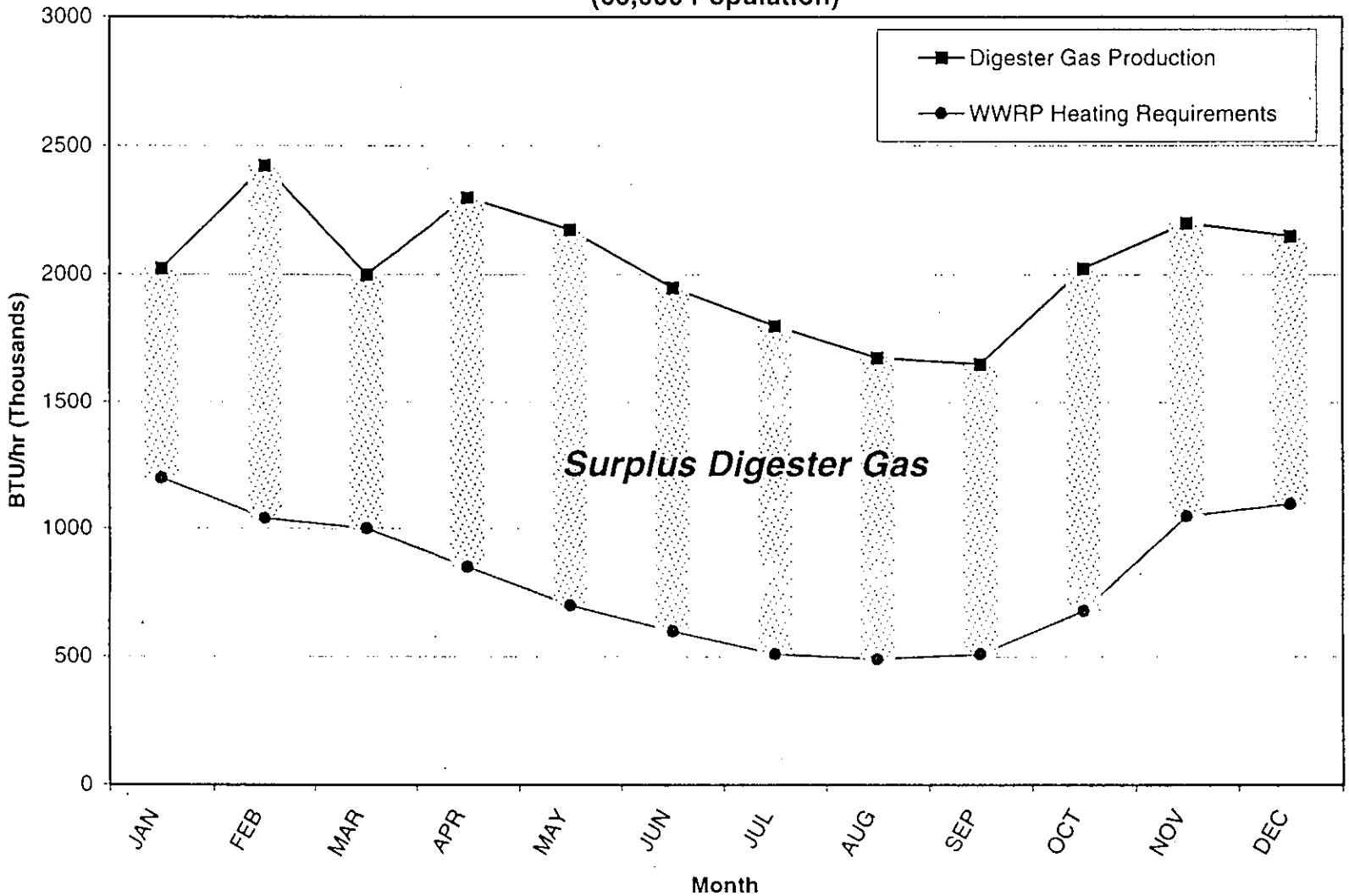
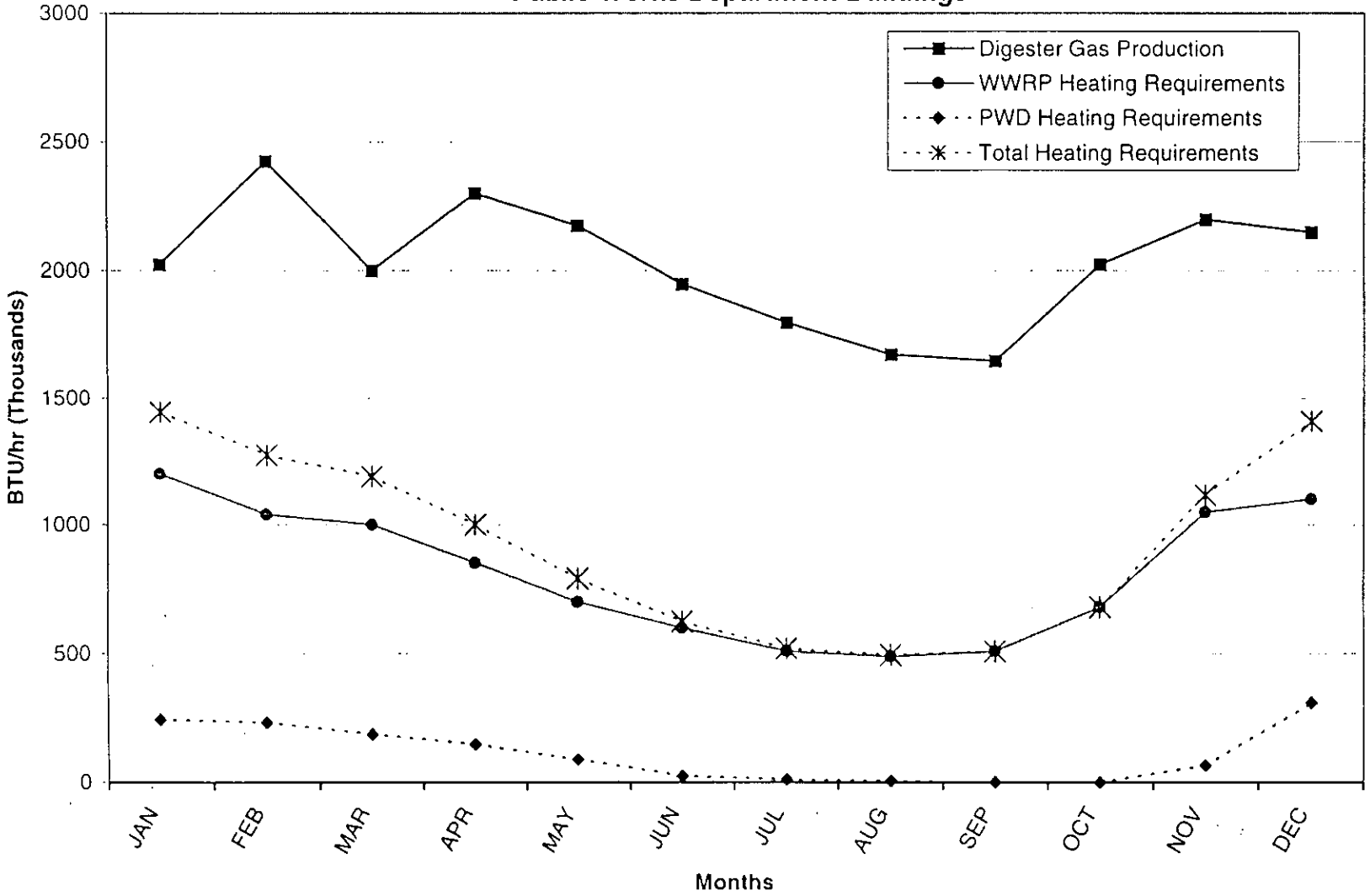


Figure 6-11
Comparison of Digester Gas Production to Heathing System Utilization for Both WWRP and Public Works Department Buildings



As a means of better utilizing the gas on an annual basis, an approach that uses a base load of digester gas is more appropriate. This allows the gas to be continuously utilized without the seasonal impacts of heating use. Both Options 1 and 2 above would provide this base load; however, the costs would be quite high and would not generate any net cost savings. An alternative approach would be to supply gas to a local industry that has a base load need for the gas, with the specific quantity being negotiable. For example, the City of Portland currently supplies digester gas to Malarkey Roofing for use in their process operation. The gas quantity supplied is fixed and minimal gas conditioning is required to meet their needs.

Development of Additional Digester Gas Utilization Alternatives

It would certainly be appropriate to incorporate utilization of digester gas into any new digester heating system requirements. In addition, other gas utilization options that should be considered during planning of future project are:

- Option No. 4—Installation of gas compression and conditioning systems for resale of excess gas to a local industry.
- Option No. 5—Utilization of the gas for thermal oxidation of odorous air streams.

Option 4—Resale of Excess Gas to a Local Utility. A potential off-site user for digester gas is Hewlett Packard. Their facility is relatively close and since they have both heating requirements and utilize steam for absorption cooling there is a year-round need for gas in their steam boilers. An analysis was conducted to determine the potential cost effectiveness of this option. This analysis was based on the following assumptions:

- A fairly high level of gas conditioning would be required consisting of hydrogen sulfide scrubbing and dehumidification.
- Approximately two-tenths of a cent per cubic foot could be obtained for the delivered gas (about one-half the cost of natural gas).
- Approximately 30,000 cubic feet per day of digester gas would be available for resale.
- The proposed gravity thickening and digester feed modifications will minimize or eliminate the shortfall in digester gas availability during the winter months.
- All costs to condition, compress, and transport the gas (including a digester gas pipeline) will be paid by the City. Hewlett Packard will pay for all modifications to their facilities.

It is unknown at this time what the potential revenue for the sale of this gas might be. However, based on the assumptions outlined above, the resulting present worth costs for this option would be about $1,290,000. This would not make a superior economic alternative to either Options 1 or 2.

Option 5—Thermal Oxidation of Odorous Air Streams. Although there are no immediate plans for major odor control improvements, this master plan identifies a long-term goal of collecting and treating the offgas from the major odorous air sources on the WWRP site: influent pump station, headworks, gravity thickening, and primary clarifiers. Treatment of these odors through thermal oxidation represents an opportunity for cost effective utilization of digester gas. Regenerative thermal oxidation (RTO) is an odor control technology that is less

costly than wet scrubbing technologies on a capital cost basis and is very cost-competitive on a life cycle basis when a waste fuel is available. It is also much more effective at removing complex organic odors from the airstream than activated carbon adsorption. Removal of more complex odorous compounds will be particularly important if pre-digestion fermentation is implemented; this is a very odorous process.

A detailed cost analysis has not been conducted because development of odorous air quantities is beyond the scope of this master plan. These costs would not be appropriate for comparison with other gas utilization options because there is no savings to assess their break-even point. The viability of this gas utilization option should be considered when assessing other odor control technologies.

Evaluation Results

Analyses included as part of this master plan and previous similar studies have shown that none of these utilization options will prove to be a cost-effective approach on its own merits. However, the environmental and community benefits of more complete combustion of the waste gas may make the costs palatable for the City. In any case, enhanced gas utilization is a potential building block for the future that should be addressed in the design of future facilities.

By reselling gas to a local industry, a baseload can be established that is matched to the available waste gas year-round. This "baseloading" is the key to minimizing the amount of waste gas that must be burned in the flare. It is also a means of providing gas utilization without the major expense of constructing and maintaining costly utilization facilities. However, this approach requires the plant to coordinate with an outside industry on gas conditioning requirements and delivery terms and conditions. It also requires the City to focus on serving an industrial customer's gas supply needs instead of the primary operational goal of wastewater treatment.

Environmental Impacts

Although none of the proposed utilization options will be cost effective, it is recognized that there is an environmental benefit to utilizing the surplus digester gas in combustion equipment rather than flaring it through a waste gas burner. One advantage to utilization is the ability to offset the use of fuel gas with a waste by-product.

Another advantage to enhanced gas utilization is the reduction in impacts to air quality. Corvallis is in an attainment area for carbon monoxide, carbon dioxide, and particulate. This means that there are no regulatory drivers to lower air emissions. However, the carbon monoxide generation from a waste gas flare is approximately 130 times greater than for a steam or hot water boiler. Therefore, these impacts should be considered, along with the economic viability, when evaluating utilization options. By way of comparison, the current waste gas burner operation generates carbon monoxide (approximately 10,350 pounds per year) that is equivalent to about 17 typical automobiles (ref: EPA 420-F-97-037). This means that although the impacts to air quality are not insignificant, they are a relatively minor component in the airshed.

Recommendations

The following recommendations are presented for future consideration of gas utilization:

1. It is recommended that the initial concept of utilizing the digester gas in the PWD boiler be considered in any future upgrades to the digestion system and/or modifications to the PWD heating system. It should be recognized that this option provides a minimal impact to the amount of digester gas to be flared.
2. The use of digester gas for thermal oxidation of odorous air might prove to be a cost effective method of utilizing gas when compared to other odor control technologies. This alternative should be considered when assessing odorous air treatment technologies as a part of any future odor control projects.

Environmental Adjacency Improvements

Environmental adjacency improvements are those projects that mitigate the plant's impact on the surrounding community. Environmental adjacency impacts include:

- Odors
- Traffic
- Noise
- Safety
- Aesthetics

Addressing these impacts can either be an integral part of process improvements or separate supplemental improvements. Outlined below are the issues to be addressed at the Corvallis WWRP in each of these areas and recommendations for the measures to be taken. As with the process improvements, the most probable and most stringent planning criteria will be utilized. Because these are community-driven issues, feedback from the surrounding community will be used to assess the level of mitigation.

Odors

Odors can be the most profound impact that a wastewater treatment plant has on the surrounding community. Therefore, it is the most likely area where improvements will be required. At the Corvallis WWRP there are two classifications of odor generation potential. These are primary odor sources and secondary odor sources. The primary odor sources are:

- Headworks
- Aerated grit chambers
- Primary clarifiers
- Gravity thickening
- Facultative sludge lagoons

The secondary odor sources, defined as those areas where odor potential exists but is low or intermittent, are:

- Trickling filters
- Digestion facilities

The most probable planning scenario would consist of mitigating the odor generation from the primary odor sources, whereas the most stringent scenario would include control of the secondary sources as well. Odor control measures for the headworks, aerated grit chambers,

primary clarifiers, and gravity thickeners (as previously discussed) would consist of covers for the exposed water surfaces, then ventilating and treating the air from the resulting headspace. In addition, a covered area and odorous air treatment system would likely be provided for the screenings area at the headworks.

It is anticipated that odor control for the facultative sludge lagoons will be the continued practice of chlorinating the water cap on the lagoons. This practice has proven successful over the last few years, and with the additional stabilization capability provided by new anaerobic digesters should work well into the future.

Trickling filter odor control would consist of a new enclosure for the trickling filter and odorous air ventilation and treatment. The predominant source of odors from an anaerobic digester is the gas pressure control valves. Mitigation typically consists of valve enclosures and an air treatment system.

Odor air treatment systems should be selected at the time that these measures are implemented to ensure that the most effective technology is selected. However, given current technology and the available space on the site, it is likely that some type of wet scrubbing technology will be utilized for the larger airflows. Biofiltration and carbon adsorption might be used for some of the smaller airflows (e.g., gravity thickeners).

Traffic

The most significant traffic impact is the biosolids hauling operation during the summer months. This consists of several trucks per day entering and leaving the plant site. The only effective mitigation for this impact would be a change from liquid sludge hauling to sludge dewatering and hauling. This has been identified as a most stringent planning scenario.

Noise

The Corvallis WWRP is generally a low noise plant because of its use of trickling filters and few pieces of noise-generating equipment. No mitigation measures are anticipated.

Safety

The most significant safety issue surrounding the plant was the use gaseous chlorine. In response to the potential impact to the community, the City has made a policy decision to utilize sodium hypochlorite for disinfection of the wastewater effluent.

Aesthetics

The greatest aesthetic impact of the plant is the view from Highway 99W. Because of the highway's elevated position directly adjacent to the plant, passersby have a clear view of the entire plant site. Most of the surrounding neighbors have no direct line-of-site view of the plant. It is recommended that plantings and landscape features be used to provide some visual mitigation if the community begins to express concern over the visibility of the plant. It is worth noting that visual mitigation can be one of the most effective measures for mitigating odor concerns. Experience at other facilities has shown that when the public cannot readily see the facility, there tends to be less concern about odors.

Implementation of the Selected Treatment Alternatives

After selecting the treatment alternatives for the liquid treatment process and the solids handling system, these alternatives were refined and improvements were identified to serve populations of 60,000, 80,000, and buildout. Improvements were defined for both the "most probable" planning scenario and the "most stringent." The improvements necessary to meet the current conditions are included in the most probable scenario and are identified separately on the proposed site layouts. However, maintaining current conditions alone was deemed to be too optimistic a scenario for financial planning purposes.

More detailed development was conducted for the most probable planning scenario because this scenario represents the criteria that will most likely apply to the treatment facilities as a whole. Therefore, it is the scenario on which a planning "road map" should be developed, a road map by which the facility improvements would be sited and constructed to serve a population of 60,000, 80,000, or probable buildout (120,000) and/or respond to more stringent regulatory requirements. The most stringent scenario could apply in whole or in part. Therefore, the incremental improvements to meet the more stringent criteria have been identified so that the planning road map can be modified in the future to meet these changing conditions. The improvements for these two planning scenarios are outlined below.

Most Probable Planning Scenario

Table 6-22 shows the sizing of the proposed improvements for the "most probable" regulatory scenario. In general, the facility improvements reflect a need for hydraulic capacity and solids handling improvements in the near term with secondary treatment capacity improvements being required later in the planning period. In particular, the capacity improvements needed in the next 10- to 12-year timeframe are:

- Primary clarifiers
- Secondary clarifiers
- Chlorine contact basin
- Solids handling
- Odor control

Figure 6-12 shows a site layout of the proposed improvements. The facilities required to serve population levels of 80,000 are shown with solid lines, while the additional facilities projected for buildout conditions are shown as dashed lines. The green facilities represent the improvements necessary if current regulatory conditions are maintained. The blue facilities represent the additional improvements necessary if the most probable planning criteria are applicable.

The total project costs to implement these improvements are summarized in Table 6-23. The costs included as existing improvements are for those upgrades necessary to address near-term capacity shortfalls. The 60,000 and 80,000 project costs are for the improvements necessary to serve up through those population levels.

Table 6-22
Sizing of "Most Probable" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		Buildout		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Influent Pumping/Headworks									
Influent Pumps	(1) 7 mgd plus (3) 14 mgd	28	None	None	22	(4) 16.7 mgd	14	(5) 16.7 mgd	mgd (WWPH)
Grit Removal									
Aerated Grit Tanks	(1) 8,460 cu.ft., (1) 11,600 cu.ft.	11.9	11.3	(2) 11,600 cu.ft.	7.8	(1) 11,600 cu.ft.	15.5	(3) 11,600 cu.ft.	mgd (WWPD)
Grit Pumps	(2) 180 gpm, (2) 250 gpm	250		(4) 250 gpm		(2) 250 gpm		(6) 250 gpm	gpm (WWPD)
Grit Cyclones	(1) 250 gpm	250		(2) 250 gpm		(1) 250 gpm		(3) 250 gpm	gpm (WWPD)
Grit Classifiers	(1) 54 cu.ft./hr	54		(1) 54 cu.ft./hr		(1) 54 cu.ft./hr		(1) 54 cu.ft./hr	cu.ft./hr
Primary Sedimentation									
Rectangular tanks	(1) 136 ft.x 31'x 9.5' (1) 136 ft.x 42'x 9.5'	5.06 6.86	11.34	(2) 136 ft.x 42'x 9.5'	7.76	(1) 136 ft.x 42'x 9.5'	15.51	(3) 136 ft.x 42'x 9.5'	mgd (WWMM) mgd (WWMM)
Primary Sludge Pumps	(2) 200 gpm	200		(2) 200 gpm		(1) 200 gpm		(3) 200 gpm	gpm (WWPD)
Primary Scum Pumps	(1) 100 gpm	not used		none		none		none	gpm (WWPD)
Trickling Filters									
Trickling Filters	(2) 160 ft dia, 8 ft deep	18,343	None	None	2,000	(1) 160 ft dia, 8 ft deep	10,884	Increase depth of media 4 ft	lb BOD/d (WWMM)
Circulation Pumps	(2) 11.5 mgd	23.0	33.0	(2) 28.0 mgd (replace existing)	44.0	(2) 28.0 mgd	16.0	(1) 28.0 mgd	mgd (WWPH)

Table 6-22
Sizing of "Most Probable" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		Buildout		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Aeration Tanks									
Aeration Tanks	(2) 156'x40'x15'	33.6	None	None	None	None	None	None	mgd (WWMM)
Aeration Blowers	(2) 1,400 cfm, (2) 5,900 cfm	14,600	None	None	None	None	None	None	cfm
Secondary Clarification									
Secondary Clarifiers	(2) 115' dia, 18' SWD	12.6	10.7	(2) 115' dia, 18' SWD	6.1	(1) 115' dia, 18' SWD	13.3	(3) 115' dia, 18' SWD	mgd (WWMM)
Return Activated Sludge (RAS) Pumping	(4) 2,000 gpm	8,000	8,000	(4) 2,000 gpm	4,000	(2) 2,000 gpm	12,000	(6) 2,000 gpm	gpm (WWMM)
Waste Activated Sludge (WAS) Pumping	(1) 100 gpm, (1) 160 gpm	260	None	None	60	(1) 160 gpm (replace exist. small pump)	200	(1) 200 gpm	gpm (WWMM)

Table 6-22
Sizing of "Most Probable" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		Buildout		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Disinfection									
Chlorine Contact Tank	(1) 110' dia, 10.5' SWD	28.0	50 (replace existing)	(2 basins) 92'w.x 100'l.x 15'd	None	None	14	(1 basins) 26'w.x 100'l.x 15'd	mgd (WWPH)
Sodium Hypochlorite Tanks	(6) 6,500 gal	40.0	None	None	10.0	(1) 6,500 gal	7.5	(1) 6,500 gal	mgd (WWPD)
Sodium Hypochlorite Metering Pumps	(4) 30 gph	28.1	None	None	21.9	(2) 70 gph (replace 2 exist. pumps)	17.1	(2) 70 gph (replace 2 exist. pumps)	mgd (WWPH)
Sodium Bisulfate Tanks	(2) 6,500 gal	60.0	None	None	None	None	None	None	mgd (WWPD)
Sodium Bisulfate Metering Pumps	(2) 12 gph	28.0	None	None	22.0	(1) 20 gph (replace 1 exist. pump)	14.0	(1) 20 gph (replace 1 exist. pump)	mgd (WWPH)
Filtration									
Deep Bed Filter	None	None	None	None	20.1	(5) cells @ 20' by 30' per cell	10.1	(2) cells @ 20' by 30' per cell	mgd (DWMM)
Backwash Pumps	None	None	None	None		(2) 7,200 gpm each		None	mgd (DWMM)

Table 6-22
Sizing of "Most Probable" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		Buildout		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Effluent Pumping									
Effluent Pumps	(2) 28 mgd	28	None	None	50	(4) 16.7 mgd	14	(1) 16.7 mgd	mgd (WWPH)
Sludge Thickening									
Gravity Thickeners	None	None	(2) 40' dia, 31' SWD (secondary digesters)	62,000	None	None	None	None	lb/d (PSL+WAS WWMM)
Sludge Digestion									
Anaerobic Digester	(1) 66' dia, 31' SWD, 795,000 gallons	16,500	2,000	(1) 50' dia, 50' SWD, 800,000 gallons	16,680	(1) 50' dia, 50' SWD, 800,000 gallons (to replace existing)	18,640	(1) 50' dia, 50' SWD, 800,000 gallons	lb/d (PSL+WAS WWMM)
Facultative Sludge Lagoon	(2) 2.2 acres each	25	None	None		(1) 2.2 acres		(1) 2.2 acres, (1) 3.3 acres	lb VSS/ 1,000 sq. ft./ day
Land Application Program									
Land		3000	None	None	None	None	None	None	acres
Sludge storage Tanks	(1) 50,000 gal.		None	None		(1) 50,000 gal.	None	None	
Sludge Pumps	(1) 750 gpm at 50 ft TDH, 50 hp		None	None		(1) 750 gpm at 50 ft TDH, 50 hp	None	None	

Table 6-23
Cost Estimates for "Most Probable" Process Improvements

	Total Project Costs [a]		
Unit Process	**Short-Term CIP**	**60,000 Population**	**80,000 Population**
Influent Pumping/Headworks			$2,606,200
Primary Treatment		$3,000,000	$1,687,500
Biological Treatment		$406,200	$3,662,500
Secondary Clarification		$4,237,500	$2,250,000
Disinfection		$2,380,000	
Filtration			$10,856,300
Effluent Pumping			$1,900,000
Solids Handling	$4,150,000		$4,300,000
Odor Control		$968,800	$150,000
Non Process Facilities[b]	$179,100	$221,000	$187,500
Total Project Cost	**$4,329,100**	**$11,213,500**	**$27,600,000**

[a] Cost estimates include 25 percent construction contingency and 25 percent allowance for engineering, legal, and administrative costs. They do not include seismic improvements. Cost estimates are in 1998 dollars indexed to the *Engineering News Record* construction cost index of 6,500 for the Seattle area.
[b] Presented in Section 8.

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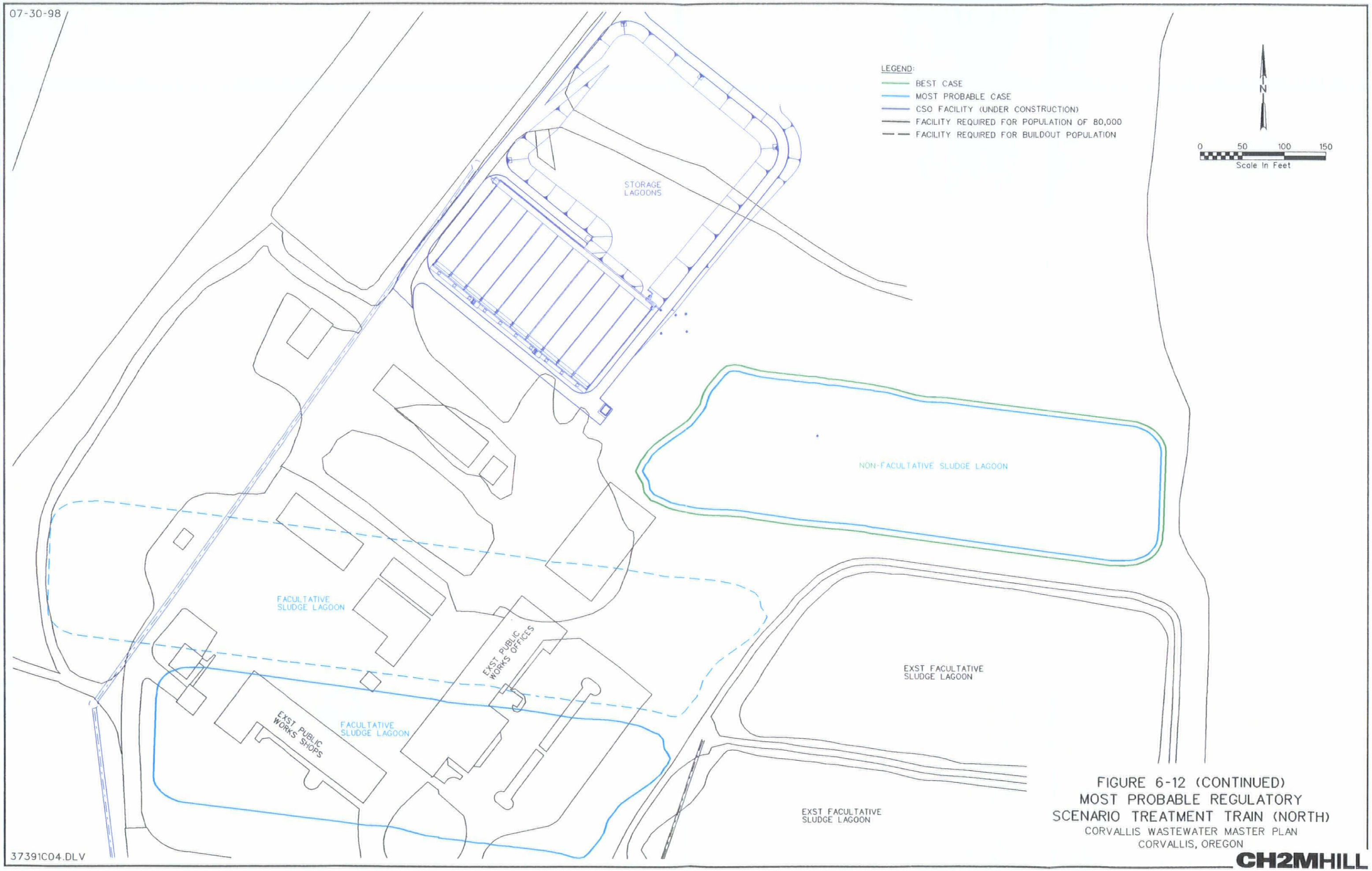

FIGURE 6-12 (CONTINUED)
MOST PROBABLE REGULATORY
SCENARIO TREATMENT TRAIN (NORTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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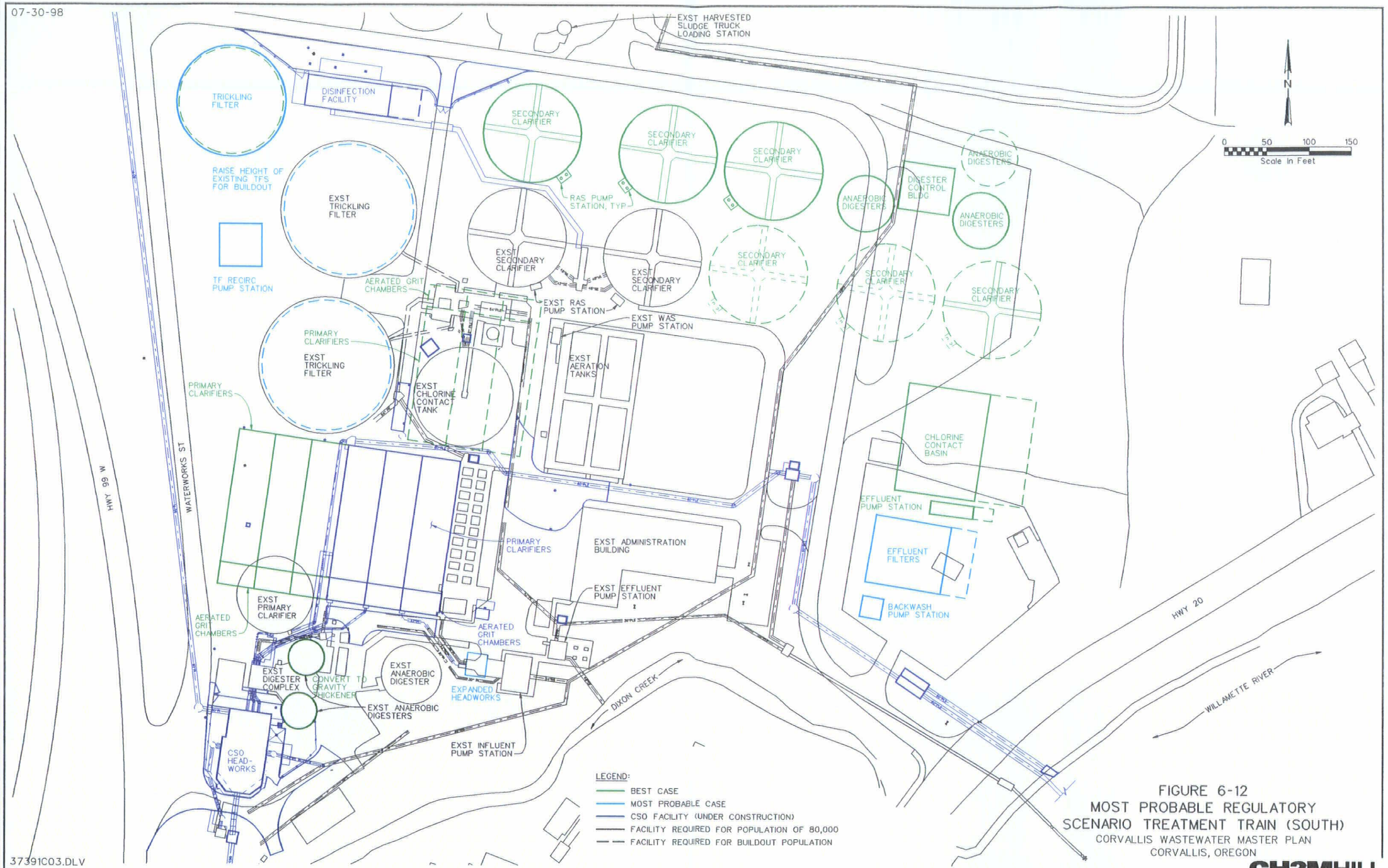

FIGURE 6-12
MOST PROBABLE REGULATORY SCENARIO TREATMENT TRAIN (SOUTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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Most Stringent Planning Scenario

Table 6-24 provides sizing for the proposed improvements to meet the "most stringent" planning criteria. Since it is not anticipated that any of the most stringent criteria would be applicable in the planning period that precedes the 60,000 population level, these improvements are shown for 80,000 and buildout conditions only. Figure 6-13 shows a site layout with all the most stringent improvements incorporated. As indicated above, these criteria are incremental, so the associated improvements may be necessary in whole or in part. For example, the improvements necessary to meet effluent nutrient limits may be necessary, while Class A biosolids treatment may not. Therefore, it is not envisioned that the WWRP treatment alternatives would be required to meet all of the most stringent criteria. Order of magnitude cost estimates have been summarized in Table 6-25 for each of the incremental improvements.

In most cases the improvements necessary to meet the most stringent criteria are add-on facilities that utilize existing treatment processes. For some unit processes, a general approach has been identified. In the future it will be necessary to define the specific treatment processes to use. A brief discussion of the unit processes that will be affected by the most stringent planning scenario is provided below.

Secondary Treatment. This unit process represents the facilities that could be the most drastically affected by a change in planning criteria. Under the most probable regulatory scenario (summer nitrification), expansion of trickling filter capacity with no increase in aeration basin capacity is the selected alternative. To provide summer biological nutrient removal, identified as the most stringent regulatory planning criteria, it is recommended that the plant convert to all suspended growth treatment (aeration basins). This suggests that analysis will be required in the future when planning for secondary treatment upgrades.

As planning begins in the future to address secondary treatment capacity needs, the regulatory environment should be re-evaluated to determine the status of nutrient limits for Willamette River discharges. If these limits still do not appear to be likely, then the most probable improvements should be implemented. If there does appear to be regulatory impetus to institute limits on nutrient discharge, then strong consideration should be given to expanding the secondary system with additional aeration basin capacity.

Disinfection. Regulatory requirements or community policy may make chemical disinfection undesirable. Under this criterion, ultraviolet disinfection has been selected for use in disinfecting the WWRP effluent. However, it is important to recognize that chlorine (sodium hypochlorite) is also being used for disinfection of CSO flows, and there is currently no reliable alternative to chemical disinfection for treating these flows. Therefore, it is anticipated that chlorination will be used for disinfection of at least the CSO flowstream for the foreseeable future.

Future planning action items for disinfection are as follows:

- During the design of new chlorine contact basins, incorporate provisions for future conversion of the basins to UV channels.

- As part of the design of future UV facilities, compare low-pressure lamps to medium-pressure systems.
- Evaluate emerging technological alternatives to chemical disinfection of CSO flowstreams.

Tertiary Treatment. Chemical phosphorus removal was selected as the tertiary treatment process to meet low phosphorus effluent limits. This would include the addition of a chemical feed system (alum) and flocculating clarifiers. However, the specific conditions of any phosphorus limits are not known at this time. These limits could be met by taking a portion of the effluent stream to an alternative discharge point, such as constructed wetlands treatment or an effluent irrigation system. If these discharge alternatives could be utilized for a comparable capital investment, they would be more desirable than chemical treatment.

In the future it would be beneficial to evaluate split-stream effluent discharge through constructed wetlands treatment or effluent irrigation as a means of cost-effectively meeting any future phosphorus limits.

Sludge Digestion. Regulatory requirements or environmental policy decisions could result in a desire to produce Class A (pathogen-free) biosolids for application to agricultural land. A high temperature process that would be adapted to the existing anaerobic digestion system was selected as the treatment approach to meet this criterion. There are a number of emerging technologies capable of achieving this treatment goal, none of which has an extensive track record in the United States. Therefore, a specific treatment process was not selected as a part of this planning process.

In the future it would be beneficial to make a comparative evaluation of the following Class A treatment processes using up-to-date operational experience:

- Pre-digestion pasteurization—pasteurization vessel prior to mesophilic anaerobic digestion
- Thermophilic digestion pretreatment—thermophilic hydrolysis (either aerobic or anaerobic) prior to mesophilic anaerobic digestion
- Thermophilic anaerobic digestion—operate the anaerobic digesters at thermophilic temperatures

Biosolids Management. Site constraints or environmental adjacency concerns have been identified as criteria that may require the plant to change from a liquid biosolids application program to application and storage of a dewatered biosolids product. Mechanical dewatering and a dewatered cake storage pad have been selected as the facilities to provide this biosolids management approach.

As planning begins in the future to address facultative lagoon capacity needs, the site availability for further expansion of the lagoons and the impact on the surrounding community should be re-evaluated. If it appears that there is limited potential to continue to expand and/or utilize facultative lagoons for storage of liquid biosolids, then consideration should be given to construction of dewatering facilities to convert to a dewatered biosolids application program.

Table 6-24
Sizing of "Most Stringent" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		120,000		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Trickling Filters									
Trickling Filters	(2) 160 ft dia, 8 ft deep	18,343	None	None	None	None	None	None	lb BOD/d (WWMM)
Circulation Pumps	(2) 11.5 mgd	23	None	None	None	None	None	None	mgd (WWPH)
Aeration Basins									
Aeration Tanks	(2) 156'x40'x15'	33.6	None	None		(5) 156'x40'x15'		(3) 156'x40'x15'	mgd (WWMM)
Aeration Blowers	(2) 1,400 cfm, (2) 5,900 cfm	14,600	None	None	5,000	(2) 5,900 cfm (replace sm. units)	10,000	(2) 5,900 cfm	cfm
Disinfection									
Chlorine Contact Tank	(1) 110' dia, 10.5' SWD	28.0	50 (replace existing)	(2 basins) 92'w.x 100'l.x 15'd	None	None (convert to UV channels)	None	None (convert to UV channels)	mgd (WWPH)
UV System	None	None	None	None	50	2,688 lamps (low pressure)	14	768 lamps (low pressure)	mgd (WWPH)
Sludge Thickening									
Mechanical Thickeners for WAS	None	None	None	None	13,000	(2) 2m GBTs	7,000	(1) 2m GBT	lb/d (WAS WWMM)

Table 6-24
Sizing of "Most Stringent" Process Improvements

Component	Existing Facilities		New Facility Requirements						Capacity Units
			60,000		80,000		120,000		
	Number and Size	Estimated Capacity	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	Additional Capacity Required	Number and Size of Additional Units	
Sludge Digestion									
Anaerobic Digester	(1) 66' dia, 31' SWD, 795,000 gallons	16,500	2,000	(1) 50' dia, 50' SWD, 800,000 gallons	16,680	(1) 50' dia, 50' SWD, 800,000 gallons (to replace existing)	18,640	(1) 50' dia, 50' SWD, 800,000 gallons	lb/d (PSL+WAS WWMM)
Predigestion Pasteurization	None	None	None	None	8,000	(2) 10' dia, 14' SWD steel vessels	4,000	(1) 10' dia, 14' SWD steel vessels	gallons (SRT=2 hrs)
Facultative Sludge Lagoon	(2) 2.2 acres each	25	None	None		(2) 2.2 acres each		(1) 3.3 acres each	lb VSS/ 1,000 sq. ft./ day
Biosolids Dewatering									
Belt Filter Presses	None	None	None	None	26,000	(3) 2m BFPs	10,200	(2) 2m BFPs	lb/d (DS WWMM)
BFP Feed Pumps	None	None	None	None		(3) 10-100 gpm		(2) 10-100 gpm	
Polymer Feed Pumps	None	None	None	None		(6) 5-50 gpm		(3) 5-50 gpm	
Land Application Program									
Land		3000	None	None	None	None	None	None	acres
Sludge Storage Pad	(1) 50,000 gal.		None	None	1,170	35,300 SF	585	12,000 SF	tons (DS AA)

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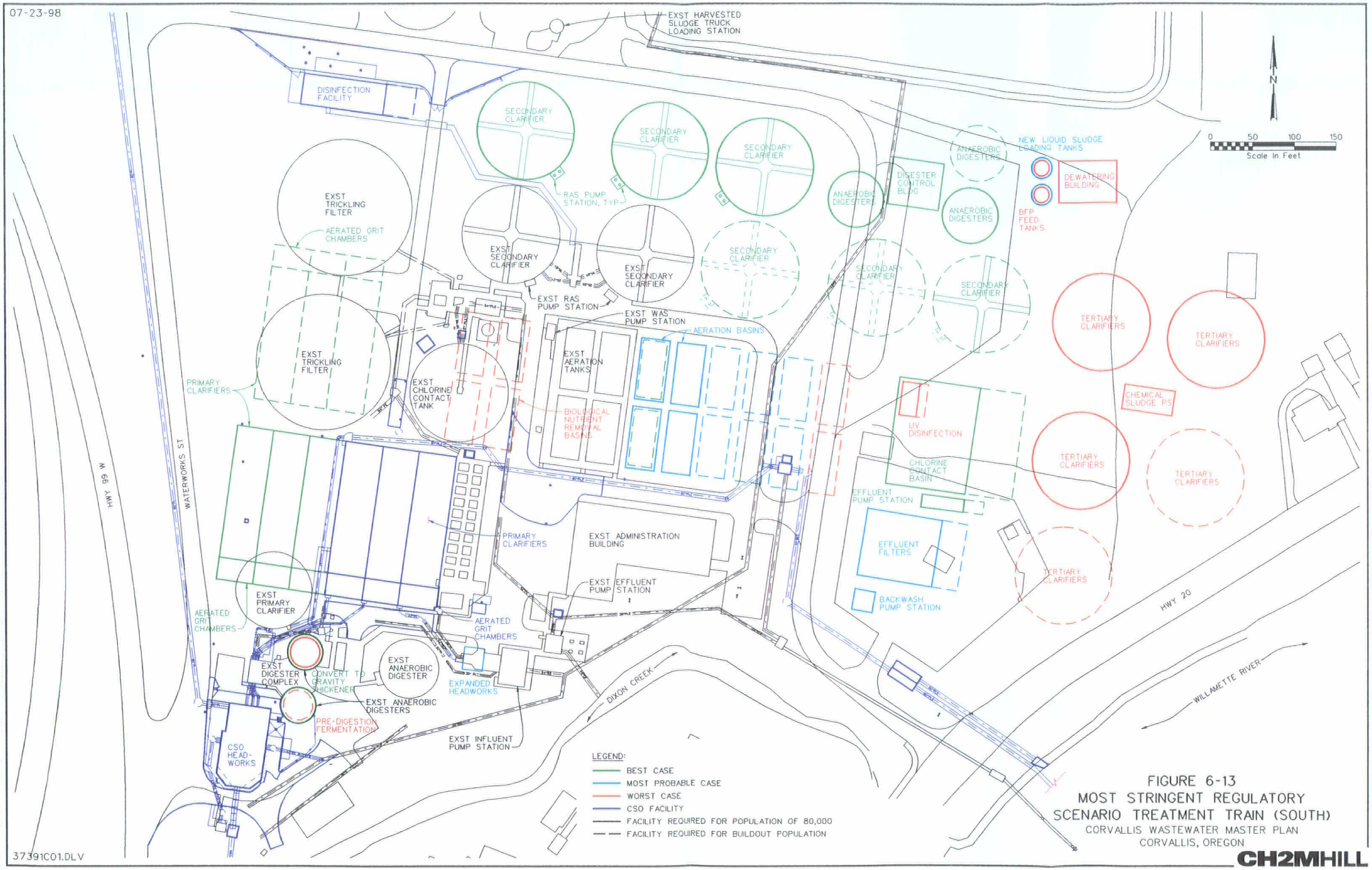

FIGURE 6-13
MOST STRINGENT REGULATORY SCENARIO TREATMENT TRAIN (SOUTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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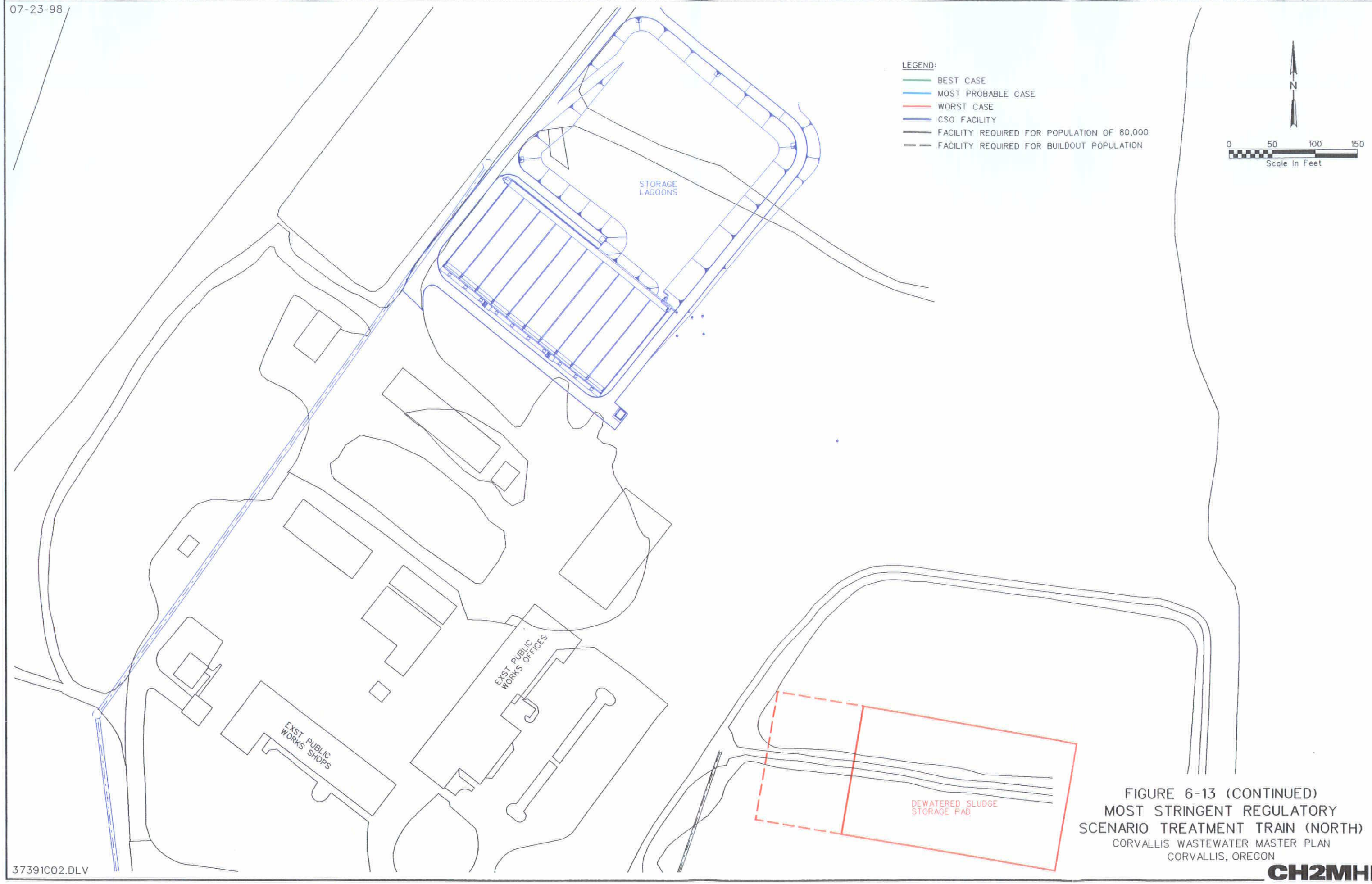

FIGURE 6-13 (CONTINUED)
MOST STRINGENT REGULATORY
SCENARIO TREATMENT TRAIN (NORTH)
CORVALLIS WASTEWATER MASTER PLAN
CORVALLIS, OREGON

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Table 6-25
Cost Estimates for "Most Stringent" Process Improvements

	Total Project Costs*		
Unit Process	**Short-Term CIP**	**60,000 Population**	**80,000 Population**
Influent Pumping/Headworks			
Primary Treatment			
Biological Treatment			$12,687,500
Secondary Clarification			
Disinfection			$3,625,000
Filtration			
Tertiary Treatment			$6,780,000
Effluent Pumping			
Solids Handling			$11,850,000
Odor Control			$2,975,000
Non Process Facilities			
Total Project Cost	**$0**	**$0**	**$37,917,500**

* Cost estimates include 25 percent construction contengency and 25 percent allowance for engineering, legal, and administrative costs. They do not include seismic improvements. Cost estimates are in 1998 dollars indexed to the *Engineering News Record* construction cost index of 6,500 for the Seattle area.

SECTION 7

Seismic Vulnerability Evaluation

Introduction

A seismic vulnerability evaluation of the Corvallis Wastewater Reclamation Plant (WWRP) facilities was conducted to identify any vulnerable portions of the plant, identify structural deficiencies, and prioritize the vulnerability of each building or process facility for potential repair or upgrade. The evaluation included the following tasks:

- Determine the regional seismicity, estimate the peak ground acceleration for an earthquake with a 10 percent chance of exceedance in 50 years, assess seismically induced ground failure hazards such as liquefaction and lateral spreading, and summarize the geotechnical issues.
- Review all available construction documents for the original plant and for subsequent additions.
- Perform onsite visual inspection to assess the condition of the existing structures and to identify any non-structural hazards.
- Perform an overview evaluation of all buildings and process facilities for post-earthquake performance based on the information gathered from the construction document review and onsite visual inspection.
- Based on the results of the vulnerability assessment, rank each building and process facility according to order of magnitude of risk for seismic vulnerability.
- Based on the results of the vulnerability assessment, suggest corrective actions or further analysis for the vulnerable portions of the structures or nonstructural components that have been identified as being at risk.

The complete report is provided in Appendix F. The major findings and recommendations are described below.

Seismic Characterization

Geomatrix Consultants completed a study in 1995 for the Oregon Department of Transportation (ODOT) describing the level of seismic hazard throughout the State of Oregon (Geomatrix Consultants, 1995). This report includes information indicating the location, site, and frequency of occurrence of earthquakes in and near Oregon. Much of the seismic-related information used in the seismic vulnerability evaluation is taken from the Geomatrix report.

Historic Seismicity

Historic seismicity in the site vicinity since approximately 1840 has been low to moderate. Jacobson (1986) indicates 14 earthquake occurrences within a 30-mile radius of Corvallis in the time period between 1841 and 1986. One of these events was greater than magnitude 5; two of these events were greater than magnitude 4; and five of these events were greater than magnitude 3.

The most recent regional event of significance was the magnitude 5.6 event that occurred on March 25, 1993, near Scotts Mills, Oregon, about 40 miles northeast of the site. No faults directly coincide with the epicenter of the 1993 Scotts Mills event, but the Mount Angel Fault is the closest known fault and the only candidate for a causative fault for that earthquake (Madin, et al., 1993). Geologic studies in the Pacific Northwest suggest that the record of historic seismicity may not be a useful indicator for expected levels of future seismicity. For this reason, the level of potential ground motion is determined by considering the activity at all seismic sources for the area, some of which have been relatively inactive in the past 160 years of recordings.

Sources of Seismicity

On the basis of geologic evidence gathered recently from coastal areas in Washington and Oregon, scientists believe that very large earthquakes have originated offshore from Oregon and Washington, along the principal tectonic feature of the Pacific Northwest. This feature involves an active subduction zone located off the western coast, where the Juan de Fuca plate subducts beneath the North American plate along the Cascadia margin. The Cascadia subduction zone west of Oregon is one of the two primary seismic sources that can affect ground motion in the state. The other primary source involves crustal earthquakes occurring within the North American plate.

Ground Acceleration and Earthquake Magnitude

Results of Geomatrix's statewide seismic design map project suggest that the peak ground acceleration on rock for the project site will be approximately 0.20 g, where g is the acceleration of gravity. This acceleration is for a 500-year return period (mean annual frequency of exceedance of 0.002), which means there is a 10 percent chance of exceedance in 50 years. This takes into account contributions from all seismic sources.

Potential Earthquake Induced Hazards

The site may be susceptible to two types of earthquake-induced hazards, as follows:

- **Liquefaction**. Liquefaction refers to the loss of shear strength that saturated soil deposits can experience during undrained cyclic loading, such as earthquake loading. Cyclic liquefaction occurs when pore pressures within soil increase to the point where the effective stress in the soil approaches zero. When this happens, the shear strength of the soil decreases and permanent deformations within the soil can accumulate to large values (greater than 5 percent strain). The susceptibility of a soil deposit to liquefaction is a function of the degree of saturation, soil grain size, relative density, percent fines,

age of deposit, plasticity of fines, earthquake ground motion characteristics, and several other factors. Liquefaction can significantly affect both settlement and slope stability.

The soils most susceptible to liquefaction at the site are the soft to medium stiff silts in the depth range of 10 to 20 feet that have SPT N-values generally below 10 blows per foot (bpf). Fortunately, these deposits also contain some degree of clay materials as indicated by their high liquid limit. Clayey deposits generally do not liquefy as easily as sands or nonplastic silts. Based on the cohesive, fine-grained nature of this upper silt material, it appears that there is a low probability that the silt could liquefy. The probability that the underlying materials would liquefy is also low because of the depth and density of the granular deposits and the depth, cohesion, and particle size of the fine-grained soils.

A 3- to 5-foot-thick layer of loose sand is present in some areas at the site that could be susceptible to liquefaction. However, this layer overlies a dense sand and gravel layer that would likely prevent significant amounts of pore pressure from building in the thin, loose sand layer.

- **Fault Rupture.** The potentially active faults that are present in the Corvallis vicinity are not adjacent to, nor do they underlie, the treatment facility. Even though these faults could be the source of an earthquake, the probability of a fault rupture near the site is low.

Vulnerability Evaluation

The evaluation assumed that the plant will be subjected to a Uniform Building Code (UBC) prescribed earthquake. The prescribed earthquake equates to an effective peak acceleration of 0.30 g with a 10 percent chance of exceedance in 50. The value of 0.30 g at the ground surface is a code-required acceleration, which is higher than the peak ground bedrock acceleration of 0.20 g predicted for the site. The evaluation was limited to the buildings and process facilities at the Corvallis WWRP site. This did not include onsite facilities that were to be completely demolished in 1998.

The buildings and process facilities were rated as high, moderate, or low for each of the three following types of risk:

- **Life Safety:** Failure of, or damage to, a building system, nonstructural secondary building component; or nonstructural process or supply system represents a direct threat to human life. For example, complete collapse of a building represents a direct threat to the occupants of the building; also, a release of chlorine into the atmosphere represents a direct threat to the occupants of the plant and nearby residents.

- **Health Hazard:** Partial failure of, or damage to, a building system, nonstructural secondary building component; or nonstructural process or supply system represents a threat to health but is not an immediate threat to life. For example, discharge of untreated waste into adjacent streams or the Willamette River would represent a threat to human health but not an immediate threat to life.

- **Economic Loss:** Localized failure of, or damage to, a building system, nonstructural secondary building component, or nonstructural process or supply system does not represent a threat to life or health. However, loss of function results in economic loss. For example, an expensive piece of equipment that may be damaged during an earthquake may not represent a threat to life or health, but the cost to replace the equipment could be significant.

The evaluations were based on the findings of the visual inspection and construction drawing review.

Prioritization

The rating results were used to prioritize the buildings and facilities according to their relative seismic vulnerabilities. The most important type of risk considered was Life Safety. Table 7-1 shows the ranking of the facilities by priority.

TABLE 7-1

Seismic Prioritization of Buildings and Process Facilities

Rank	Facility No.	Facility	Life Safety Risk	Health Hazard Risk	Economic Loss Risk
1	1000	Operations and Maintenance	High	High	High
2		Diesel Fuel Building	High	Moderate	High
3	6400	Digester Control Building	High	Low	High
4	4300	Secondary Sludge Pump Station	Moderate/High	Low	Moderate
5	6200,6300	Digester Tanks	Moderate	Moderate	Moderate
6	6100	Primary Digester	Moderate	Moderate	Moderate
6	4010,4020	Trickling Filters	Moderate	Moderate	Moderate
7	2000	Influent Pump Station	Moderate	Moderate	Moderate
8	0300	Effluent Treatment Facility	Moderate	Low	Moderate
9	2500	Headworks	Low	Moderate	Moderate
10	5300,5400	Secondary Clarifiers	Low	Low	Low
11	6700	Primary Clarifier	Low	Low	Low
11	4200	Secondary Clarifier/Chlorine Contact Basin	Low	Low	Low
11		Filter Pump Station	Low	Low	Low
11	0400	Control Gate Structure	Low	Low	Low
11	3100	Primary Sedimentation	Low	Low	Low
11	5100	Aeration Tanks	Low	Low	Low
10		Outfall Diversion	Low	Low	Low
10		Tunnels	Low	Low	Low
11		Garage	Low	Low	Low

Recommendations

Recommendations for corrective actions or further analysis are described below for each of the at-risk buildings or process facilities at the Corvallis WWRP. The facilities are listed in order of importance from high risk to low risk. No attempt was made to rank the structures based on cost of upgrade or replacement; therefore, it is recommended that prior to proceeding with any of the following corrective actions, a design and construction costs should be estimated. The cost estimates should be based on further analysis and code requirements. Further analysis is recommended next fiscal year.

Operations and Maintenance (Facility 1000)

Lateral Force System

A complete analysis of the structural lateral forces resisting system should be performed at the Building Department-mandated code force levels. Particular attention should be paid to those columns and roof beams that are not part of the lateral force system but must deflect with the building in an earthquake. Also, the separation of the building segments should be verified as sufficient to handle expected lateral displacement.

Precast Panel Connections

The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Chlorine Tank Storage

Chlorine storage tanks should be anchored or stored in a manner that prevents any horizontal movement during an earthquake. This corrective action should be implemented immediately because the presence of chlorine poses the greatest direct risk to life safety at the plant. This action is necessary only until the gaseous chlorine system is removed during the Combined Sewer Overflow improvements in the summer of 1998.

HVAC Units (UH1225)

All hanging HVAC units should be braced laterally to prevent horizontal movement.

Blowers and Piping (B1111, B1112, B1113, and B1114)

Provide nuts on all blower anchor bolts. Laterally brace all blower piping and ductwork to prevent horizontal movement.

Electrical/Control Equipment

Verify anchorage of all electrical and control equipment in excess of 400 pounds or supported overhead or off of walls. Provide anchorage for any equipment that lacks it.

Diesel Fuel Building

Diesel Fuel Tank

Anchor tank to concrete slab to prevent movement and tipping during an earthquake.

Digester Complex

Digester Tanks (Facilities 6200 and 6300)

Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls.

Digester Control Building (Facility 6400)

A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels. Particular attention should be paid to the steel columns and concrete roof beams that are not part of the lateral force system but must deflect with the building in an earthquake.

Brick Veneer

Provide retrofit anchorage of the brick veneer to the existing building and tank walls.

Equipment Anchorage

Provide anchorage for boilers 6521 and 6511 and Tank T-6525. Verify anchorage of all equipment in excess of 400 pounds or that represents a hazard.

Secondary Sludge Pump Station (Facility 4300)

Lateral Force System

A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels.

Precast Panel Connections

The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Equipment Anchorage

Verify anchorage of all equipment in excess of 400 pounds or that represents a hazard.

Primary Digester (Facility 6100)

Tank Sloshing

Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls.

Brick Veneer

Provide retrofit anchorage of the brick veneer to the existing building and tank walls.

Gantry Crane

Remove gantry crane from the roof or provide a means to prevent movement or rolling of the crane during an earthquake.

Trickling Filters (Facilities 4010 and 4020)

Tank Walls

Perform structural analysis of the existing tank for the lateral acceleration of the filter media in an earthquake. The analysis should evaluate potential cracking or overloading of the walls.

Influent Pump Station (Facility 2000)

Lateral Force System

A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels.

Precast Panel Connections

The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Electrical/Control Equipment

Verify anchorage of all electrical and control equipment in excess of 400 pounds. Provide anchorage for any equipment that lacks it.

Roof-mounted HVAC Equipment (UH2605)

Verify anchorage of roof-mounted HVAC equipment. Provide anchorage for any equipment that lacks it.

Wet Well Divider Wall

The wet well divider wall should be analyzed for the effects of sloshing. The analysis should evaluate potential cracking or overloading of the divider wall.

Effluent Treatment Facility (Facility 0300)

Electrical Transformers (TFR102, TFR103, TFR104, and TFR105)

Anchor all transformers to the concrete pad.

Pump Cover

Evaluate existing steel moment connections for increased seismic loads.

Pipe Support

Anchor pipe support on top slab to concrete.

Headworks (Facility 2500)

Basin Walls

Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls

Bar Screens

Evaluate anchorage of bar screens for increased seismic loads.

Secondary Clarifiers (Facilities 5300 and 5400)

Control Equipment at Center Pier of Clarifier 5400

Anchor all control equipment at center pier of clarifier.

RAS Pump Cover

Evaluate existing steel moment connections for increased seismic loads.

RAS Piping

Modify existing pipe supports to prevent damage to piping during an earthquake.

Filter Pump Station

Large Diameter Piping

Modify existing pipe supports to prevent damage to piping during an earthquake.

Aeration Tanks (Facility 5100)

Air Piping

Provide lateral bracing for air piping.

WAS Pump Cover

Evaluate existing steel moment connections for increased seismic loads.

Tunnels

Heat Piping and Circulated Sludge Piping

Provide strapping to prevent lateral movement of piping during an earthquake.

References

Converse, Davis and Associates, Inc. 1974-1977. Various Geotechnical investigations for the Corvallis WWRP.

Geomatrix Consultants. January 1995. *Seismic Design Mapping, State of Oregon*. Prepared for Oregon Department of Transportation.

Jacobson, R.S. 1986. Map of Oregon Seismicity. Oregon Department of Geology and Mineral Resources Geologic Map Series GMS-49, Scale 1:100,000.

Madin, I.P. Priest, G.R. Mabey, M.A., Yelin, T.S., and Meier, D. 1993. March 25,1993, Scotts-Mills Earthquake – Western Oregon's Wake-up Call. Oregon Geology, Vol. 55, No. 3, pp. 51-57.

Orr, E.L., Orr W.N., and Baldwin, E.M. 1993. *The Geology of Oregon*. Kendall/Hunt Publishing Company: Dubuque, Iowa.

Walker, G.W. and N.S. MacLeod. 1991. Geologic Map of Oregon. U.S. Geological Survey, Scale 1:500,000.

SECTION 8

Non-Process Facilities

This section provides recommendations for improvements to the non-process facilities. The non-process facilities include those facilities located at the Corvallis Wastewater Reclamation Plant (WWRP) that support operation and maintenance of the treatment plant and collection system. This includes office space, meeting areas, shower and locker facilities, maintenance shop, materials storage, and laboratory spaces. Included in this section is a summary of the existing staffing levels and the facilities to support that staff, an assessment of the existing facility's ability to meet both current and long-term needs, and recommended improvements for meeting those facility needs.

Existing Staff and Facilities

The facilities and staff at the WWRP support a variety of public works functions for the City. These are outlined below.

Operations

The WWRP operates on an 8-hour-per-day, 7-day-per-week schedule. There are currently a supervisor, eight full-time operators and five seasonal (summer) staff that provide operations for the following functions:

- Wastewater treatment
- Biosolids land application
- Operations of the United Chrome (Superfund) site
- Automated control and monitoring systems

The full-time staff provide support for all of the functions mentioned above. One of the current operator positions is being converted to a automation technician that will provide support for I&C system software development, computer maintenance, and the SCADA systems for both water and wastewater. The five seasonal positions primarily are used to support the biosolids land application program.

Laboratory

There are currently three full-time laboratory staff that provide analytical support for the following treatment programs:

- Wastewater
- Water
- Industrial source control program

This staff is currently located in the existing WWRP Administration Building.

Maintenance

The maintenance staff consists of a supervisor and five full-time staff members. The full-time staff consists of three mechanics and two electrician/instrument technicians. This staff supports the following public works functions:

- Water treatment
- Offsite water facilities (reservoirs and pump stations)
- Wastewater treatment
- Wastewater lift stations
- Biosolids land application
- Operations of the United Chrome (Superfund) site
- Automated monitoring and control systems

These staff are divided between two locations. The instrument technicians are located in an instrument shop in the basement of the digester control building, and the remaining maintenance staff are located in the maintenance shop in the WWRP Administration Building.

Materials Storage

The materials storage area is required to provide storage for spare parts, consumable materials (e.g., oil, lubricants, chemicals) and maintenance equipment (e.g., submersible pumps, portable generators). This storage is currently provided at a number of different locations both on and off the WWRP site. Onsite storage is currently provided in the following locations:

- Lube oil storage building
- Maintenance shop
- Blower room
- Basement of the digester control building (adjacent to the instrument shop)
- Various locations in the basement of the older portion of the WWRP

The following offsite storage is also provided:

- Rolling stock storage on the Public Works Department site
- Pump and spare parts storage at the water booster pump station sites

Assessment of Existing Non-Process Facilities

Table 8-1 shows a comparison of existing Corvallis administration and laboratory facilities with other treatment plants in the Pacific Northwest. As can be seen from this table, there is a fair amount of variability in the non-process facility provisions for these plants caused by hours of operation (number of operations shifts), slight differences in the services that are supported from a particular facility, and how recently a significant plant expansion has occurred. This is particularly true of maintenance facilities where the services being provided can include support for only the immediate plant facilities, system-wide facilities (e.g., support of all the wastewater facilities), or regional maintenance facilities (e.g., support for water, wastewater, and collection system similar to Corvallis). Also, the amount of maintenance support that is out-sourced can have a significant impact on the size of City-owned

Table 8-1
Comparison of Administration and Laboratory Facilities Areas
February 1998

	Corvallis WWRP	Eugene/ Springfield WWTP	Columbia Boulevard WWTP	Durham (USA) WWTP	Rock Creek (USA) WWTP	Salem Willow Lake WWTP	Seattle West Point WWTP	Spokane WWTP	Tacoma Central WWTP	Tacoma North End WWTP	Yakima WWTP
Design Flows (MGD)											
Average Wet Weather	15	66	-	41.7	41.7	-	133	-	42	7.2	18
Average Dry Weather	9.7	49	100	20	20	35	110	44	22	7.2	17
Peak	28	175	300	90	90	105	440	77	95	10	25.5
Level of Treatment Provided	Secondary	Secondary	Secondary	Tertiary	Tertiary	Secondary	Secondary [E]	Secondary	Tertiary	Secondary	Secondary
Year(s) Constructed	1979	1983	1952-1995	1977-1994	1978-1995		1995	1958-1977	1952-1989	1968	1936-92
FTE Staff											
Administrative Staff [A]	2	13	29	7	8	11.5	18		9		
Laboratory Staff	3	13	7	5	7	7	6		18 [F]		
Operations Staff [H]	8	19	55	22	22	18	36		50		
Maintenance [I]	5										
Totals	18	45	91	34	37	36.5	60	70 [D]	59	6 [D]	40 [D]
Administration											
No. offices	2	5	25	9	12	5	18	5	18	1	1
Administrative areas, ft^2 [B]	312	750	-	2,142	3,953	927	4,291	600	4,317	174	2,070
Conference/training rooms, ft^2	see lunch rm.	1,600	5 rooms	2,815	2,591	N/A	2,094	2,000	568	1 room	1 room
Storage area, ft^2		350	5 rooms	43	302	634	505	48	766	-	1 room
Total area, ft^2	872	2,700	14,546 [G]	5,000	6,846	1,561	6,890	2,648	5,651	174 + 1 room	2070 + 2 rooms
Area per Admin. Staff	290	208	502	714	856	136	383	N/A	628	N/A	N/A
Personnel											
Mens Locker/restroom facilities, ft^2	460	700	7(+5 coed) rooms	1,060	904	553	1,734	1,900	502	143	494
Womens Locker/restroom facilities, ft^2	270	300	7 rooms	460	584	122	1,074	600	352	N/A (unisex)	260
Lunch room(s), ft^2	560	(incl. in conf. rms)	4 rooms	783	1,341	840	489	1,500	463	Conference Rm.	736
Total area, ft^2	1290	1,000	6,478	2,303	2,829	1,515	3,297	4,000	1,317	143	1,490
Area per Total Staff, ft^2	72	22	71	68	76	42	55	57	22	24	37
Laboratory											
No. offices/total office area, ft^2	0	1/150	4/276	1/302	9/1636	4/881	1/96	2	-	1	8
Sample storage, ft^2	100	250	(see "Process")	176	484	675	234	150	1,251	-	200
Process, ft^2	1,252	2250	5,458	2477	2655	1853	2894	3500	4,267	-	1,200
Total area, ft^2	1352	2,500	5,458	2,653	3,139	2,528	3,128	3,650	5,518	N/A	1,400
Area per Laboratory Staff, ft^2	450	192	780	531	448	361	521	N/A	307	N/A	N/A
Other Areas, ft^2 [C]		150	-	772	877	243	124	-	-	-	-

NOTES

A Administrative Staff includes supervisory staff in laboratory and operations.
B Administrative area includes offices & common work areas outside of conference/training rooms and storage.
C Includes such areas as mud rooms, linen/laundry rooms, janitor rooms.
D Survey response to this question was not broken down to specific staff categories
E Seattle West Point WWTP has <1 MGD water re-use facility.
F Lab staff at Tacoma Central WWTP has Superfund work and industrial source control program as well as plant process analysis.
G Area estimates for Columbia Blvd. include hallways and general circulation areas. Lab does industrial source control program as well as plant process analysis.
H Includes 5 seasonal staff (5 sludge truck drivers and student interns) counted as 1 FTE.
I Maintenance supervisor's office, mechanic's office and shop in same building as Administration and Laboratory.

facilities. For example, utilizing contract vehicle maintenance can eliminate the need for City-owned vehicle maintenance facilities.

This comparison does show that, in general, laboratory and administrative space is below that seen at other treatment facilities. Of the facilities that do show comparable areas, many are currently planning expansion of non-process facilities (e.g., Eugene-Springfield has recently obtained approval for a laboratory expansion that will include some additional office space).

A discussion of the specific non-process facilities is included below.

Operations

In general, the area requirements for plant operations staff are being met by the existing facilities. The exception to this is the area provided for automation. This area is inadequate to meet the growing needs of highly automated treatment and collection system facilities and an overall growing reliance on the day-to-day use of personal computers for reporting, record-keeping, and data management. Typically, these types of areas provide a separately partitioned space for operating LAN and server systems, a computer workstation for each automation staff person responsible for programming, and an equipment maintenance area that provides parts storage and a work area. The area currently designated for this function is located in partition walls within the existing process control area of the lobby. It is approximately one-half the area needed for parts storage, work area, and for locating servers for the computer network. There is currently no location where operating computers and servers could be located to better control the operating environment.

In addition, the following observations were made:

1. The existing process control area provides adequate space for operations staff, but is poorly utilized. This area should be renovated to eliminate the obsolete graphic display panel and better utilize the space available.

2. The existing lobby area of the Administration Building, although seeming quite large when compared to other areas, provides space for staging of plant tours and large groups of plant visitors. These tours are conducted on a regular basis (between grade school groups and classes at Oregon State University) and benefit from having a location where the groups can gather and the process control graphics can be accessed on the plant computer system. This space could be utilized for expansion (e.g., for additional automation area) if it were determined that the existing lunchroom/training area were acceptable for staging these plant tours. It is normally best to avoid the use of this space if possible because meetings or training may be underway or plant staff may be taking breaks or eating lunch when these tours occur.

3. The existing resource office adjacent to the plant superintendent's office provides an area where plant operations staff can update the operational database, perform record keeping functions, and complete regulatory monitoring reports. This area should be retained in future non-process facility modifications.

4. The existing areas used for process control testing conducted by plant operations staff are inadequate. These areas include a significant portion of Sample Receiving and some of the Laboratory analytical space itself.

Future needs for operations will consist primarily of any staff additions that may be required to support growth of the treatment facilities at the plant. The new wet weather treatment facilities and the proposed solids handling improvements are examples of facility expansion that could impact the operational requirements for the plant. Given current staff levels, strong consideration should be given to one additional operations staff member to support these new facility improvements.

Laboratory

The analytical space in the laboratory area was described by staff as being cramped, but workable. The crowding of the space is evidenced by the need to continually clear space to perform tests (an inefficient practice) and by using carts in the aisles to perform tests (an unsafe practice).

The analytical space is marginally adequate to meets its mission of regulatory and process control testing. The overall space is, however, adequate to safely house additional analytical staff. With two exceptions, the current full-time staffing level should continue as it is for the foreseeable future. Future process changes for nutrient control should require part-time seasonal (summer) staff. And future storm water monitoring requirements should require part-time seasonal (winter) staff.

Like some wastewater utilities, e.g. Eugene-Springfield, Corvallis may face regulatory requirements for "clean" sampling and analysis for metal contaminants. Clean sampling and analysis refers to the measurement of metals at lower than typical concentrations (tenths of parts per billion and lower) for risk assessment purposes. "Clean sampling" requires extraordinary procedures to prevent contamination; "clean analysis" requires, as a minimum, HEPA filtered Class 100 air (like an electronic industry clean room) for sample handling and preparation. While "clean sampling" is usually performed by the utility, the frequency of "clean analysis" does not economically justify development of "clean" facilities, so the work is typically out-sourced. If "clean analysis" is required, we recommend out-sourcing to a qualified contract laboratory.

In addition, the following deficiencies were identified:

- Currently, laboratory staff must do all records keeping and report preparation at two bench-top workstations within the laboratory. Providing office space for these functions would provide improved utilization of the analytical area. These bench-top work stations house some analytical equipment that does not fit in the analytical area.
- There is currently inadequate space for storage of laboratory records and files within the laboratory area.
- There is one inadequate analyst computation space located in a high traffic area (south side).
- Sample receiving and sample storage (refrigeration) is inadequate to meet the requirements for routine testing and especially for "clean" sampling and analysis. Additionally,

there is insufficient room to properly log samples and transfer samples required to meet chain-of-custody protocols (for the pretreatment program).

- The two fume hoods marginally meet current needs and would be inadequate for additional nutrient testing
- The existing Administration Building HVAC system recirculates laboratory exhaust air, which is prohibited by current code and safety standards.

Staff Support Spaces/Code and Access Compliance

The existing Operations and Maintenance Building was designed and constructed in 1975. Since then new building codes and the Americans with Disabilities Act (ADA) accessibility code has been adopted. These modified and new codes place additional requirements on new construction and rehabilitation of existing buildings. Although new code requirements are typically not applied to areas of a building not being modified, code provisions do exist to require a portion of the budget be applied to "barrier removal." For example, if a rehabilitation project is budgeted for $100,000, then up to $25,000 may be directed towards ADA compliance. Limits of required compliance are difficult to quantify and often negotiated with local building officials. Several areas of concern from both a code and functional relationship include:

- General exterior and interior circulation. Circulation to and through the building does not meet current code requirements. Currently many doors are not provided with adequate clear space requirements. There is not an accessible path of travel from the office/operations area to the shop area. A ramp accessing the current breezeway does not appear to meet current access code requirements.
- Toilet and locker spaces do not meet current access and space requirements. Spaces are not accessible, plus current building codes require equal facilities for male and female employees. Additionally, from a functional standpoint, if the number of female staff increases, the current space may not be adequate to accommodate increased staff needs.
- Both of the above concerns are magnified by the increasing frequency of public tours often given at public utilities. Compliance with current codes benefit the visitor as well as staff.
- Given the age of the building, a thorough analysis should be made of architectural items that may be at the end of their life. Examples might include replacement of floor materials, leaking windows, inoperative doors, worn casework, etc.

Maintenance

The facilities provided for mechanical maintenance are adequate to meet both current and anticipated future needs. This 1,400-square-foot area should be adequate to support the addition of one more mechanic in the future.

The primary deficiency for maintenance facilities is the space and location for electrical and instrument maintenance. The 300-square-foot instrument shop, currently located in the basement of the digester control building, is cramped and inconveniently located. A space providing approximately 200 square feet per technician would more adequately meet the

needs for work area, and space for equipment and maintenance manuals. In addition, new piping being installed as part of the CSO improvements will eliminate the 200-square-foot storage area for electrical equipment and parts, thus requiring that this storage be relocated to an area away from the shop. As a further consideration, the solids handling improvements currently in design will require additional floor space in the basement of digester control building to locate the thickened sludge pumps for the gravity thickener modifications (see Section 6).

Given the constraints on this area, it is recommended that the instrument shop and its storage area be relocated to an area of the plant that is adjacent to parts shipping and receiving, and, if possible, the mechanical shop area.

Materials Storage

- The existing materials storage is inadequate to provide the space necessary on site. In addition, the space that is available is split between a number of locations. Further compounding this storage shortfall is the fact that storage space provided in the old influent pump station, the lube oil storage building, and in the basement of the digester control building is being displaced by the CSO improvements. The following observations were made regarding materials storage. The storage needs being served by the old influent pump station and the 200-square-foot I&C storage area in the basement of the digester control building are being displaced by CSO improvements and will need to be replaced with new facilities. An existing metal building is being converted to appropriate facilities to replace the lube oil storage building.
- Pumps and spare parts are currently being stored at water booster pump stations. This equipment could be better accessed, monitored, and protected from the weather if covered (but not enclosed) storage facilities were provided on the WWRP site.
- A significant volume of spare parts inventory will be delivered as a part of the contract requirements for the new CSO treatment facilities. A storage location of approximately 120 square feet will need to be identified prior to completion of these facilities (January 2000) to store this parts inventory. This storage should include storage bins and/or shelving.

Recommended Non-Process Facility Improvements

Table 8-2 shows a summary of the recommended non-process facility improvements. This table identifies the new square footage being proposed, the existing spaces to be renovated, and the estimated costs for these improvements. It is recommended that these improvements be implemented in phases based on need or as a part of other process-related improvements. A discussion of these recommendations is included below, as well as the basis of costing.

TABLE 8-2
Summary of Non-Process Facility Improvements

Facility Improvement	Renovation Area (ft^2)	New Area (ft^2)	Estimated Cost ($ 1998)
Shower/Locker Rooms	800	300	$187,500
Computer/Automation Area		300	$67,500
Process Control Area	500		$36,000
Laboratory			
Process Control Laboratory		250	$46,900
Laboratory Sample Receiving		250	$46,900
Laboratory Office/Records Management	275		$41,200
Laboratory Process	310		$28,000
HVAC System Upgrade			$22,000
Instrument Shop	1,040		$111,600
Totals	**2,925**	**1,100**	**$587,600**

Notes:

1. Laboratory Process area includes current workstations (2) and workstation counter on south wall. Renovated cabinetry totals 53 linear feet in the 310 square foot area.

2. All cost estimates include a 25 percent construction contingency and 25 percent allowance for engineering, legal, and administration costs.

3. Cost estimates in 1998 dollars indexed to the Engineering News Record construction cost index of 6500 for the Seattle area.

4. These costs do not include seismic improvements.

Operations

It is recommended that the existing process control area be modified to eliminate the graphic display panel and re-configure the space with open-office type partition walls. This would make for a flexible layout that could be modified in the future to address changing needs. This renovation work was estimated at $40 per square foot.

It is also recommended that a new 300-square-foot enclosed space be constructed for computer/automation support. This space would provide 200 square feet of workstation for software development, computer maintenance, and storage of file copies of computer and control system software. In addition, a 100-square-foot area could be subdivided to provide an area for locating servers for the computer and SCADA networks. This area was estimated as an addition to the existing Administration Building at the southwest corner of the lobby

area. If it was deemed an appropriate use of the space, the lobby could be renovated to provide this area and the costs could be reduced.

It is recommended that a 250 square foot Operations Laboratory be developed for process control testing. The Operations Laboratory could be housed in space created by enclosing the existing 600 square foot breezeway between the existing lab area and chlorinator room.

Laboratory

To provide office and record management area for the laboratory, it is recommended that the existing chlorinator room be renovated to provide these functions. This is approximately 275 square feet that could be modified to provide 3 office-type workstations, a records storage area, and shelving for references and vendor catalogs. This would reduce office needs in the laboratory space.

It is recommended that a 250 square foot Sample Receiving area be developed and located for easy access by operations and other city staff without direct entry into the laboratory (to assure chain-of-custody protocols). Sample Receiving could be housed in space created by enclosing the existing 600 square foot breezeway.

In addition, the laboratory layout should be modified and renovated to provide:

- One additional 8 foot fume hood for sample preparation
- One 8 foot canopy hood for incubators, water baths, and autoclave
- Conversion of office casework to laboratory casework with laboratory grade counter tops
- Three analyst computation spaces, two smaller and one larger than existing in-laboratory spaces
- Better access to the Laboratory Equipment Room

It is recommended that the HVAC system be upgrade to bring it to code compliance and to accommodate the recommended operations and laboratory improvements. The current upgrade in cooling capacity (separate project) and a large portion of the ducting would be reused, but increases in air handling and heating system capacity would be anticipated.

It should be noted that the locations identified in our recommendations for the laboratory, sample receiving, and operations laboratory could, and perhaps should, be modified in facility predesign. Enclosing the breezeway or building additions are options to meet the needs. Enclosing the breezeway could provide additional storage space for Operations as well.

The basis of cost for these improvements is outlined below:

Chlorinator room modifications	$100/ft^2
Breezeway modifications	$150/ft^2
Fume hood	$11,000
Canopy hood	$700
Casework modifications	$350/linear foot

Staff Support/Code and Access Compliance

Recommended modifications to the Operations and Maintenance Building should include the addition of a code compliant path of travel from the parking lot to and through the building. This may include a new ramp accessing the front entrance to the building, providing required clear spaces at doors and potentially a wheelchair lift to allow access to the Maintenance Shop level. As noted previously, a percentage of the construction budget must be targeted towards barrier removal, and the limits of this work will need to be negotiated with the building official. Additionally, because of the increasing frequency of public tours, the potential for necessary handicapped access increases.

In the toilet and locker rooms, additional space needs to be provided for future increases in staffing level. A good "rule of thumb" is to allocate 50 square feet per current and future employee. With the current configuration, ADA codes are not met in the existing spaces, nor are the current building codes that require equal toilet facilities for female staff. Some of this work may be delayed until need occurs, but, as noted earlier, some immediate modification may be necessary because of barrier removal code provisions. The recommendation would be to enlarge and upgrade these facilities in one step.

Recommendations for upgrading other building-wide architectural items will be analyzed during predesign activities and included at that time.

Maintenance

The recommended modifications to maintenance facilities consist of relocating the instrument shop to the recently vacated chlorine gas storage area. This area would be renovated to provide 1,040 square feet for three work areas for instrument technicians (600 square feet) and I&C storage (440 square feet). Because this is a high-bay area, a mezzanine level could be created for storage of spare parts and other materials with a ship's ladder for access. Note that relocation of the instrument shop may become necessary in the near-term to accommodate thickened sludge pumping being incorporated into the new solids handling modifications.

Costs for these modifications were based on $60 per square foot. This cost is to provide instrument shop and storage, as well as a mezzanine storage area.

Materials Storage

Materials storage is the non-process area with the greatest deficiencies. These non-process facilities must first address the area being eliminated by the CSO improvements and then provide for additional storage needs. By implementing the maintenance improvements outlined above, the storage requirements for both I&C parts and materials and mechanical spare parts will be addressed.

Implementation of Non-Process Facility Improvements

The most pressing near-term non-process facility modification is the relocation of the instrument shop. This space in the digester control building basement will be required for installation of the new thickened sludge pumps associated with the solids handling improvements. These improvements are scheduled for completion prior to the startup of the CSO treatment facilities. In addition, by renovating the chlorine gas storage area, the proposed mezzanine

storage area will become available for storage of spare parts being delivered at the completion of the CSO remediation project. However, because the disinfection facilities that will allow this area to be de-commissioned will be constructed as a portion of the CSO project, very close coordination with all three projects (CSO, solids handling and non-process facilities) will be required to allow these modifications to be properly phased. Since the instrument shop is being housed in a temporary building while construction of both the CSO and solids handling improvements are underway, it would be best to schedule the non-process facility improvements for the new instrument shop after completion of the new disinfection facilities in the CSO project.

The next highest priorities are the new automation area and the laboratory improvements. Similar to the instrument shop relocation, the laboratory improvements would need to be coordinated with the CSO project to ensure that the chlorinator room is available for renovation. The proposed modifications for providing the new automation area are not constrained by any other projects, but will become more critical as automation needs for both the CSO and solids handling projects come on line.

It is important to recognize that seismic improvements should be implemented in the existing Administration Building in addition to the non-process facility improvements shown. The costs for these improvements cannot be estimated at this time and have not been included in the cost estimates in Table 8-2. Potential corrective actions and additional analysis requirements have been identified in Section 7. However, building codes will likely require that these upgrades occur before or as a part of any renovation of existing structures. Therefore, costs, priorities, and implementation schedules of the non-process facility improvements should be closely coordinated with the seismic upgrade requirements.

SECTION 9

Wastewater Utility Management and Policy Recommendations

Utility Management and Policies

For the Wastewater Utilities master plan objectives to be achieved, utility management and policies must coincide and support achieving the plan objectives. The utility is currently well managed and operated. The utility has provided a high level of customer service and implemented improvements protective of public health and the environment. Compliance with regulatory requirements has been outstanding. The master plan identifies a schedule and priority of utility improvements necessary to maintain the current wastewater infrastructure and serve new customers while meeting increasingly more stringent regulatory requirements. Some current utility policies will be need to be enhanced and new policies and management guidelines adopted. Table 9-1 identifies recommended policies to more fully assure the master plan objectives are met. The policies are grouped into five categories and are briefly discussed below. Development of the policies, either new or enhancement of existing policies, should be undertaken by the utility management with support and approval from the City Council. Periodic review of the policies is appropriate given potentially changing community values and the regulatory environment.

Collection System

As presented in Section 3 Collection System Hydraulic Analysis, portions of the existing collection system date back to the turn of the century. Average age of the collection system is over 30 years old. Infiltration and inflow (I/I) heavily influence flows in the collection system. Policies shown in this category are largely intended to control I/I in the current collection system and new sewer extensions to serve developing areas. I/I in the existing system will need to be maintained at no more than current levels or reduced where cost-effective by a dedicated commitment to increased funding for the identification and removal of I/I and to maintain the structural integrity of the system. A large contributor to I/I is attributed to defects or illegal cross connections associated with private laterals. Policies are recommended that will reduce this source of I/I. I/I from new development must be controlled by more stringent construction standards that require new construction materials and installation methods as they become available and by more stringent inspection programs to ensure proper workmanship.

Collection system policies are also recommended to ensure new developments provide acceptable areas for the cost effective and environmentally acceptable construction of future City sewer pipelines and lift station sites as well as their maintenance of these needed utility infrastructure improvements.

A policy aimed at providing a level of environmental and public health protection against personal property or street flooding by sewage in a 1 in 5-year frequency is recommended. Overflows should be prioritized so that they discharge first to the Willamette River, then to Marys River, then to urban streams, then to the streets (flooding), and then, as a last resort, to personal property. Collection system improvements recommended in the master plan have been based on achieving this level of human health protection. This overflow protection level and prioritization is consistent with the Oregon Department of Environmental Quality regulations.

Wastewater Reclamation Plant

Recommended policies for the wastewater reclamation plant at subdivided into three categories: 1. Liquid/hydraulic processes, 2. Biosolids, and 3. Environmental Adjacency.

Liquid/Hydraulic process recommended policies are largely aimed at the implementation of capital improvements that cost effectively reduce operation and maintenance (O&M) costs. The city has historically made improvements to control O&M costs. Technology advances in automation and efficiency continues to provide an opportunity to minimize the requirements for staffing, energy, chemicals, and other O&M related expenditures. Incorporation of these technological advances will help offset O&M cost increases brought about by more stringent regulatory requirements and escalating labor, chemical, and energy costs.

The level of wastewater treatment provided should be consistent with regulatory requirements and community values. The citizens of Corvallis have always placed a high value on the quality of life in Corvallis and the environment. The community has consistently shown a willingness to make capital and O&M investments which provide a proportionate environmental benefit; a willingness to do more than required by environmental regulations. Operation of the utility, for instance dechlorination, should be consistent with community values.

The utility administers a very efficient biosolids beneficial reuse program based on land application and soil enhancement. The program is well received by farmers and the community. Success of the program is dependent on maintaining this support. A continual outreach program to the farming community and diversifying the program by maintaining and increasing available dedicated land application sites under the control of the City is recommended.

As with liquid/hydraulic processes, O&M costs for biosolids processes can be reduced by the cost-effective implementation of new technologies when appropriate. Including periodically examining opportunities to increase digester gas utilization. Currently the plant uses a cost-effective and environmentally appropriate amount of available digester gas produced at the plant. Technology changes or community interests may encourage a greater proportion of digester gas to be used in the future. Periodically, the use of digester gas should be examined in the interest of increasing the amount that can be appropriately used.

The utility prides itself in being a good neighbor with respect to residents who must live or work near the wastewater facilities, in particular sewage lift stations and the wastewater reclamation plant. Operating personnel take very seriously all citizens' complaints and are diligent in responding to the complaints. As the community grows, increasingly it will be

necessary to make facility improvements to maintain this desirable good neighbor image. Periodic outreach to surrounding neighbors to make them aware of upcoming projects and address their concerns is encouraged. It is also recommended that the buffer around the wastewater reclamation plant, particularly to the east, be increased as residential parcels are made available for purchase.

Regulatory

The master plan has been prepared to enable the City to logically respond to the probability of more stringent environmental requirements. The timing and merits of more stringent requirements can't be fully predicted. However as a stakeholder affected by these future regulatory requirements utility personal should be active participants in National, State, and special committees and professional organizations influencing the development of the requirements. By being active participants, utility management will be in a more favorable position of assessing the impact to the Corvallis community, environment benefit and influencing the nature and timing of the requirements.

Stakeholders

Citizens, businesses, and public institutions all have a vested interest in the planning and operation of the wastewater utility. Through a proactive, concerted community outreach program aimed at creating understanding and informing, the stakeholders can take an informed role in making decisions and guiding the utility to ensure it's continued responsiveness to community values.

Fiscal

The City has maintained a fiscal program based on utility service rates, system development charges, and State and Federal grants or loans when available. Periodically, the City has reviewed the funding requirements and revenue sources to ensure an adequate source of funding to utility obligations. The most recent rate study should be reviewed and updated in light of capital improvements recommended in this master plan and with the goal of maintaining a reliable, predictable source of revenues. As part of the review, consideration should be given to establishing a dedicated source of funding to control I/I.

TABLE 9-1
Corvallis Wastewater Utility Master Plan
Revised Policies Listing
April 17, 1998

Policy	Enhance Existing	New	Benefit Derived
Manage the wastewater utility to protect public health, meet all regulatory requirements, and benefit the Willamette River water quality commensurate with other environmental, public, and fiscal requirements	X		Cost-effective utility rates and fees commensurate with community values and regulatory compliance.

TABLE 9-1
Corvallis Wastewater Utility Master Plan
Revised Policies Listing
April 17, 1998

Policy	Enhance Existing	New	Benefit Derived
Collection System			
Maintain and/or increase efforts for I/I monitoring and source detection	X		Continual identification of I/I sources for consideration of possible removal and the long-term monitoring of system deterioration that can lead to increased levels of I/I.
Increase funding as necessary for sewer system rehabilitation to maintain and/or reduce current I/I levels	X		Avoidance of more costly methods to convey and treat I/I and potential regulatory violations.
Remove cost effective sources of inflow whether on public or private property	X		Enables I/I to be maintained most cost effectively to existing or lower levels, regardless of property ownership.
Implement a program of private lateral inspection and replacement, if needed, at time of mainline repairs		X	Enables laterals to be replaced most economically due to economy of scale and increases the overall effectiveness of City mainline repair/replacement projects.
Maintain combined sewers in all combined areas; discontinue the practice of installing storm sewers when replacing combined lines		X	With the addition of planned Combined Sewer Overflow facilities, the policy will provide the greatest water quality benefit by conveying and treating all stormwater up to the CSO design storm.
Evaluate the feasibility of implementing a program of private lateral inspection and replacement, if needed, at the time a home or business sells		X	The cost effective removal of I/I arising from defective laterals on private property avoiding more costly collection and treatment system improvements by the City and/or extending the service life of the utility infrastructure.
Ensure development standards incorporate the Wastewater Utilities Master Plan recommendations	X		Will ensure the long-term integrity of the Master Plan and the plan objectives of protecting human health and the environment will be achieved most economically.
Continually assess construction standards aimed at controlling I/I		X	Ensures that advancement in construction materials and workmanship for newly developed areas will have I/I levels at or below that planned for in the Master Plan.
Evaluate implementing a contractor 1-year warranty on all new home or commercial construction for laterals		X	Will help ensure laterals will be constructed to City standards and limit I/I from private property.
Acquire necessary land for future facilities, i.e. pump stations		X	Ensures the Master Plan integrity allowing the collection system to be developed most economically.
Ensure development standards provide adequate setbacks in environmentally sensitive areas to allow subsequent construction and maintenance of wastewater utilities	X		Ensures the Master Plan recommended improvements can be environmentally permitted, constructed and maintained cost effectively.

TABLE 9-1
Corvallis Wastewater Utility Master Plan
Revised Policies Listing
April 17, 1998

Policy	Enhance Existing	New	Benefit Derived
Provide prioritized protection of environment and personal property from flooding up to a design storm condition of 1/5 years		X	Balances and prioritizes the protection of environment, human health, and property against the higher cost associated with implementing facilities affording a greater level of protection.
Provide protection of sanitary sewage overflows into receiving streams and rivers up to a design storm condition of 1/5 years		X	Provides environmental protection consistent with State regulations, hence protection against enforcement action.
Wastewater Reclamation Plant Liquids			
Incorporate automation and energy conservation measures in all capital improvements and equipment replacement that reduce labor and minimize O&M costs	X		Cost effective control of operation and maintenance costs.
Implement near term projects using Operations budgeting if necessary	X		Provides the flexibility to implement cost effective Master Plan improvements sooner than otherwise might be possible.
Provide secondary effluent de-chlorination when environmentally beneficial		X	Reduces the potential the effluent may be harmful to aquatic species or hinder fish migration.
Periodically evaluate impacts to WWRP of changing plant operations and regulations	X		Maintains favorable permit compliance record and the implementation of capital improvements on a schedule which is most cost advantageous.
Incrementally implement non-process facility improvements as part of other WWRP improvement projects		X	Enables favorable maintenance, laboratory, and operations space needs to be met most cost effectively.
Biosolids			
Make cost effective capital improvements that reduce O&M costs	X		Cost effective control of operation and maintenance costs.
Maintain the cost effective, environmentally beneficial land application biosolids reuse program	X		Provides for the beneficial safe and economical reuse of biosolids removed from the wastewater.
Maintain a biosolids outreach program to farmers	X		Helps ensure the continued support, viability, and reliability of the biosolids land application program.
Maintain and utilize the City's dedicated biosolids disposal site	X		Preserves the site for periodic use or use in an emergency back-up situation in the event conditions preclude application to permitted agricultural sites.
Continually evaluate options for increasing the availability of dedicated land application sites		X	Will help insure the reliability and dependability of the biosolids land application program.

TABLE 9-1
Corvallis Wastewater Utility Master Plan
Revised Policies Listing
April 17, 1998

Policy	Enhance Existing	New	Benefit Derived
Periodically re-assess additional digester gas beneficial use options	X		Will enable the City to assess emerging technologies and if they are implemented when feasible, they can reduce air emissions and decrease plant energy costs.
Environmental Adjacency			
Incrementally implement odor control measures w/ other improvement projects		X	Enables odor control to be implemented most cost effectively leading to a decrease in odor complaints and greater public acceptance of the facility operations.
Continue to evaluate disinfection alternatives in response to technology improvements, regulations, or community values			
Make aesthetic improvements along the south and west property lines to screen and/or mass facilities		X	Improving the appearance of the facility through economical landscaping will improve the overall image and public acceptance of the facility.
At least annually outreach to adjoining neighbors	X		Outreach to neighbors will strengthen the relationship between operators and neighbors and understanding of each parties concerns and interests.
Provide periodic neighborhood briefing(s) for upcoming WWRP projects		X	Keeping the neighbors appraised of upcoming projects, potential impacts, and efforts to eliminate or mitigate the construction impacts will build goodwill.
Implement a proactive neighborhood issues/response program	X		Timely response and follow up to all neighborhood complaints will build goodwill and ensure adjacency impacts are addressed.
Consider purchasing residential property(s) to the northeast should they become available		X	Facility master plan build out plans call for expanding the facility to the east and northeast. Purchasing of residential property to the east should it become available may reduce or eliminate more costly facility mitigation expenditures as the facility expands.
General Regulatory			
Continue to monitor and participate in processes that involve discharges to the Willamette River Basin		X	Participation and influence in the development of sound, regulatory policies affecting discharges
Participate in national wastewater management, regulatory and technical organizations, i.e. AMSA, WEF	X		Participation in such organizations will enable the utility to be aware of and influence trends in the wastewater profession which may be advantageous to Corvallis.

TABLE 9-1
Corvallis Wastewater Utility Master Plan
Revised Policies Listing
April 17, 1998

Policy	Enhance Existing	New	Benefit Derived
Participate in State water quality agencies/activities, i.e. ACWA and trienial standards review process	X		Similarly, participation in State level organizations and regulatory policy setting processes can have a pronounced impact on formulation of regulatory policies and regulations having a direct impact on Corvallis.
Participate in special committees/task forces, i.e. Willamette River task force	X		Similar influence over policies and regulations covering environmental issues affecting Corvallis can result from committee or task force participation.
Maintain favorable relationships with DEQ/EPA	X		Favorable relationships with regulators will help ensure fair, reasonable application of regulatory requirements.
Maintain an "environmental stewardship" role by making cost-effective expenditures which provide a proportionate environmental benefit	X		Will help ensure the utility is administered consistent with Corvallis community values and the preservation and/or enhancement of the Corvallis environment.
Stakeholders			
Maintain/enhance the utility's community understanding, education, and outreach program		X	The policy will help the community make informed decisions to assist the utility balance level of service commensurate with environmental or public benefit.
Ensure the community outreach program reaches and involves the multiplicity of stakeholders		X	All stakeholders must be informed and involved in decision making to gain widespread support for utility practices and acceptance of utility rates.
Fiscal			
Commit and budget for a dedicated source of funding to control I/I		X	Will ensure funds are annually available to control and/or reduce I/I to achieve cost effective conveyance and treatment of wet weather flows and maintain regulatory compliance.
Maintain a steady, predictable rate of utility rate increases as necessary to support wastewater utility infrastructure needs	X		Provides a favorable environment for businesses to make informed decisions and elected officials to meet community expectations.
Periodically review the sources of utility revenue to meet capital and operating requirements and to maintain an appropriate reserve	X		Provides for a fair equitable source of revenue from all utility stakeholders to match expenditures.
Consider expanding the wastewater System Development Charge program to include WWRP improvements that increase capacity		X	More equitable allocation and recovery of utility costs.

SECTION 10

Public Outreach and Community Values Program

Implementation of the master plan is dependent on ensuring capital improvements and utility policies are responsive to community values and supported by the community. The citizens of Corvallis take great pride in the quality of life available in their community. As a result, they take an active role in the planning and operations of all City services and infrastructure. Appreciative of this, the wastewater utility management seeks community understanding and input about wastewater utility projects. For instance, outreach programs were conducted regarding the selection and adoption of the $30 million Combined Sewer Overflow (CSO) Remediation Program and addressing citizen's concerns regarding flooding caused by the severe precipitation events of 1996. Community outreach and input have been sought throughout the development of the master plan. Initially this was done through the Mayor-appointed Water Infrastructure Committee (WIC) and subsequently through community outreach meetings, the Council Urban Services Committee (USC), and City Council. Table 10-1 lists the outreach activities associated with preparation of the master plan. Following completion of the master plan outreach activities, incorporation of community input, and approval from the City Planning Commission, the City Council unanimously adopted the master plan.

TABLE 10-1
Public Outreach Summary

Activity	Date	Participation	Comments
1. Water Infrastructure Committee	1/9/97	Full Committee (8)	Endorsed Outreach Plan
2. City Council	3/16/98	Full City Council	Endorsed Outreach Plan
3. First Community Open House	4/22/98	Citizens	See description in this section and Appendix D
4. Urban Services Committee	5/26/98	Full Committee (3)	
5. Second Community Open House	6/9/98	Citizens	See description in this section and Appendix D
6. Urban Services Committee	6/22/98	Full Committee (3)	
7. Planning Commission	8/19/98	Full Commission	Approved unanimously
8. City Council	9/21/98	8 of 9 Councilors present	Approved unanimously

For the first community open house, a direct mailing was sent to 95 individuals, neighborhood associations, or organizations soliciting their attendance at community outreach meetings and their input for the preparation of the master plan. For the second meeting, 417 direct mailings were sent. In addition to direct mailings to individuals and organizations, notices of master plan public meetings were advertised in the Gazette Times

on three dates, in the City Newsletter, in the City activity calendar, and on the TCI public access channel. Copies of direct mailing addresses, advertisements and other outreach materials are contained in Appendix G. Also included in Appendix G are the staff reports to the City Urban Services Committee following each public outreach meeting.

At each outreach activity, presentations were provided by utility personnel and consultant staff. Attendees asked specific question, and an opportunity was provided for attendees to document their input, either at the meeting or afterwards by mailing their comments and suggestions to the utility. Copies of the Citizen response forms that were provided at the meetings are included in Appendix G.

Despite extensive community notification, the initial master plan public meeting was lightly attended. The meeting agenda encompassed the master plan objectives, utility background, planning and regulatory criteria, and conveyance and treatment capacity deficiencies. Deficiencies were presented for existing population levels as well as future increases. Community input was sought on potential solutions to capacity limitations. Those in attendance speculated the light attendance was partly attributed to the utility's ongoing concerted outreach efforts associated with prior specific wastewater utility issues, i.e., flooding that occurred in 1996. Comments received from attendees were generally aimed at a particular problem that they had experienced with their wastewater service, e.g., basement flooding, or on the planning criteria, in particular planning for more stringent environmental requirements and the appropriate technology that should be evaluated. Public works staff are responding to and addressing specific concerns expressed by citizens about their wastewater service.

The second community open house presented the range of alternatives being considered for capacity deficiencies, costs, non-monetary considerations, prioritization and implementation schedules, and recommended utility policies to ensure the master plan objectives are fully achieved. Because the first community open house was so lightly attended, considerable additional efforts were expended to attract more citizens to the second open house. However, attendance at the second open house was also low. The feedback received varied, but was supportive of the planned major improvements to protect human health and the environment and fiscal planning. Considerable supportive discussion was held concerning the proposed prioritization and schedule for implementation of the wastewater utility improvements. Specific questions were raised about west Corvallis service, appropriate technologies for varying levels of effluent treatment, biosolids beneficial reuse, and severity of infiltration and inflow in the wastewater utility.

In summary, a concerted effort was made to obtain community input and guidance into the development of the master plan. Feedback received has helped guide the master plan. Specific questions raised at each outreach meeting were addressed by staff and consultants present. As appropriate, City staff are following up with specific solutions to ensure adequate wastewater service at residences and businesses. Citizen comments and suggestions from the outreach meetings have been integrated into the master plan.

SECTION 11

Wastewater Management Program Summary

Table 11-1 is a combined summary of collection system and wastewater reclamation plant improvements and program costs anticipated to be necessary to serve short-term needs and population projections of 60,000 and 80,000. Short-term improvements are those that should be included in the City's next 5-year Capital Improvement Program. A detailed discussion of the methodology to determine the improvements has been presented in prior chapters.

All the cost estimates have been prepared for guidance in project evaluation and implementation from the information available at the time of the estimate. The final costs of the projects will depend on actual labor and material costs, competitive market conditions, final project scope, implementation schedule, and other variable factors. As a result, the final project costs will vary from the estimates presented herein. Because of this, project feasibility and funding needs must be carefully reviewed before making specific financial decisions to help ensure proper project evaluation and adequate funding. It is normally expected that a cost estimate of this kind will be accurate within +50 percent or –30 percent. The range is based on statistical information covering a wide range of projects.

The project costs shown include 25 percent construction contingency and an allowance of 25-percent for engineering, legal, and administrative services necessary to implement the projects. Costs are based on an *Engineering News Record* construction cost index (CCI) of 6500 for the Seattle area.

Key information associated with some of the collection system projects follows. The timing of collection system projects can be significantly affected by the rate and location of population growth in the City and the City's efforts to control infiltration and inflow, either within the existing system or from newly developed areas.

Marys River Pump Station: Construction of the recommended pump station is a short-term need to prevent sanitary sewer overflows. Costs shown assume a new submersible type pump station with a structure large enough to serve a South Corvallis population associated with 120,000 persons will be initially constructed but only pumps to serve up to the 60,000 planning criteria will be installed initially. Additional pumps will be added to meet the 80,000 and 120,000 criteria.

Brooklane Pump Station: construction of the recommended new submersible type pump station must occur before 2010 to prevent sanitary sewer overflows consistent with DEQ regulations. This is likely to occur before the 60,000 planning criterion is reached. In the interim, conveying of wastewater from this pump station during large storm events will necessitate both existing pumps operate. (Generally one pump will be held in reserve should the other fail.)

South Corvallis Parallel Interceptor: construction of the new Marys River pump station allows the parallel interceptor to be deferred until the 80,000 planning horizon. The depth of the interceptor should be evaluated during the project predesign to confirm the viability of eliminating the Herbert Street pump station.

TABLE 11-1
Capital Project Summary

Projects	Total Project Costs[a]			
	Short-Term CIP	Population 60,000	Population 80,000	Total (All Phases)
Collection System Projects				
Marys River Pump Station	$3,320,000	$340,000	$510,000	$4,170,000
Brooklane Pump Station		$3,320,000	$490,000	$3,810,000
South Corvallis Parallel Intercept & Divers			$12,410,000	$12,410,000
Herbert Pump Station High Flow Bypass	$30,000			$30,000
Marys River Interceptor			$1,200,000	$1,200,000
Country Club Trunk			$800,000	$800,000
North Ninth Street Interim Grinder Pump Station	$40,000			
North Ninth Street Diversion		$1,570,000[b]		$1,570,000
Garfield Relief Sewers	$110,000	$90,000	$330,000	$530,000
Crescent Valley Interceptor			$2,300,000	$2,300,000
Subtotal Collection System Projects	***$3,500,000***	***$5,320,000***	***$18,040,000***	***$26,820,000***
WWRP Projects				
Influent Pumping/Headworks			$2,606,200	$2,606,200
Primary Treatment		$3,000,000	$1,687,500	$4,687,500
Biological Treatment		$406,200	$3,662,500	$4,068,700
Secondary Clarification		$4,237,500	$2,250,000	$6,487,500
Disinfection		$2,380,000		$2,380,000
Filtration			$10,856,300	$10,856,300
Effluent Pumping			$1,900,000	$1,900,000
Solids Handling	$4,150,000		$4,300,000	$8,450,000
Odor Control		$968,800	$150,000	$1,118,800
Non Process Facilities	$179,100	$221,000	$187,500	$587,600
Subtotal WWRP Projects	***$4,329,100***	***$11,213,550***	***$27,600,000***	***$43,142,600***
Total Project Cost	**$7,829,100**	**$16,533,500**	**$45,640,000**	**$69,962,600**

[a] Cost estimates include 25 percent construction contingency and 25 percent allowance for engineering, legal, and administrative costs. They do not include seismic improvements. Cost estimates are in 1998 dollars indexed to the *Engineering News Record* construction cost index of 6500 for the Seattle area.

[b] Postponement of diversion to potentially 60,000 population is dependent on the installation of an interim grinder pump station to serve a single existing business.

Herbert Street Pump Station: the cost and the useful life of the high flow bypass improvement at to the pump station should be weighed against the schedule for the South Corvallis parallel interceptor and additional cost of constructing the interceptor deeper so as to eliminate the pump station. The bypass is a short-term need required well in advance of the South Corvallis parallel interceptor to meet the DEQ regulations by 2010.

Country Club Trunk: Increased capacity at the Brooklane pump station reduces deficiencies in the Country Club trunk and allows improvements to be deferred until the 80,000 planning horizon.

North Ninth Street Diversion: Predicted surcharging of the Ninth Street sewer has been validated by flooding occurring at a business along Ninth Street during an extreme storm event. If an interim solution can be implemented for the one business affected, it may be possible to defer the diversion improvement for several years. This would require the identification and isolation of properties affected by high water surface elevations within the system. The installation of an interim grinder pump station at a selected location(s) may provide the protection necessary to defer this project. In which case, deferral to about 60,000 population may be possible. If deferred, it should be considered an early, priority 60,000 population project.

Garfield Relief Sewers: several small projects are required during the planning horizon to alleviate isolated surcharge conditions spread throughout the subbasin interceptor.

Key information associated with some of the projects for the wastewater reclamation plant follows. The timing of plant projects can be significantly affected by the rate of population growth in the City, City's efforts to control infiltration and inflow, and changes in regulatory environmental requirements.

Influent Pump Station: costs shown assume the existing pump station, in conjunction with a small "peaking" pump station, can be increased to a capacity of about 60 mgd. Additional study and potential scale model testing of the pump station will be necessary to confirm it's expansion potential.

Primary Treatment and Secondary Clarification: the timing and number of additional clarifiers will largely be governed by the effectiveness of infiltration and inflow control activities. At minimum, one new clarifier will be needed when the population reaches 60,000.

Biological Treatment: to serve the 60,000 population criteria, improvements to the trickling filter recycle pump station are required. Prior to the biological treatment process expansion to serve the 80,000 criteria, a reassessment of the potential for effluent nutrient limits for the plant effluent should be made. If nutrient removal is imposed or appear likely, then movement away from trickling filters/solids contact to a full suspended growth activated sludge process should be considered. In which case, biological treatment expansion costs would be higher than shown.

Disinfection: construction of a new chlorine contact basin is required to serve a population of 60,000 and cost-effectively meet the anticipated *E. coli* effluent bacteria standards. However, deferring construction of the chlorine contact basin until such time will be dependent on effectiveness of using chlorination/dechlorination to achieve bacterial effluent limits in

the existing hydraulically limited basin. If found to be ineffective, the new basin will need to be constructed sooner.

Filtration: the plan assumes some increase in the WWRP effluent mass limits for $CBOD_5$/ TSS will be achievable however not sufficiently to allow a population of 80,000 to be served without the addition of a filtration process.

Effluent Pump: a new effluent pump station will be required at which time the discharge to Dixon Creek under high flow and high river stage conditions is unacceptable. Given the recent improvements to the existing effluent pump station that have been implemented and the relatively infrequent discharge to Dixon Creek, it is assumed a new pump station won't be required until a planning population of 80,000 is reached.

Solids Handling and Odor Control Improvements: short-term solids handling and odor control improvements are currently being implemented. Improvements consist primarily of gravity thickener conversions and a new primary digester, both with appropriate odor control technologies. Additional improvements will be required in the future.

Non-Process Facility Improvements: Short-term non-process facility improvements focus primarily on renovation of the chlorination area in the existing Administration Building which being vacated as a result of the new sodium hypochlorite disinfection facilities being constructed as a part of the CSO improvements. The existing chlorinator room is to be converted to much-needed office space and records storage for the laboratory. This will also provide space in the existing laboratory for additional work area and one additional fume hood. The chlorine storage area is to be converted to an instrument shop and spare parts storage. The existing instrument shop and the associated parts storage area is being displaced by the solids handling modifications and the CSO improvements, respectively. The costs shown also address new facilities for housing automated control and monitoring system equipment and personnel, expanded toilet and locker facilities, and additional laboratory space.

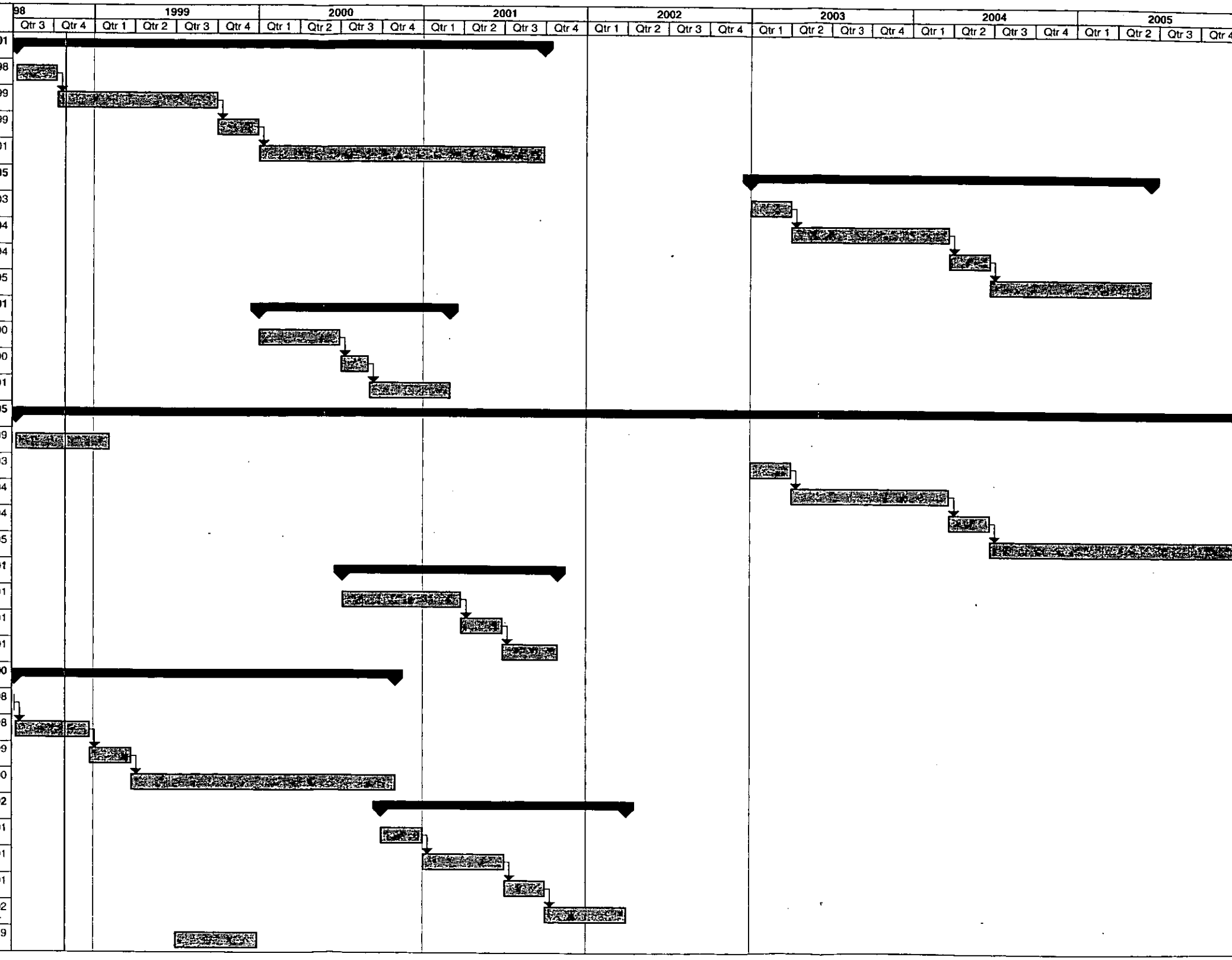
In addition to the projects shown in Table 11-1, seismic improvements to facilities at the WWRP should be implemented in the short-term time frame, the costs of which cannot be estimated at this time and therefore are not shown in Table 11-1. Potential corrective actions or further analysis required have been identified in Section 7. Costs, priorities, and the implementation schedule for the seismic improvements can be estimated after further analysis and code requirements are performed. Further analysis is recommended next fiscal year.

The schedule for implementing the short-term (CIP) recommended improvements is shown in Figure 11-1. The major phases of each project, consisting of professional services procurement, design, construction advertising and award, and construction, are shown for each project. The improvements need to be designed and constructed before the associated population levels are projected to occur so that the system is ready to serve the City's customers. Additionally, the professional services procurement and design of the new Brooklane pump station project is shown because of its need to be operational by 2010. Annually, the listing of projects should be reviewed and updated to reflect the actual implementation status of each project as well as need to begin the implementation of projects shown for the new level of population to be served as appropriate, i.e., 60,000 population criteria projects.

Sewer utility management and policy recommendations to support achieving the master plan objectives are presented in Section 9. Development of the recommended policies, either new ones or enhancement of existing policies, should be undertaken by the utility management with support and approval from the City Council. Periodic review of the policies is appropriate given the potentially changing community values and regulatory environment.

Figure 11-1
Short-term Capital Improvement Projects
Corvallis Wastewater Utility Master Plan

ID	Task Name	Duration	Start	Finish
1	**Marys River Pump Station**	**841 days**	**Fri 7/10/98**	**Fri 9/28/01**
2	Professional Services Procurement	66 days	Fri 7/10/98	Fri 10/9/98
3	Design	254 days	Mon 10/12/98	Thu 9/30/99
4	Advertisement/Award	66 days	Fri 10/1/99	Fri 12/31/99
5	Construction	455 days	Mon 1/3/00	Fri 9/28/01
6	**Brooklane Pump Station**	**640 days**	**Wed 1/1/03**	**Tue 6/14/05**
7	Professional Services Procurement	66 days	Wed 1/1/03	Wed 4/2/03
8	Design	254 days	Thu 4/3/03	Tue 3/23/04
9	Advertise/Award	66 days	Wed 3/24/04	Wed 6/23/04
10	Construction	254 days	Thu 6/24/04	Tue 6/14/05
11	**Herbert Pump Station High Flow Bypass**	**305 days**	**Mon 1/3/00**	**Fri 3/2/01**
12	Design (Assumed by City Staff)	130 days	Mon 1/3/00	Fri 6/30/00
13	Advertise/Award	45 days	Mon 7/3/00	Fri 9/1/00
14	Construction	130 days	Mon 9/4/00	Fri 3/2/01
15	**North Ninth Street Diversion**	**1950 days**	**Fri 7/10/98**	**Thu 12/29/05**
16	Interim Grinder Pump Station(design/const.)	150 days	Fri 7/10/98	Thu 2/4/99
17	Professional Services Procurement	66 days	Wed 1/1/03	Wed 4/2/03
18	Design	254 days	Thu 4/3/03	Tue 3/23/04
19	Advertisement/Award	66 days	Wed 3/24/04	Wed 6/23/04
20	Construction	396 days	Thu 6/24/04	Thu 12/29/05
21	**Garfield Relief Sewer**	**342 days**	**Fri 7/7/00**	**Mon 10/29/01**
22	Design (assumed by City staff)	188 days	Fri 7/7/00	Tue 3/27/01
23	Advertisement/Award	66 days	Wed 3/28/01	Wed 6/27/01
24	Construction	88 days	Thu 6/28/01	Mon 10/29/01
25	**Solids Handling/Odor Control**	**607 days**	**Tue 7/7/98**	**Wed 11/1/00**
26	Professional Services Procurement (completed)	1 day	Tue 7/7/98	Tue 7/7/98
27	Design	118 days	Fri 7/10/98	Tue 12/22/98
28	Advertisement/Award	66 days	Wed 12/23/98	Wed 3/24/99
29	Construction	420 days	Thu 3/25/99	Wed 11/1/00
30	**Non-Process Facilities**	**391 days**	**Mon 10/2/00**	**Mon 4/1/02**
31	Professional Services Procurement	66 days	Mon 10/2/00	Mon 1/1/01
32	Design	130 days	Tue 1/2/01	Mon 7/2/01
33	Advertise/Award	65 days	Tue 7/3/01	Mon 10/1/01
34	Construction	130 days	Tue 10/2/01	Mon 4/1/02
35	**Seismic Analysis, Phase 2**	130 days	Thu 7/1/99	Wed 12/29/99



Project: SCH0707D
Date: Wed 10/28/98

Task | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Milestone | Rolled Up Progress | External Tasks | Project Summary | Split | Rolled Up Split

CORVALLIS
ENHANCING COMMUNITY LIVABILITY

CITY OF CORVALLIS

Wastewater Utility Master Plan

APPENDICES

REGISTERED PROFESSIONAL ENGINEER 10519 OREGON JULY 13, 1979 GORDON A. NICHOLSON

10/98

CH2MHILL

NOVEMBER 1998

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E Present-Worth Cost Estimates for Treatment Facilities
F Seismic Hazard Evaluation Report
G Public Outreach Materials
H Abbreviations and Acronyms

APPENDIX A

Technical Memorandum 2.4—Infiltration and Inflow Analysis

TECHNICAL MEMORANDUM 2.4

CH2M HILL

PREPARED FOR: City of Corvallis

FROM: Gordon Nicholson

PREPARED BY: Joe Plaskett

REVIEWED BY: Mark Johnson
David Crawford

DATE: July 28, 1995

SUBJECT: Corvallis CSO Remediation Plan Rainfall Derived Infiltration and Inflow Analysis

PROJECT: 108086.B0.2C

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Executive Summary

Technical Memorandum (TM) No. 2.4, Corvallis CSO Rainfall Derived Infiltration and Inflow (RDII) Analysis, presents the results of the analysis of wet weather flows in the Corvallis separated sewer collection system. The purpose of the RDII analysis is to identify sewerage basins which are large contributors of RDII, to quantify these wet weather flows, and determine the impact on the Corvallis Waste Water Reclamation Plant (WWRP), combined sewer system, and combined sewer overflows (CSO). CH2M HILL has successfully implemented the RDII analysis in the past, and continues to use this analysis technique on other projects to accomplish these objectives. Based upon intensive flow monitoring during February, 1995, regression equations have been developed to predict flow based on rainfall and selected dry weather flow patterns. The flow estimates have been used in modeling efforts to generate design storm flows in the separated collection system. The RDII analysis identified basins which were potentially large contributors of RDII in the collection system. Based upon the results of this study it is recommended that a cost effectiveness analysis of RDII reduction alternatives be performed by the City. Additional information has been included to guide and support the City's RDII reduction efforts.

Flow monitoring was performed by ADS Environmental Services at 25 sites. Appendix A contains hydrographs for 25 temporary monitors between February 18 and March 13, 1995, and 9 permanent flow monitors for January 1 to March 31, 1995. Appendix B contains hydrograph plots generated by the regression analysis for each monitor for the study period. Tables 2.4.1 and 2.4.2 list RDII characteristics for each monitor, and by extension the sewershed contribution to that monitor.

Introduction

This task is part of the Combined Sewer Remediation Program of Corvallis. The estimates and analysis of RDII flows forms a basic component of the characterization of flows in the sewer system, particularly with respect to flows to the WWRP. Flows in the separated sewer system have a direct impact upon the combined system, and therefore affect CSO magnitude, frequency, and duration, and will affect sizing of CSO control alternatives.

The scope of this task includes the following activities:

- Review monitoring data and develop regression equations.
- Compute total million gallons of flow (base flow and RDII), dry weather flow (base flow) characteristics (quantity and diurnal pattern), and total million gallons of RDII.
- Determine percentage of peak RDII flow resulting from precipitation occurring in selected time periods prior to the peak flow, and peak to average flow rates for each storm event and each monitor.
- Prioritize monitored sewer-sheds (with the participation of City Personnel) to identify basins recommended for additional investigation and the type of investigation, and basins where no additional investigation is recommended.
- Summarize analysis results.

- Recommend appropriate additional investigations.

Figures 2.4.1 and 2.4.2 show the study area, major basins and flow monitored subbasins, flow monitor and rain gage locations used in this study.

Smoke Testing

Smoke testing was conducted in October, 1994 on approximately 16% of the Garfield basin after initial flow monitoring indicated high RDII quantities. The smoke testing identified a significant number of system defects and possible inflow or infiltration sources. A rehabilitation plan for a portion of the Garfield basin was developed based on the results of the smoke testing. At the request of the City, estimates of RDII quantities in this basin for a 5-year 24-hour winter storm were made from the smoke testing results. For the portion of the Garfield basin that was smoke tested, it was predicted that inflow to the system due to system defects would be approximately 600,000 gallons. The infiltration rate was not expected to increase. Results of this current analysis will give a more refined estimate of RDII in the basin.

Data Storage

The flow monitor and rain gage data used in this phase of the sewer system evaluation study were collected during the period of January 1, 1995 through March 31, 1995. Rain gage data from October and December of 1994 were also used. Data were supplied by the City in dBase IV database file format. A Microsoft Access database was developed to store and manage data. The database is backed up daily as part of CH2M HILL's standard network operations. This database is attached as computer files on floppy disks in Appendix D.

Data Verification

After importing the data files into the Access database, hydrographs were plotted for each of the flow monitors with precipitation data superimposed. The plots were then examined for indications of monitor downtime and suspect data. The Garfield Meter, Crescent Valley, Jefferson, and Fillmore permanent flow monitors showed missing or questionable data. Also, the rain gage at the wastewater treatment facility was not used because the gage was not operating for a portion of the study period. The remainder of the data was sufficient to complete a good quality analysis. Two periods of reliable flow monitor data were selected to generate the regression equations: one to represent typical dry weather flow, and the other to represent flow response to rainfall.

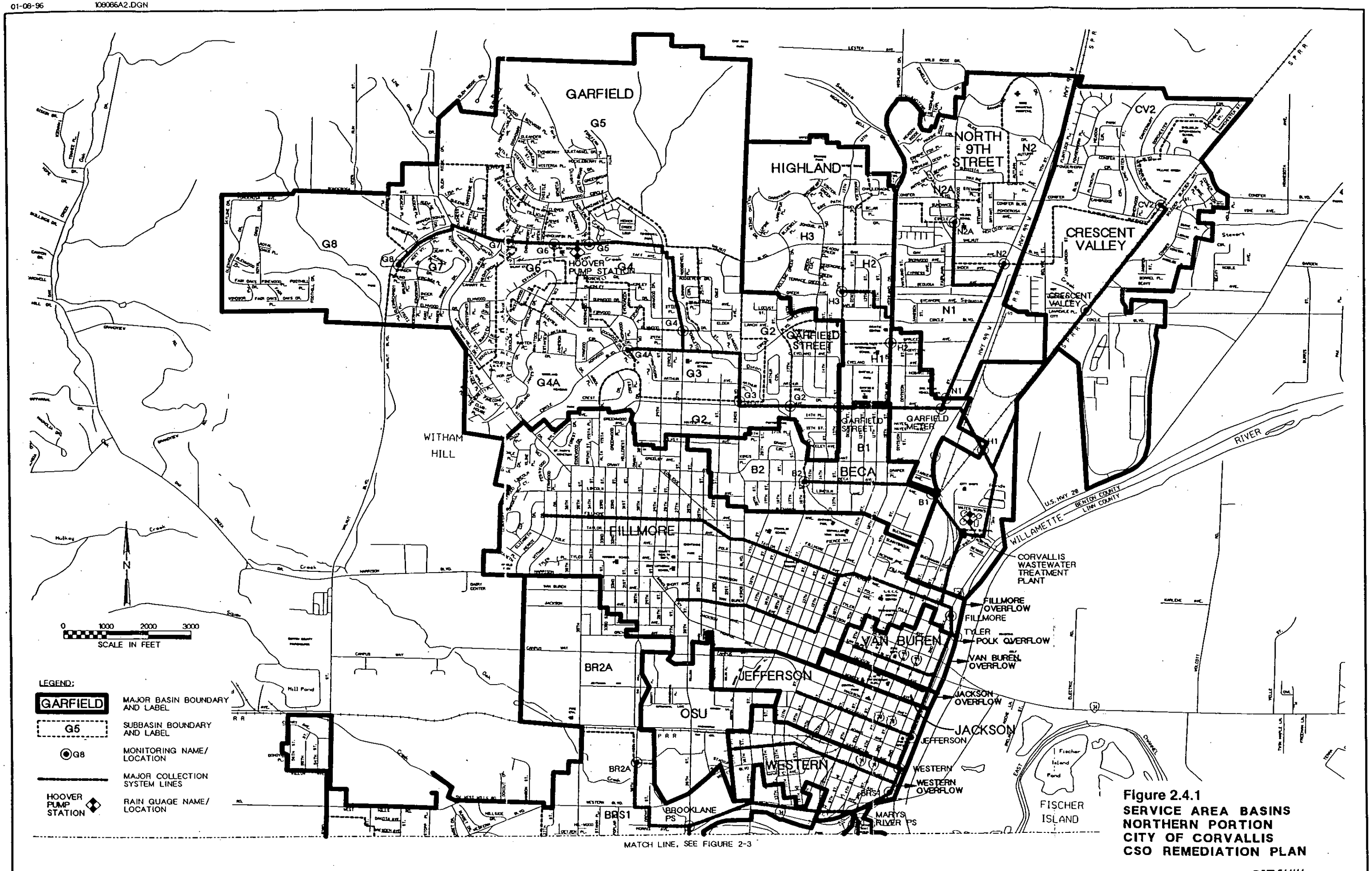


Figure 2.4.1
SERVICE AREA BASINS
NORTHERN PORTION
CITY OF CORVALLIS
CSO REMEDIATION PLAN

CH2M HILL

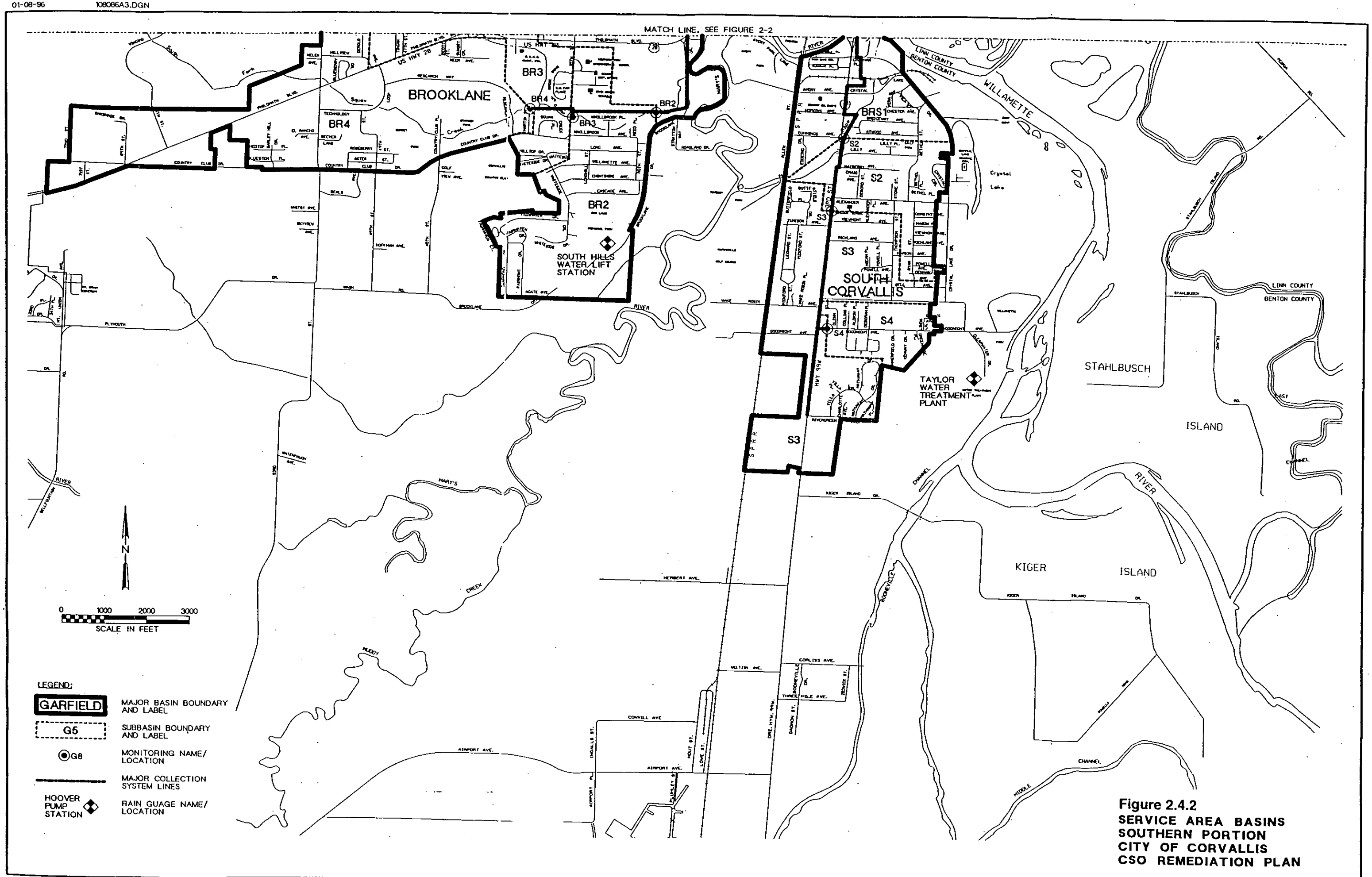


Figure 2.4.2
SERVICE AREA BASINS
SOUTHERN PORTION
CITY OF CORVALLIS
CSO REMEDIATION PLAN

CH2M HILL

Storm Selection

Four distinct wet weather rainfall events were recorded. Storms were selected visually from plots of precipitation data and from hydrographs showing flow response to rainfall. Storm 1 began on January 6, 1995 and ended January 19, 1995; storm 2 began January 26, 1995 and ended February 2, 1995; storm 3 began February 15, 1995 and ended February 20, 1995; storm 4 began March 8, 1995 and ended March 16, 1995. Four storms were used in the RDII analysis in order to obtain the collection system's response to a variety of storm patterns. Antecedent conditions, soil moisture and other conditions affecting RDII are believed reflective of typical wet weather conditions. The system response is dependent upon these antecedent conditions.

Wet Weather Average Base Flow

The wet weather average base flow (ABF) at a flow monitoring site was developed by selecting seven days of flow data from a dry period (no precipitation) during the winter study period. An ABF hydrograph, composed of sanitary flow and base groundwater infiltration, is developed for each location. The composite 24-hour ABF hydrograph was created by determining the minimum flow for each hour from flow monitor data recorded over the dry days selected. The average base flow is used in the calculation of RDII.

Rainfall Dependent Infiltration and Inflow

RDII is the flow entering the sewer system as a direct result of rain. RDII increases total flow volume and peak flow, and consists of two components: infiltration, which slowly percolates into the collection system; and inflow, which reaches a peak shortly after rainfall intensity is greatest and falls off rapidly when rain subsides. Since the flow monitors directly measure total flow, RDII may be estimated by subtracting ABF from the total flow.

Flow Estimates

The flow monitor measures flow from all upstream sources, and in some instances is affected by backwater conditions from downstream conduits. To isolate RDII originating from a contributing area between two monitors, flows from upstream basins are subtracted from the flows measured at the downstream monitor. As is typical during any flow monitoring program, some of the monitors had unreliable or missing data. A method was employed to fill in these gaps and to replace unreliable data. This correction allowed the estimation of RDII for all of the basins for the four storms.

Estimates of RDII are based on a regression relationship between rainfall and flow. RDII is flow resulting from rainfall that has entered the collection system over the past hours, days

and weeks. For this analysis, it was assumed that a multiple linear relationship existed between RDII and past rainfall. Wet weather flow, ABF, and rainfall recorded at the nearest representative rain gage was extracted from the database and imported into an Excel spreadsheet. Rainfall during the 15 days (360 hours) prior to each hourly flow measurement was summed for the following 9 periods: 1 hour, 2 to 3 hours, 4 to 6 hours, 7 to 12 hours, 12 hours to 1 day, 1 to 2 days, 4 to 7 days, and 7 to 15 days. Excel was used to perform multiple linear regression on the correlation between the rainfall sums and the measured flow to determine the regression coefficients for each rainfall sum.

The regression equation has the form:

$$RDII = C1*Rain_{1hr} + C2*Rain_{2hr} + \ldots + Cn*Rain_{nhr}$$

with:

$$\text{Total Flow} = ABF + RDII$$

Figure 2.4.3 is an example of the results of this analysis. The analysis was done for each monitoring station using software that allows interaction between the Access database and Excel. Once the regression equations are developed and visually checked, flow may be estimated for any period by applying the equation to precipitation data collected at other periods or for specific design storm events. Figure 2.4.4 is an example of applying the regression equations to generate flow when monitor data is missing or unreliable. These regression equations have been used in CSO modeling efforts to generate design flow input to the collection system. The regression equations were also used to identify flow characteristics within a basin as described in the next section.

Accuracy of Flow Estimates

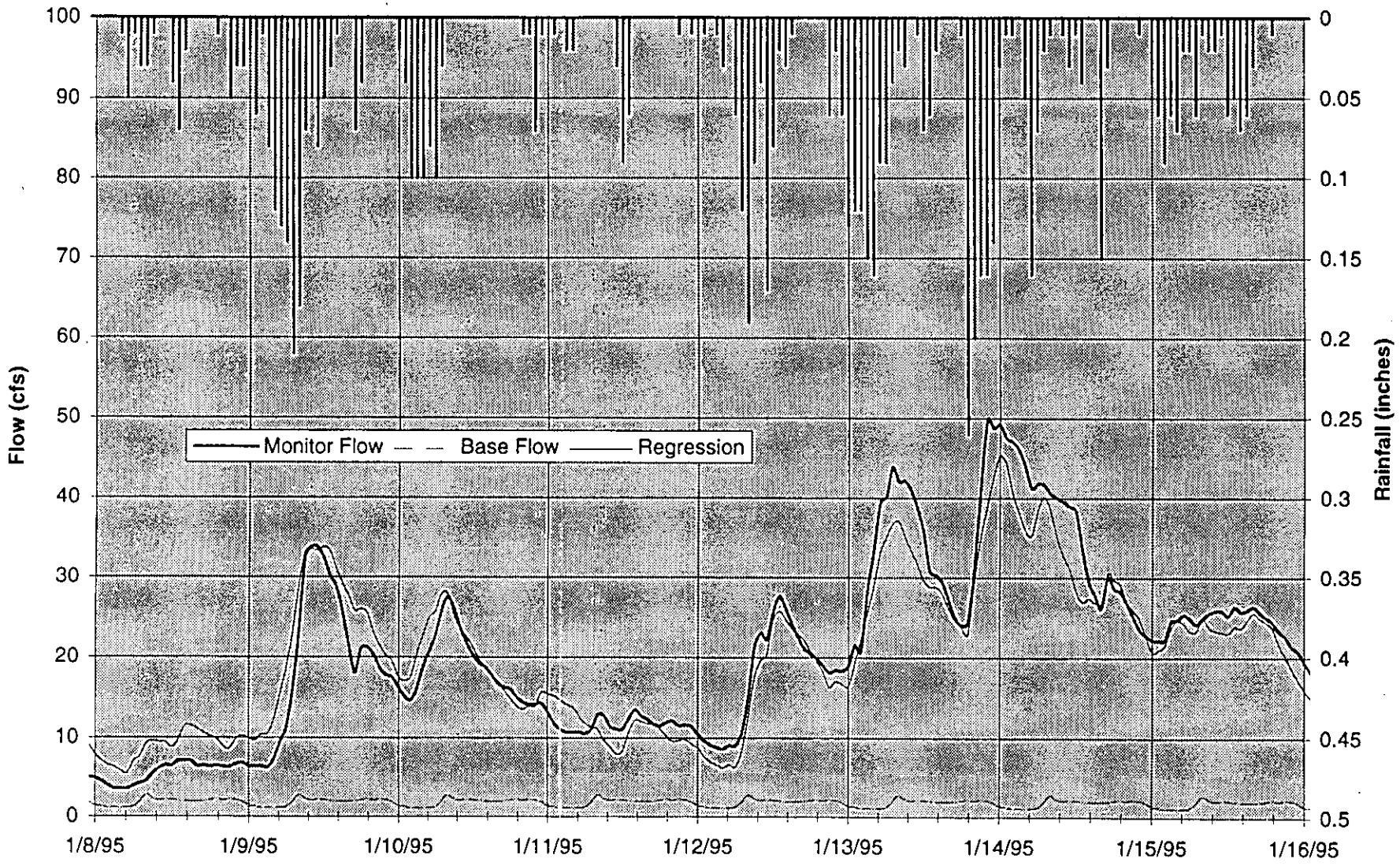
In order to verify that the regression equations produced accurate flow predictions, flow calculated from the regression equations were plotted and compared against measured flows for the four storms. The majority of the regression plots showed that the regression equations produced flow estimates that closely matched measured flows. Differences between flows computed from the regression equations and measured flows are a result of one or more of the following:

- System operation. The effects of flow diversions, pump stations, and wet weather bypasses are not consistent from storm to storm and result in potentially irregular system flows under similar rainfall events.

- Rainfall Distribution. The regression equations were generated from the rain gage that was thought to best represent the rainfall distributed over the entire monitor basin. However, variability of rainfall volume and intensity is normal across basins, resulting in differences in flow volume and timing.

Figure 2.4.3

Create Regression
Flow Monitor: MMGAM Rain Gage: HR



Equation: RDII = 29.34*1 hr + 28.94*3 hr + 27.11*6 hr + 17.95*12 hr + 8.99*24 hr + 4.24*48 hr + -0.09*96 hr + 1.11*168 hr + 0.15*360 hr

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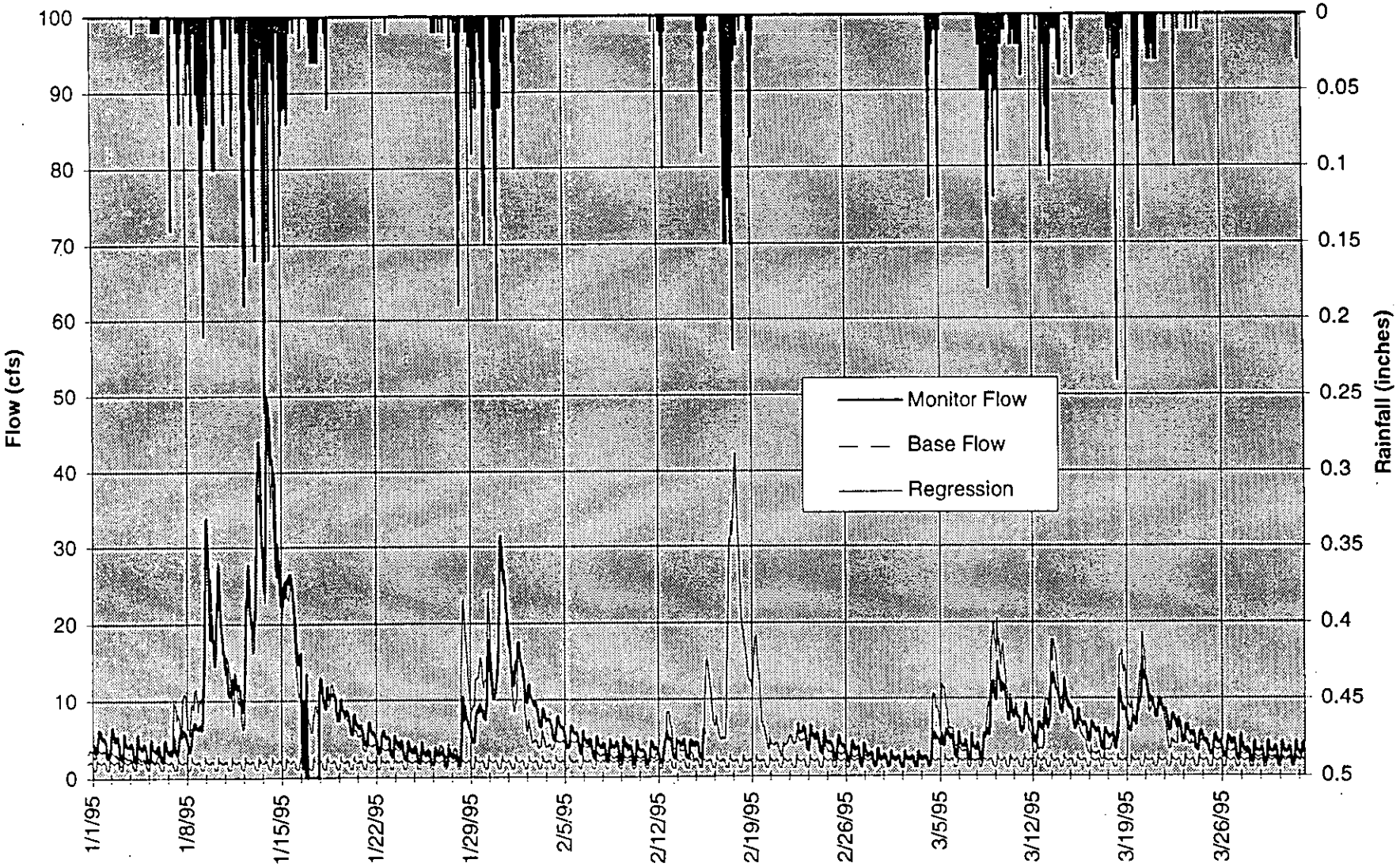
Figure 2.4.4

Apply Regression
Flow Monitor: MMGAM Rain Gage: HR



Equation: RDII = 29.34*1 hr + 28.94*3 hr + 27.11*6 hr + 17.95*12 hr + 8.99*24 hr + 4.24*48 hr + -0.09*96 hr + 1.11*168 hr + 0.15*360 hr

GFIELD.XLS

- Monitor Data. It is common to have intermittent problems with flow measurement, particularly due to mismeasurement of velocity. The velocity probe on a flow monitor can be fouled by debris and scum, or backwater effects can change the velocity to depth relationships. In addition, some monitors were not operating during portions of the monitoring period. The regression equations were produced from storm events during periods where the monitor data appeared to be the most reliable. The majority of the monitored data are reasonable and appropriate for the uses of this study.

- Antecedent Conditions. RDII predicted by the regression equations will be most accurate when applied to periods when the storm intensity, duration, and antecedent conditions are similar to those used to generate the regression equations.

Design Storms

The 5-year, 24-hour winter storm for the Corvallis area is 2.4 inches. A 25-year, 24 hour storm event was recorded at the Taylor Water Treatment Plant (TR) for the 24-hour period beginning January 12, 1995 at 11:00 p.m. The total precipitation measured during this time period was 3.28 inches. At the South Hills Water Lift Station (SR), 2.70 inches of precipitation was recorded for the same time period. This approaches the 10-year, 24-hour winter storm rainfall depth of 2.8 inches. The maximum 24-hour precipitation total for the Hoover Pump Station (HR) gage during the January 1 through March 31, 1995 study period was 2.22 inches for the 24-hour period beginning February 16, 1995 at 10:00 p.m. Using the regression equations, and the rainfall analysis (synthetic storms) for the Corvallis area previously completed for the City by CH2M HILL, flow estimates were calculated for selected basins for the 5-year, 24-hour winter storm, and are included in Appendix C. These flow estimates are based on an isolated 24-hour rainfall event, assuming normal winter base flow as the antecedent condition. Therefore, for the isolated storm event, it was assumed that there was no rainfall for at least seven days prior to this rainfall event, it rained for exactly 24 hours, then stopped. Under normal circumstances, the 5-year storm would not occur as an isolated event, but would be part of an overall storm pattern, and therefore different antecedent conditions, as can be observed from the plots in Appendices A and B. These plots show winter storm patterns within the City of Corvallis.

In order to demonstrate the system response to a more realistic case, plots are included in Appendix C which include a 5-year 24-hour storm event as part of an overall storm pattern. The storm pattern is simply that from the South Hills (SR) rain gage for the period January 6 to January 19, 1995, with rainfall values scaled downward by a factor which equates the peak 24-hour rainfall to a 5-year event. It will be noted that peak flows for this storm pattern are higher when compared to the isolated storm event.

Flows were estimated for a synthetic summer storm corresponding to 10-year 24-hour design storm, and are also included in Appendix C. It will be noted that the total precipitation of 1.7 inches for this summer storm is less than for the 5-year 24-hour winter design storm.

Basin Categories

To categorize the response of each monitored basin to wet weather events, the following calculations were made for each of the four storms:

- Total flow volume(base flow and RDII)
- Total RDII flow volume
- The ratio of peak instantaneous flow to ABF
- The percentage of RDII resulting from defined periods of prior rainfall

Sewer sheds monitored by a specific monitor (or series of monitors) were placed into categories to help direct source detection and remediation efforts. A sewer shed's conduits conveying a high percentage of flow resulting from precipitation that occurred days or weeks prior to the flow measurement indicates the contributing basin is receiving water from infiltration sources. These sources include defective joints in the sewer, broken and cracked pipe, improper connections, defects in manhole walls, etc. Table 2.4.1 shows the percentage of the total peak RDII flow that results from rainfall during each of three time periods prior to the peak flow: 1 through 6 hours, 7 through 48 hours, and 49 through 360 hours. A monitor that has a higher percentage of the peak flow derived from more recent precipitation (hours vs. days) indicates the basin is receiving water from inflow sources. These sources include catch basins, roof drains, foundation drains, open manhole covers, and other direct or illicit connections.

The following categories were developed to rank the basins by priority (Category 1 being the highest priority) for further study and source detection activities:

Category 1: Return > 20% - High Inflow and Infiltration Response

RDII volume greater than 20% of precipitation, 50% of peak flow results from precipitation occuring more than 6 hours prior to peak.

These basins "return" a high percentage of the rainfall falling on the basin and show a high inflow response. These basins were selected for additional analysis because of their high inflow and high infiltration.

Category 2: Return > 10 % - Inflow Response

RDII volume greater than 10% of precipitation volume, 50% of peak flow results from precipitation occuring within 6 hours of peak.

These basins have peak flows with more than 50 percent of the RDII peak flow resulting from precipitation within the previous 6 hours and a lower return percentage for RDII. These basins were selected for additional analysis because of their high inflow potential.

Category 3: Return < 10% - Inflow Response

RDII volume less than 10% of precipitation volume, 50% of peak flow results from precipitation occuring less than 6 hours prior to peak.

These basins have a relatively low "return" percentage for RDII with more than 50 percent of the RDII peak flow resulting from precipitation within the previous 6 hours. Basins fitting this profile have less priority for continued investigation.

Category 4: Return < 20% - Infiltration Response

RDII volume less than 20% of precipitation volume, 50% of peak flow results from precipitation occuring more than 6 hours prior to peak.

These basins have a relatively low "return" percentage for RDII and show low inflow response. Infiltration is a larger contributor than inflow, and therefore these basins are not critical contributors to the peak flow condition. Basins fitting this profile are not ranked as a high priority for continued investigation primarily because of their low inflow potential.

Category 5: Total Wet Weather Peak Flows < 1 MGD

Total flow less than 1.0 MGD

These basins are small contributors to peak flow and are not critical to reduction of the peak flow. There is less benefit in continuing to study these basins.

Category 6: Monitored Special Interest Basins

This category consists of basins which have had indications of RDII problems in the past or require further study to determine whether RDII problems exist. Inconclusive monitoring and data analysis did not allow for their placement in a specific category.

Permanent Monitors

These basins were monitored using different flow monitoring equipment than the other basins, and were therefore categorized separately. Basins within this group include monitored CSO basins and pump stations (PS).

Table 2.4.2 shows the basins categorized under each of the above screening criteria. Data generated by the regression equations were used in place of unreliable or missing flow monitor data for the Fillmore, Crescent Valley, Jefferson, and Garfield Meter basins in order to calculate RDII. Regression equations were used to generate data for all 25 of the temporary monitors due to the limited flow history available. Figures 2.4.5 through 2.4.7 show the basin categorizations graphically.

Recommendations

Additional field testing is recommended for those basins which are potentially a significant source of RDII. A smoke testing program, similar to that done by the city in October of

Table 2.4.1:
CORVALLIS CSO REMEDIATION PLAN
CONTRIBUTION TO PEAK FLOWS

	Storm 1, 1/6/95 - 1/19/95				Storm 2, 1/26/95 - 2/2/95				Storm 3, 2/15/95 - 2/20/95				Storm 4, 3/8/95 - 3/16/95			
STATION	Peak RDII	Time Period (hours)			Peak RDII	Time Period (hours)			Peak RDII	Time Period (hours)			Peak RDII	Time Period (hours)		
	(mgd)	1 thru 6	7 thru 48	49 thru 360	(mgd)	1 thru 6	7 thru 48	49 thru 360	(mgd)	1 thru 6	7 thru 48	49 thru 360	(mgd)	1 thru 6	7 thru 48	49 thru 360
Brooklane PS	6.69	54%	38%	8%	4.56	58%	27%	15%	5.02	55%	39%	7%	2.51	45%	55%	0%
Crescent Valley	1.75	54%	35%	10%	1.03	51%	33%	16%	1.22	43%	48%	9%	0.86	55%	45%	0%
Fillmore	31.70	70%	27%	4%	23.78	78%	18%	4%	31.44	65%	32%	2%	18.14	85%	13%	2%
Garfield Meter	19.67	53%	35%	12%	14.31	49%	36%	15%	16.69	52%	39%	9%	7.56	74%	20%	6%
Garfield Street	13.88	56%	37%	7%	9.68	60%	31%	10%	12.83	53%	42%	5%	5.62	75%	22%	4%
Jefferson	1.05	77%	23%	0%	0.86	88%	12%	0%	1.10	79%	21%	0%	0.65	93%	7%	0%
Mary's River PS	7.89	46%	46%	8%	5.09	55%	30%	16%	5.17	52%	41%	7%	2.67	33%	67%	0%
Tyler	16.83	49%	46%	5%	11.22	55%	37%	7%	15.71	46%	51%	3%	6.91	29%	71%	0%
Western	2.44	87%	13%	0%	1.89	97%	3%	1%	1.53	93%	7%	0%	1.22	49%	4%	0%
ADSB1	1.58	54%	41%	4%	1.00	71%	23%	6%	1.53	54%	43%	3%	0.72	40%	60%	0%
ADSB2	0.96	44%	45%	11%	0.67	48%	38%	14%	0.84	47%	45%	7%	0.38	33%	67%	0%
ADSBR2	2.32	29%	61%	9%	1.62	11%	67%	22%	1.61	23%	69%	8%	0.85	14%	86%	0%
ADSBR2A	0.91	51%	44%	5%	0.64	63%	28%	8%	0.69	52%	45%	4%	0.41	47%	53%	0%
ADSBR3	1.36	32%	60%	8%	0.87	11%	68%	21%	0.88	29%	63%	8%	0.52	15%	85%	0%
ADSBR4	0.94	8%	87%	5%	0.65	10%	68%	21%	0.59	22%	69%	9%	0.30	11%	89%	0%
ADSBRS1	11.79	44%	46%	10%	7.97	46%	35%	18%	8.01	48%	44%	9%	3.81	28%	72%	0%
ADSCV2	1.14	52%	44%	4%	0.80	57%	38%	5%	1.00	52%	45%	3%	0.44	34%	66%	0%
ADSG2	4.81	42%	51%	7%	3.26	47%	44%	10%	4.15	43%	52%	5%	1.87	26%	74%	0%
ADSG3	3.84	42%	50%	8%	2.66	46%	43%	11%	3.28	44%	50%	6%	1.46	25%	75%	0%
ADSG4	2.07	42%	50%	7%	1.41	47%	43%	10%	1.80	43%	51%	5%	0.80	27%	73%	0%
ADSG4A	1.05	44%	48%	9%	0.72	48%	41%	11%	0.86	46%	47%	6%	0.38	30%	70%	0%
ADSG5	1.60	40%	52%	8%	1.08	45%	44%	10%	1.40	41%	54%	5%	0.63	25%	75%	0%
ADSG6	1.12	14%	82%	5%	0.79	13%	72%	14%	0.96	39%	54%	6%	0.44	24%	76%	0%
ADSG7	0.75	11%	85%	4%	0.52	12%	77%	12%	0.66	3%	92%	5%	0.31	17%	83%	0%
ADSG8	0.48	38%	56%	6%	0.32	13%	76%	11%	0.45	10%	86%	4%	0.21	23%	77%	0%
ADSH1	9.60	49%	47%	5%	6.62	54%	40%	6%	8.60	49%	48%	3%	3.89	31%	69%	0%
ADSH2	0.78	51%	43%	6%	0.56	57%	36%	7%	0.78	49%	48%	4%	0.35	29%	71%	0%
ADSH3	0.53	60%	21%	19%	0.42	60%	19%	22%	0.47	61%	27%	13%	0.22	73%	15%	13%
ADSN1	2.28	53%	41%	6%	1.61	58%	34%	8%	2.10	51%	45%	4%	0.92	32%	68%	0%
ADSN2	0.57	52%	42%	6%	0.40	56%	36%	8%	0.53	52%	44%	4%	0.24	35%	65%	0%
ADSN2A	0.44	45%	48%	7%	0.30	49%	42%	9%	0.38	46%	49%	5%	0.17	28%	72%	0%
ADSS2	6.20	43%	52%	5%	2.48	82%	18%	0%	2.04	41%	57%	3%	1.17	24%	76%	0%
ADSS3	3.47	35%	61%	5%	1.19	35%	61%	4%	1.15	19%	79%	2%	0.69	24%	76%	0%
ADSS4	1.02	42%	55%	4%	0.40	93%	7%	0%	0.33	34%	64%	2%	0.20	25%	75%	0%

Table 2.4.2: Percentage Performance for Metered Basins
Corvallis CSO Remediation Plan

Basin Designation	Flow Monitor	Total Upstream RDI/I Volume (MG) Stm. 1	Stm. 2	Stm. 3	Stm. 4	Upstream Monitors	Local Subbasin RDI/I Volume (MG) Stm. 1	Stm. 2	Stm. 3	Stm. 4	Peak RDI/I Flow (mgd) Stm. 1	Stm. 2	Stm. 3	Stm. 4	ABF (MGD)	Rain Gage	Precipitation Volume (MG) Stm. 1	Stm. 2	Stm. 3	Stm. 4	Area (ac.)
CATEGORY 1: Return > 20 % - High Inflow and Infiltration Response																					
RDI/I VOLUME GREATER THAN 20% OF PRECIPITATION, 50% OF PEAK FLOW RESULTS FROM PRECIPITATION OCCURING MORE THAN 6 HOURS PRIOR TO PEAK																				Subtotals	152
South Corvallis	ADSS2	30.08	7.81	3.72	4.11	ADSS3	13.37	3.44	1.62	1.82	6.20	2.48	2.04	1.17	0.82	TR	44.9	13.0	6.2	6.4	152
CATEGORY 2: Return > 10% -- Inflow Response																					
RDI/I VOLUME GREATER THAN 10% OF PRECIPITATION VOLUME, 50% OF PEAK FLOW RESULTS FROM PRECIPITATION OCCURING WITHIN 6 HOURS OF PEAK																				Subtotals	669
Highland	ADSH1	47.28	17.22	16.66	14.90	ADSN1, ADSH2, ADSG2	6.20	2.69	2.61	2.04	9.60	6.62	8.60	3.89	2.18	HR	49.2	20.7	19.6	15.9	223
North Ninth	ADSN1	11.20	3.99	3.85	3.51	ADSN2, ADSN2A	5.23	1.73	1.77	1.41	2.28	1.61	2.10	0.92	0.54	HR	49.6	20.9	19.8	16.0	225
Crescent Valley	ADSCV2	5.43	2.01	1.94	1.72		5.43	2.01	1.94	1.72	1.14	0.80	1.00	0.44	0.36	HR	48.7	20.5	19.4	15.7	221
CATEGORY 3: Return < 10% - Inflow Response																					
RDI/I VOLUME LESS THAN 10% OF PRECIPITATION VOLUME, 50% OF PEAK FLOW RESULTS FROM PRECIPITATION OCCURING LESS THAN 6 HOURS PRIOR TO PEAK																				Subtotals	130
Beca	ADSB1	7.44	2.73	2.64	2.34	ADSB2	1.92	0.89	0.86	0.64	1.58	1.00	1.53	0.72	0.30	HR	28.7	12.1	11.4	9.2	130
CATEGORY 4: Return < 20% - Infiltration Response																					
RDI/I VOLUME LESS THAN 20% OF PRECIPITATION VOLUME, 50% OF PEAK FLOW RESULTS FROM PRECIPITATION OCCURING MORE THAN 6 HOURS PRIOR TO PEAK																				Subtotals	1596
Brooklane	ADSBR2	13.58	4.59	3.77	4.01	ADSBR3	5.85	1.94	1.58	1.72	2.32	1.62	1.61	0.85	0.60	SR	76.5	33.2	26.1	24.0	320
Brooklane	ADSBR3	7.73	2.65	2.19	2.29	ADSBR4	2.43	0.88	0.73	0.73	1.36	0.87	0.88	0.52	0.57	SR	60.5	26.2	20.6	19.0	253
Garfield	ADSG2	25.99	9.14	8.85	8.13	ADSG3	4.51	1.71	1.66	1.43	4.81	3.26	4.15	1.87	1.14	HR	34.6	14.6	13.8	11.2	157
Garfield	ADSG3	21.49	7.43	7.19	6.69	ADSG4, ADSG4A	4.43	1.50	1.45	1.37	3.84	2.66	3.28	1.46	1.03	HR	42.3	17.8	16.9	13.7	192
Garfield	ADSG4	11.26	3.94	3.81	3.52	ADSG5	2.45	0.86	0.83	0.77	2.07	1.41	1.80	0.80	0.56	HR	27.8	11.7	11.1	9.0	126
Garfield	ADSG5	8.81	3.08	2.98	2.75	ADSG6	2.32	0.85	0.82	0.73	1.60	1.03	1.40	0.63	0.47	HR	35.3	14.9	14.1	11.4	160
Garfield	ADSG6	6.49	2.23	2.16	2.02	ADSG7	2.37	0.78	0.76	0.73	1.12	0.79	0.96	0.44	0.35	HR	15.9	6.7	6.3	5.1	72
South Corvallis	ADSS3	16.72	4.37	2.10	2.29	ADSS4	12.31	3.20	1.53	1.68	3.47	1.19	1.15	0.69	0.67	TR	69.8	20.3	9.7	9.9	236
South Corvallis	ADSS4	4.41	1.17	0.56	0.61		4.41	1.17	0.56	0.61	1.02	0.40	0.33	0.20	0.13	TR	23.7	6.9	3.3	3.3	80
CATEGORY 5: Total Wet Weather Peak Flows Less than 1 mgd																					
TOTAL FLOW LESS THAN 1.0 MGD																				Subtotals	1714
Beca	ADSB2	5.52	1.85	1.78	1.70		5.52	1.85	1.78	1.70	0.96	0.67	0.84	0.38	0.11	HR	21.8	9.2	8.7	7.0	99
Garfield	ADSG4A	5.80	1.99	1.92	1.80		5.80	1.99	1.92	1.80	1.05	0.72	0.86	0.38	0.18	HR	33.5	14.1	13.4	10.8	152
Garfield	ADSG7	4.12	1.45	1.41	1.29	ADSG8	1.58	0.54	0.52	0.49	0.75	0.52	0.66	0.31	0.24	HR	43.2	18.2	17.2	13.9	196
Garfield	ADSG8	2.54	0.91	0.88	0.80		2.54	0.91	0.88	0.80	0.48	0.32	0.45	0.21	0.06	HR	3.7	1.6	1.5	1.2	17
Highland	ADSH2	3.89	1.40	1.35	1.22	ADSH3	0.68	0.45	0.44	0.26	0.78	0.53	0.78	0.35	0.21	HR	22.3	9.4	8.9	7.2	101
Highland	ADSH3	3.20	0.95	0.91	0.96		3.20	0.95	0.91	0.96	0.53	0.42	0.47	0.22	0.17	HR	37.9	16.0	15.1	12.2	172
North Ninth	ADSN2	3.61	1.43	1.28	1.36		3.61	1.43	1.28	1.36	0.57	0.40	0.53	0.24	0.09	HR	33.5	14.1	13.4	10.8	152
North Ninth	ADSN2A	2.36	0.83	0.81	0.74		2.36	0.83	0.81	0.74	0.44	0.30	0.38	0.17	0.09	HR	25.1	10.6	10.0	8.1	114
Brooklane	ADSBR2A	4.12	1.51	1.23	1.24		4.12	1.51	1.23	1.24	0.91	0.64	0.69	0.41	0.23	SR	49.5	21.5	16.9	15.5	207
Brooklane	ADSBR4	5.30	1.77	1.46	1.56		5.30	1.77	1.46	1.56	0.94	0.65	0.59	0.30	0.32	SR	120.4	52.3	41.1	37.8	504
CATEGORY 6: MONITORED SPECIAL INTEREST BASINS																				Subtotals	410
Brooklane/South Corvallis	ADSBRS1	65.67	21.90	17.92	19.33	ADSBR2A, ADSBR2, ADSS2	17.89	7.99	9.21	9.97	11.79	7.97	8.01	3.81	0.57	SR	98.0	42.5	33.4	30.7	410
PERMANENT MONITORS																				Subtotals	6168
Brooklane	Brooklane PS	32.35	12.00	8.84	8.08	ADSBR2, ADSBR2A	14.65	5.91	3.84	2.83	6.85	6.17	6.15	2.26	1.62	SR	47.8	20.7	16.3	15.0	200
Crescent Valley	Crescent Valley	7.76	2.45	2.37	2.37	ADSCV2	2.33	0.44	0.43	0.65	1.75	1.03	1.22	0.86	0.36	HR	67.0	28.2	26.7	21.6	304
Fillmore (CSO)	Fillmore	106.52	39.35	37.84	33.50		106.52	39.35	37.84	33.50	31.70	23.78	31.44	18.14	5.07	HR	373.5	157.3	149.0	120.5	1694
Garfield	Garfield Street	66.76	18.51	22.20	18.89		66.76	18.51	22.20	18.89	16.11	9.09	15.23	4.20	1.27	HR	253.8	106.9	101.3	81.9	1151
Garfield	Garfield Meter	113.72	35.29	34.91	32.57	Garfield Street	46.96	16.78	12.71	13.69	30.90	19.49	16.69	7.78	1.82	HR	183.7	77.4	73.3	59.3	833
Jefferson (CSO)	Jefferson	2.82	1.16	1.11	0.91		2.82	1.16	1.11	0.91	1.05	0.86	1.10	0.65	0.18	HR	37.5	15.8	15.0	12.1	170
Brooklane/South Corvallis	Mary's River PS	34.70	15.18	11.46	13.87		34.70	15.18	11.46	13.87	3.87	3.41	3.54	2.26	0.29	SR	321.9	139.7	109.7	101.0	1347
Tyler (CSO)	Tyler	36.82	31.16	22.59	25.15		36.82	31.16	22.59	25.15	9.81	12.33	8.47	7.36	1.06	HR	26.2	11.1	10.5	8.5	119
Western (CSO)	Western	12.29	2.48	4.33	6.64		12.29	2.48	4.33	6.64	6.98	2.47	4.50	2.35	0.68	SR	83.6	36.3	28.5	26.2	350

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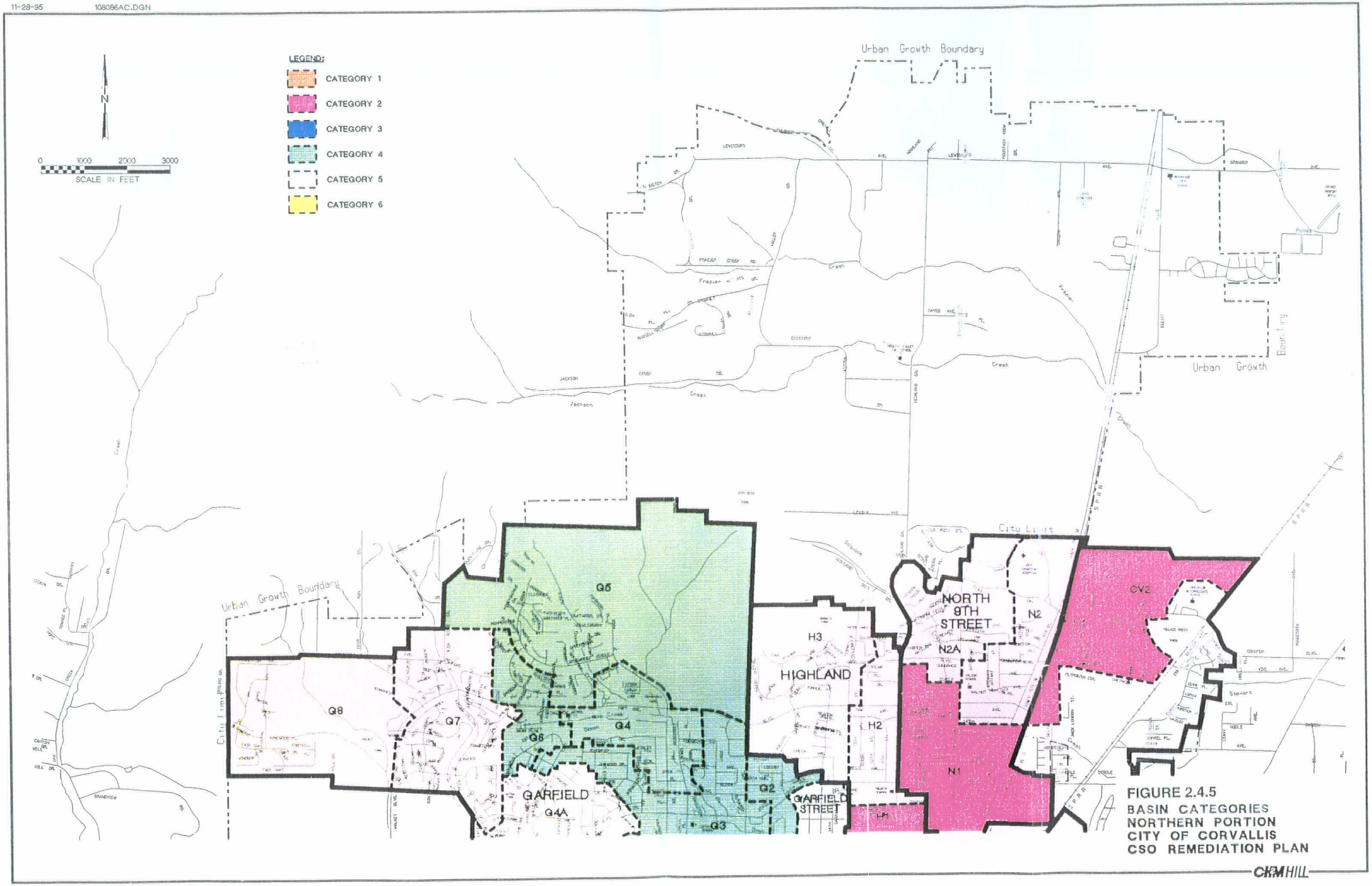

FIGURE 2.4.5
BASIN CATEGORIES
NORTHERN PORTION
CITY OF CORVALLIS
CSO REMEDIATION PLAN

CH2M HILL

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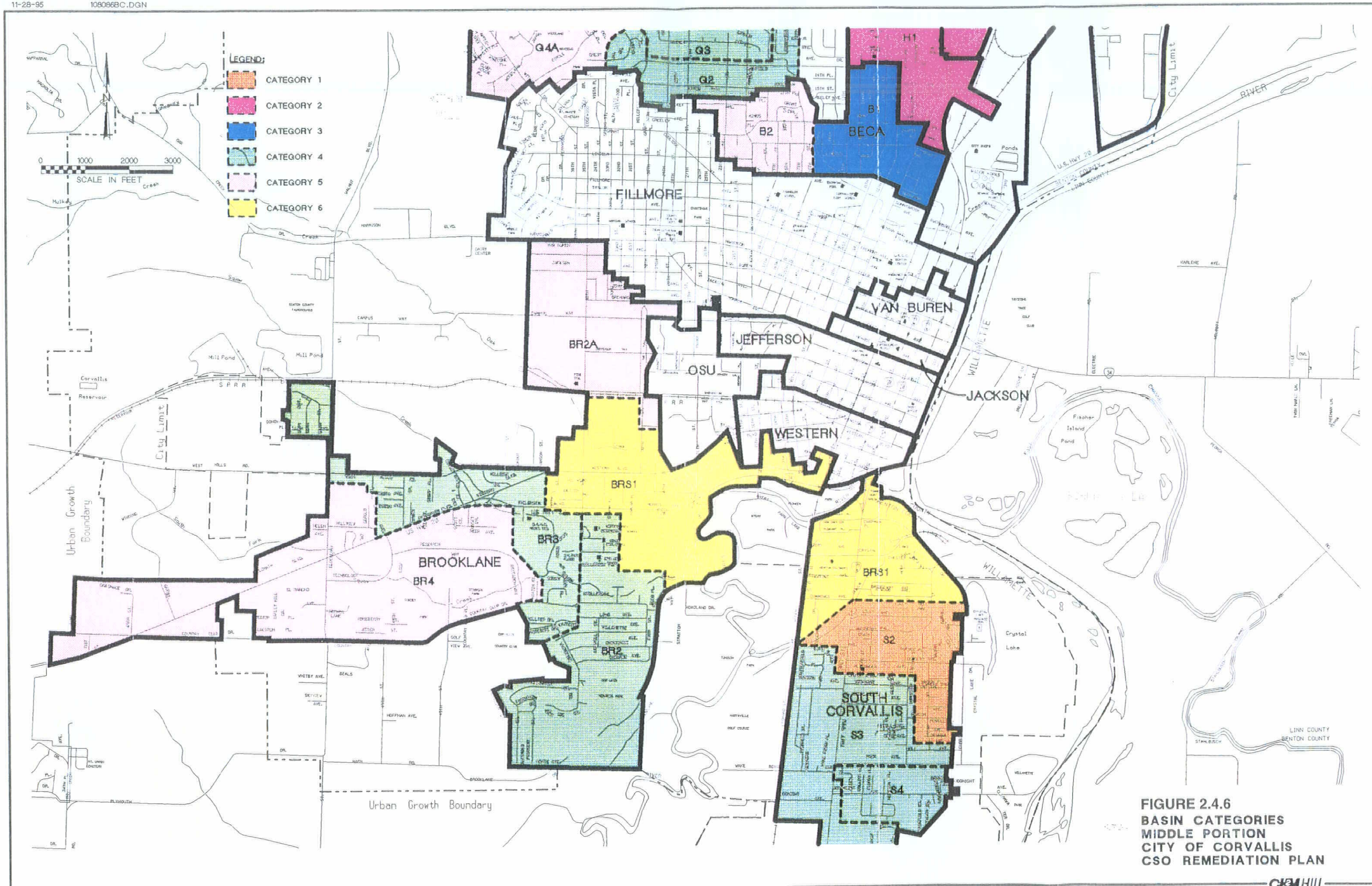

FIGURE 2.4.6
BASIN CATEGORIES
MIDDLE PORTION
CITY OF CORVALLIS
CSO REMEDIATION PLAN

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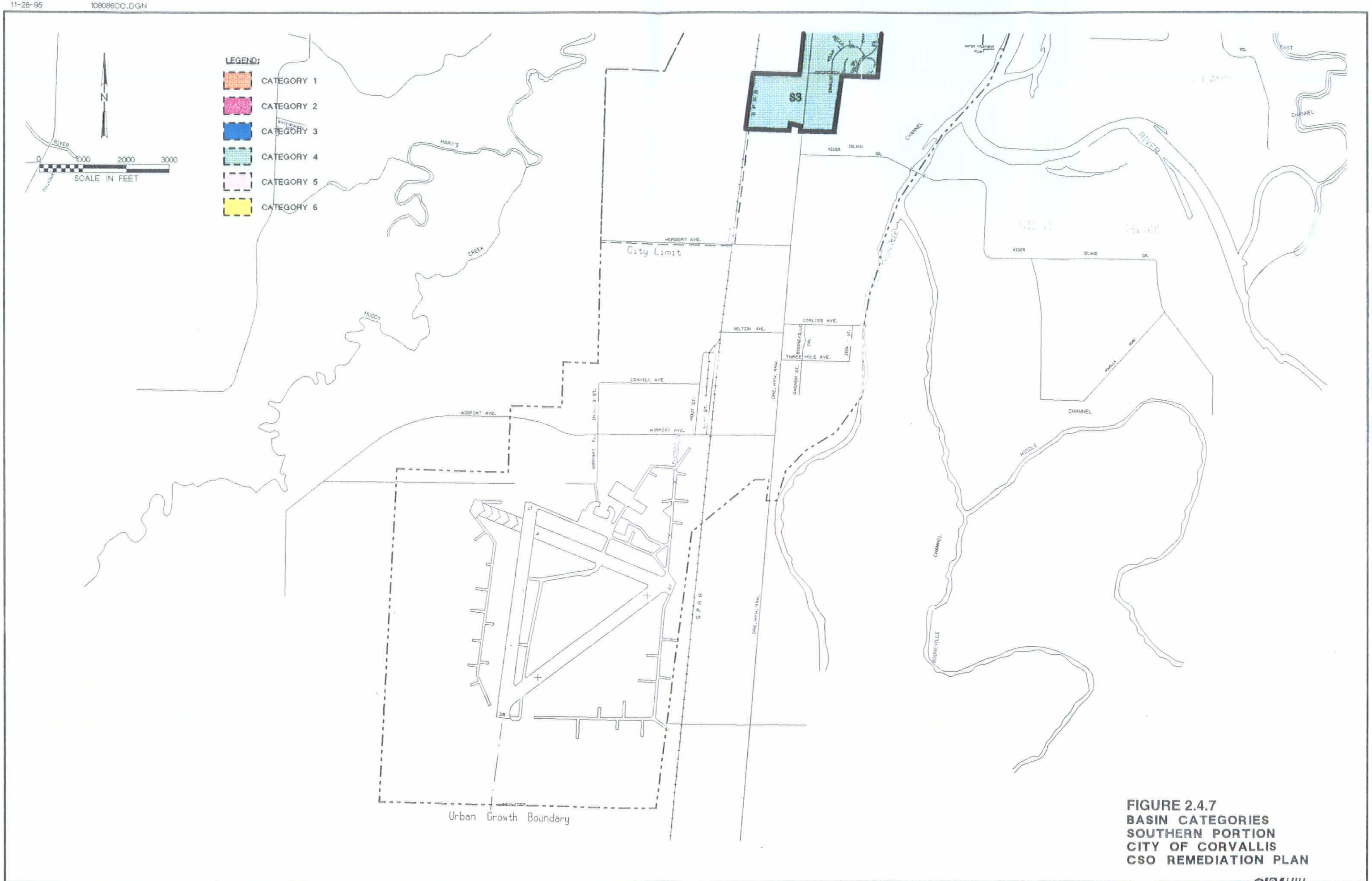

FIGURE 2.4.7
BASIN CATEGORIES
SOUTHERN PORTION
CITY OF CORVALLIS
CSO REMEDIATION PLAN

CH2M HILL

APPENDIX B

Corvallis CSO Hydraulic Model Calibration Technical Memorandum

TECHNICAL MEMORANDUM CH2M HILL

PREPARED FOR: City of Corvallis

PREPARED BY: David Wood/ATL
Gordon Nicholson/PDX

COPIES: Kim Erickson/CVO

DATE: September 13, 1995

SUBJECT: Corvallis CSO Hydraulic Model Calibration

PROJECT: 108086.A0.03

Purpose

This technical memorandum presents the City of Corvallis CSO existing condition model results and documents the final model calibration task.

Introduction

A SWMM model was developed for the Corvallis combined sewer system in July, 1994.

The model was preliminarily calibrated using a limited amount of available flow monitoring data. The preliminary model was used to support the development and evaluation of a series initial potential alternatives for CSO remediation. Based on this effort, the original list of alternatives was screened down to three through a public participation and citizens advisory process.

A comprehensive flow monitoring program was initiated during the months of January and February, 1995 to obtain data with which to refine the preliminary model. Data obtained from this program were used to calibrate and finalize the model. The final model was used to characterize existing CSO volume and to evaluate the effectiveness of the three remaining CSO remediation alternatives under the 5-year winter storm SFO design criteria.

CSO Characterization

The final model was run under two 5-year winter storm scenarios to establish peak CSO flow and total volume over the storm durations. Both storms produce 2.4-inches in 24 hours. The first, a synthetic storm, produces 2.4-inches with no rainfall preceding or subsequent to the 24-hour period. The second, an actual storm, is based on a 15-day period of rainfall. The actual storm has significant rainfall both before and after the 24-hour maximum rainfall interlude within the 15-day period. CSO volumes and peak flows for the two storms are presented in Table 1.

Table 1 Existing Condition CSO Volume and Peak Flow 5-Year, 24-Hour Winter Storm				
	Synthetic Storm		*Actual Storm*	
CSO	**Volume (mg)**	**Peak Flow (cfs)**	**Volume (mg)**	**Peak Flow (cfs)**
Fillmore	10.8	52.0	39.2	49.6
Polk	15.3	44.5	26.4	32.5
Van Buren	0	0	0.8	5.3
Jackson	0	0	6.8	8.6
Western	0.5	5.4	15.2	13.4
Total	26.6	-	88.4	-

Model Calibration

Flow monitoring was conducted during the months of January - February, 1995 to obtain data with which to calibrate and finalize the preliminary model. During these two months, three wet-weather events occurred suitable to be used for final model calibration: January 5-19, January 26 - February 1, and February 14-20.

Calibration involved adjusting specific hydrologic and hydraulic parameters such that model predicted flows, depths and velocities more closely matched observed data.

The calibration effort consisted of the following adjustments:

1. Subcatchment base flow contribution
2. Subcatchment percent of directly connected impervious area
3. Conduit roughness in the 1st Street Interceptor
4. Outfall river stage boundary conditions. River stage levels are available at the Taylor water treatment plant, and were extrapolated to the CSO outfall locations.

Calibration data were obtained at five locations in the combined sewer system: Fillmore, Tyler, Jefferson, and Western.

Fillmore Monitor

The Fillmore monitoring location recorded depth, velocity and flow in the conduit draining the Fillmore sewershed. The meter was located in the conduit just upstream of the Fillmore CSO regulating structure. Figures 1-3 present flow comparisons (modeled versus monitored) for the three calibration storms.

Tyler Monitor

The Tyler monitoring location recorded depth, velocity and flow in the 1st Street Interceptor upstream of the Polk CSO. Figures 4-6 present flow comparisons (modeled versus monitored) for the three calibration storms.

Jefferson Monitor

The Jefferson monitoring location recorded depth, velocity and flow in the conduit draining the Jefferson sewershed. The meter was located in the conduit just upstream of the junction with the 1st Street Interceptor. Figures 7-9 present flow comparisons (modeled versus monitored) for the three calibration storms.

Western Monitor

The Western monitoring location recorded depth, velocity and flow in the conduit draining the Western sewershed. The meter was located in the conduit just upstream of the Western CSO regulating structure. Figures 10-11 present flow comparisons (modeled versus monitored) for the three calibration storms. Monitoring data were not available for the February 14-20 storm period at the Western monitor.

SUMMARY

The City of Corvallis SWMM model has been successful calibrated using the results of the comprehensive 1995 flow monitoring program. The model accuracy has been significantly improved over the preliminary model. The model is of sufficient accuracy to support the preliminary engineering of the final recommended CSO remeditation alternative.

Figure 1

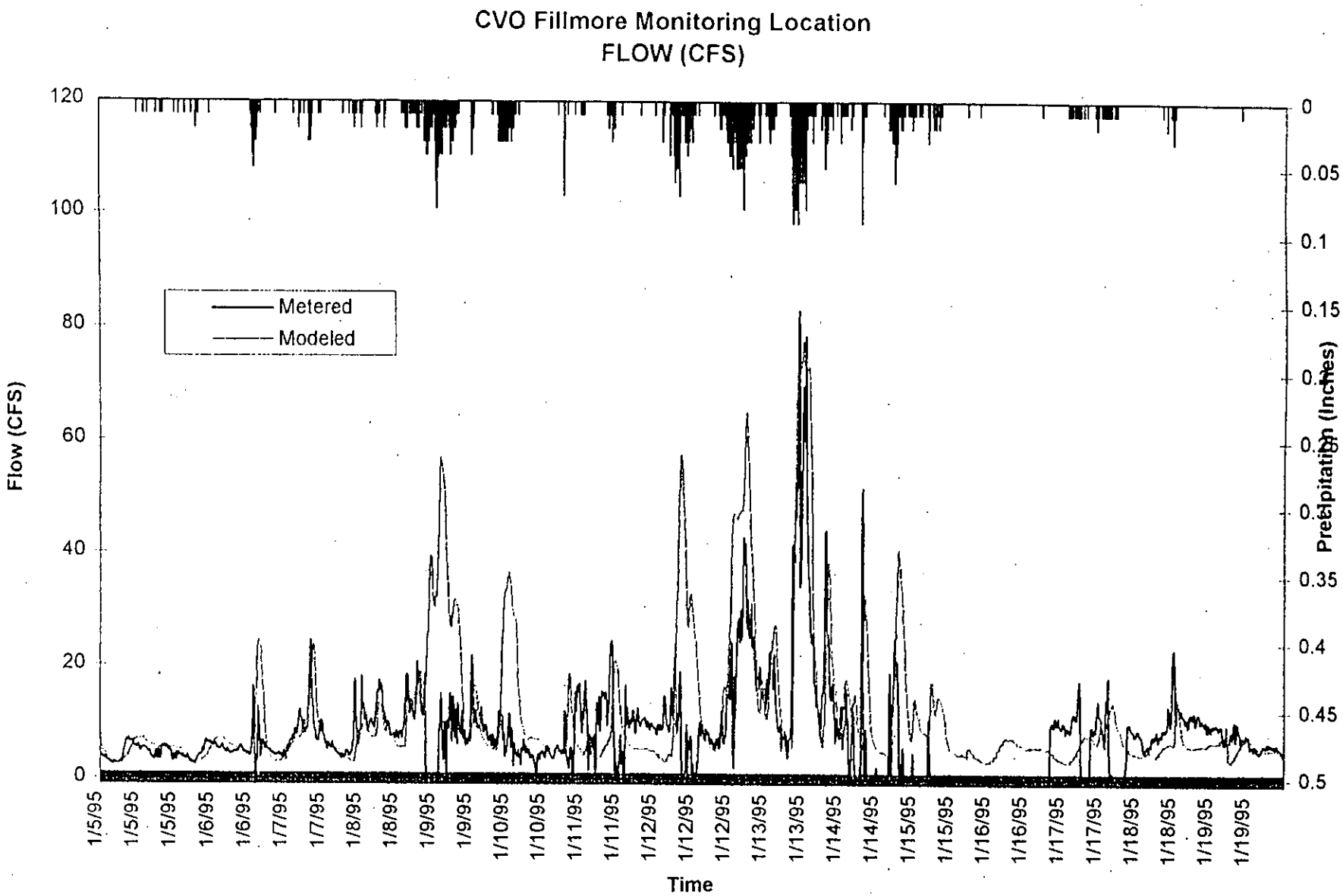

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Figure 2

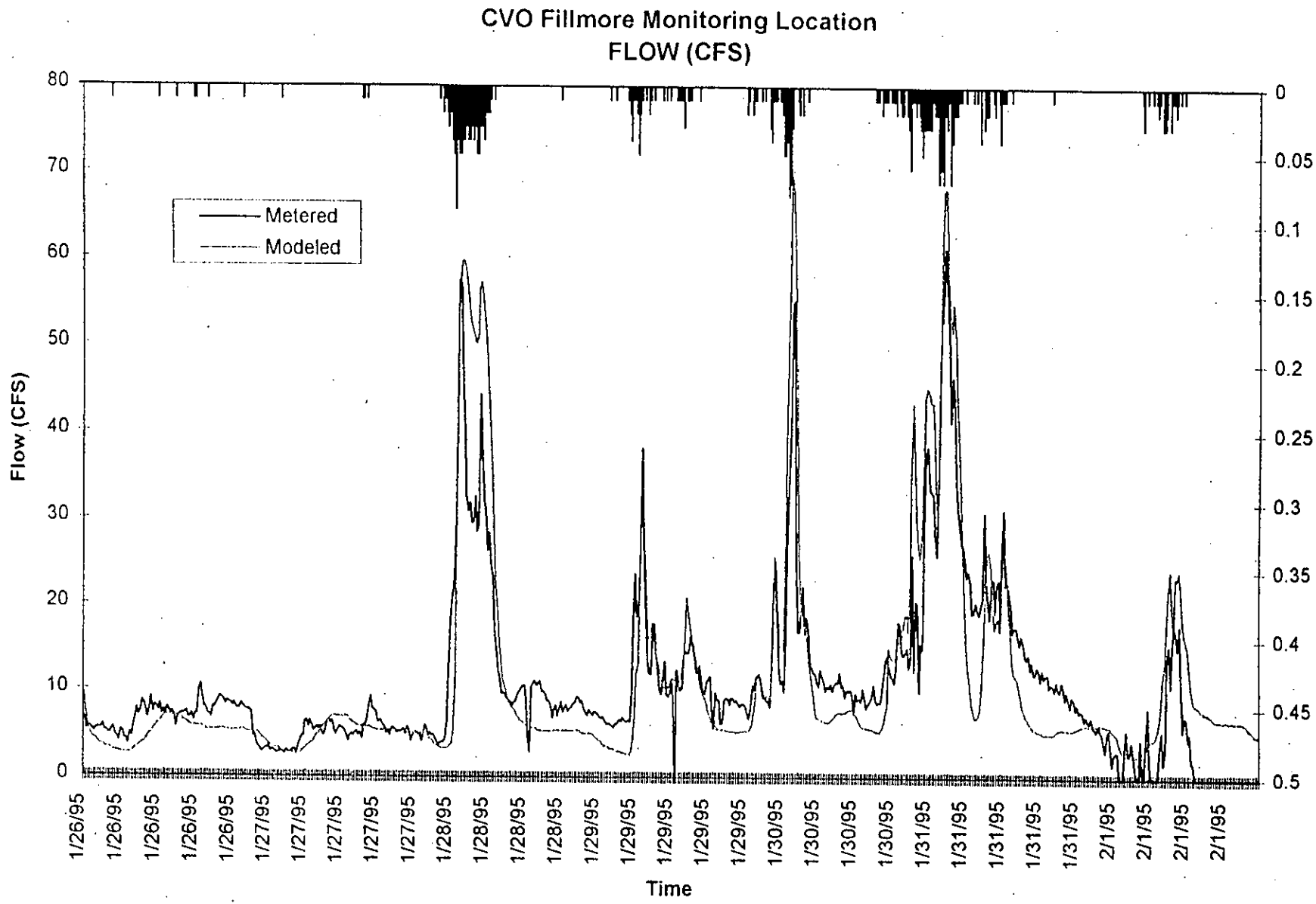

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Figure 3

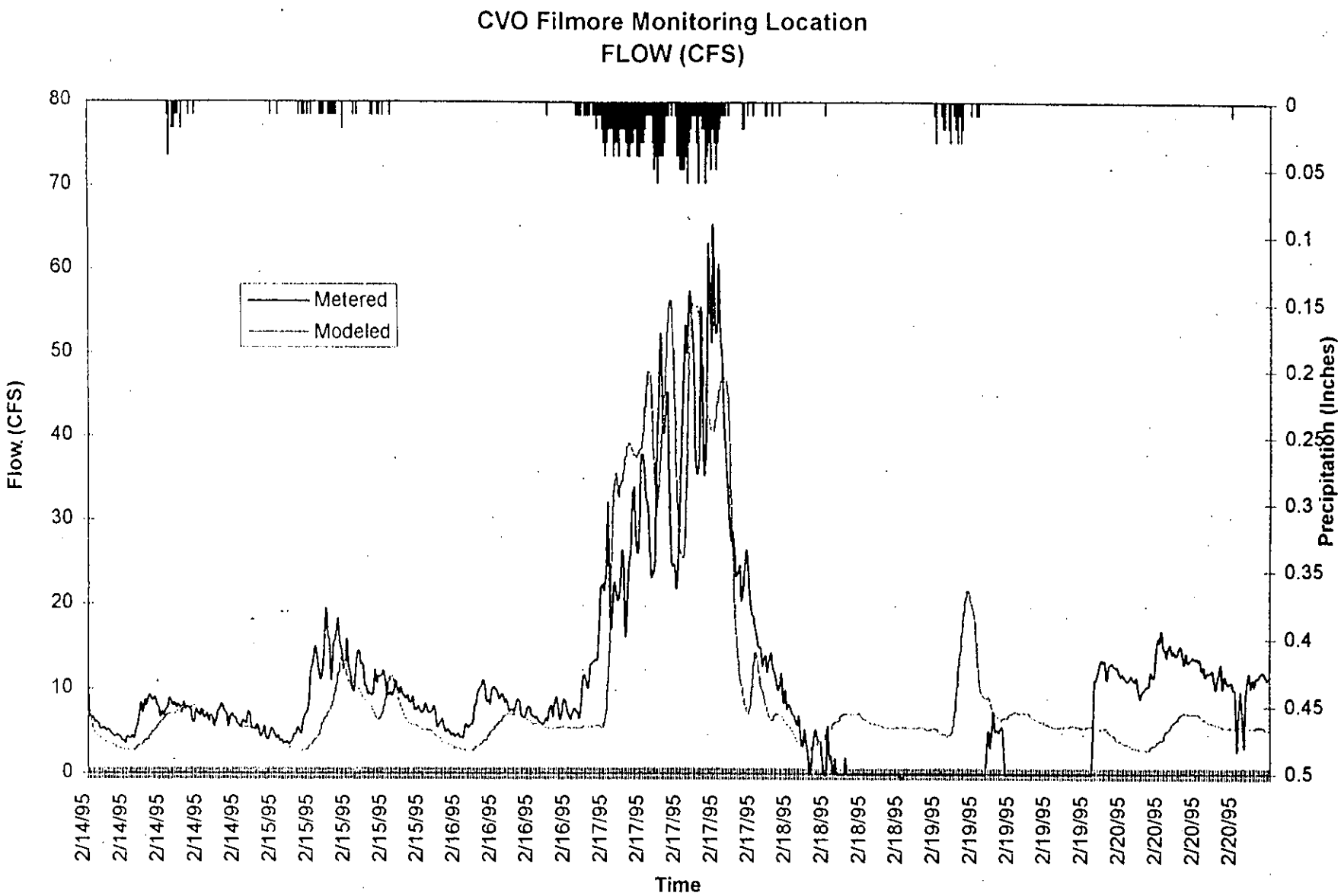

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Figure 4

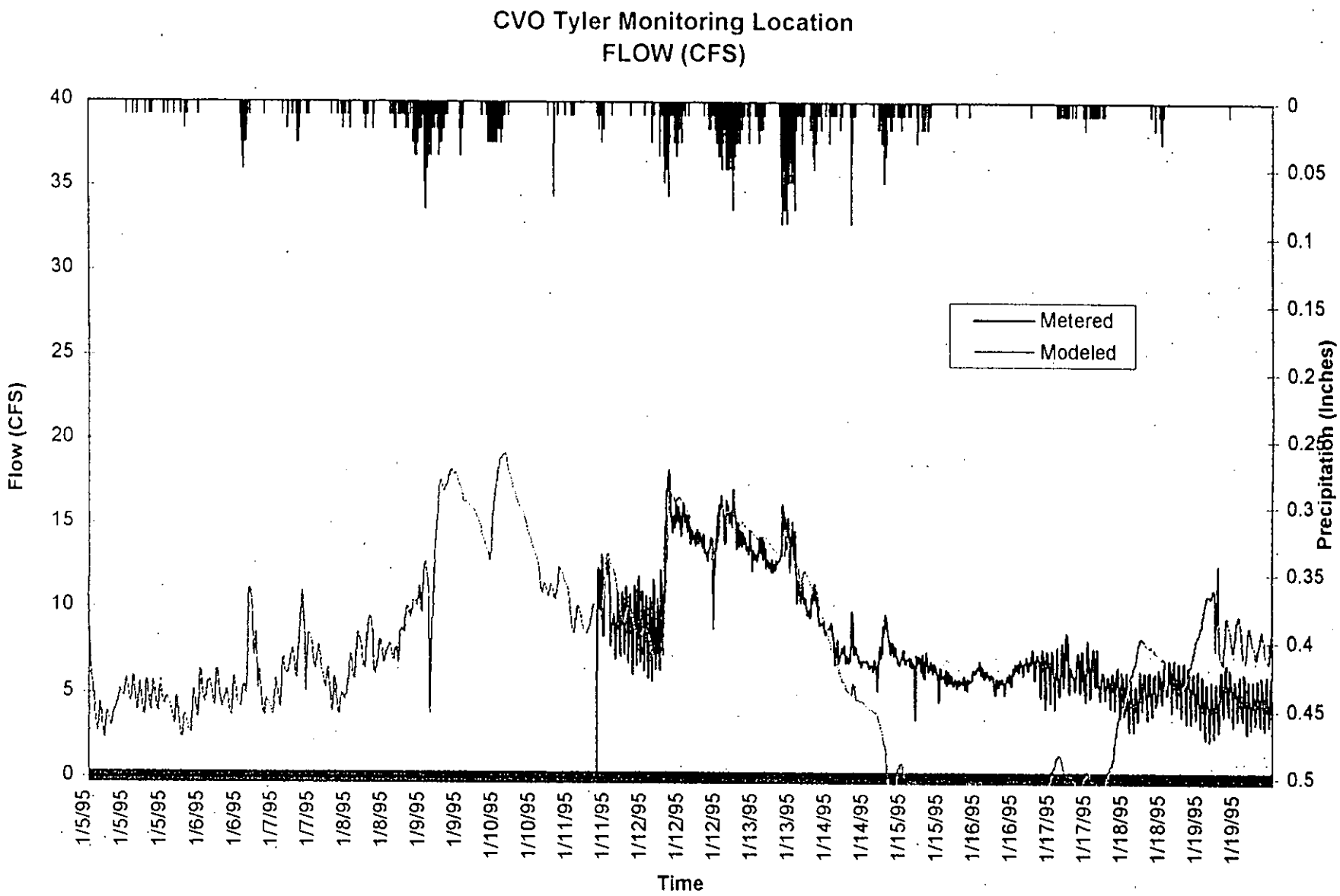

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Figure 5

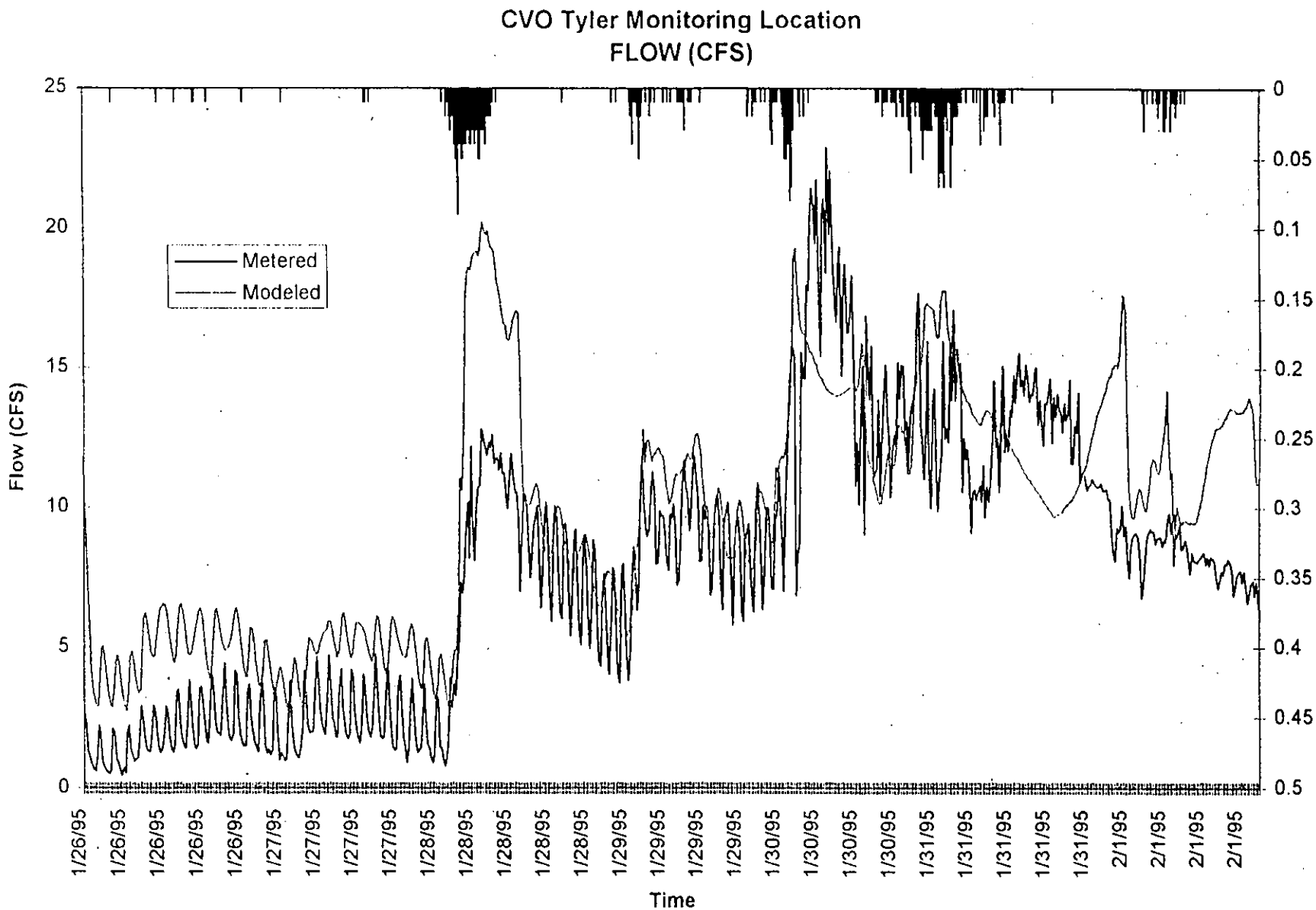

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Figure 6

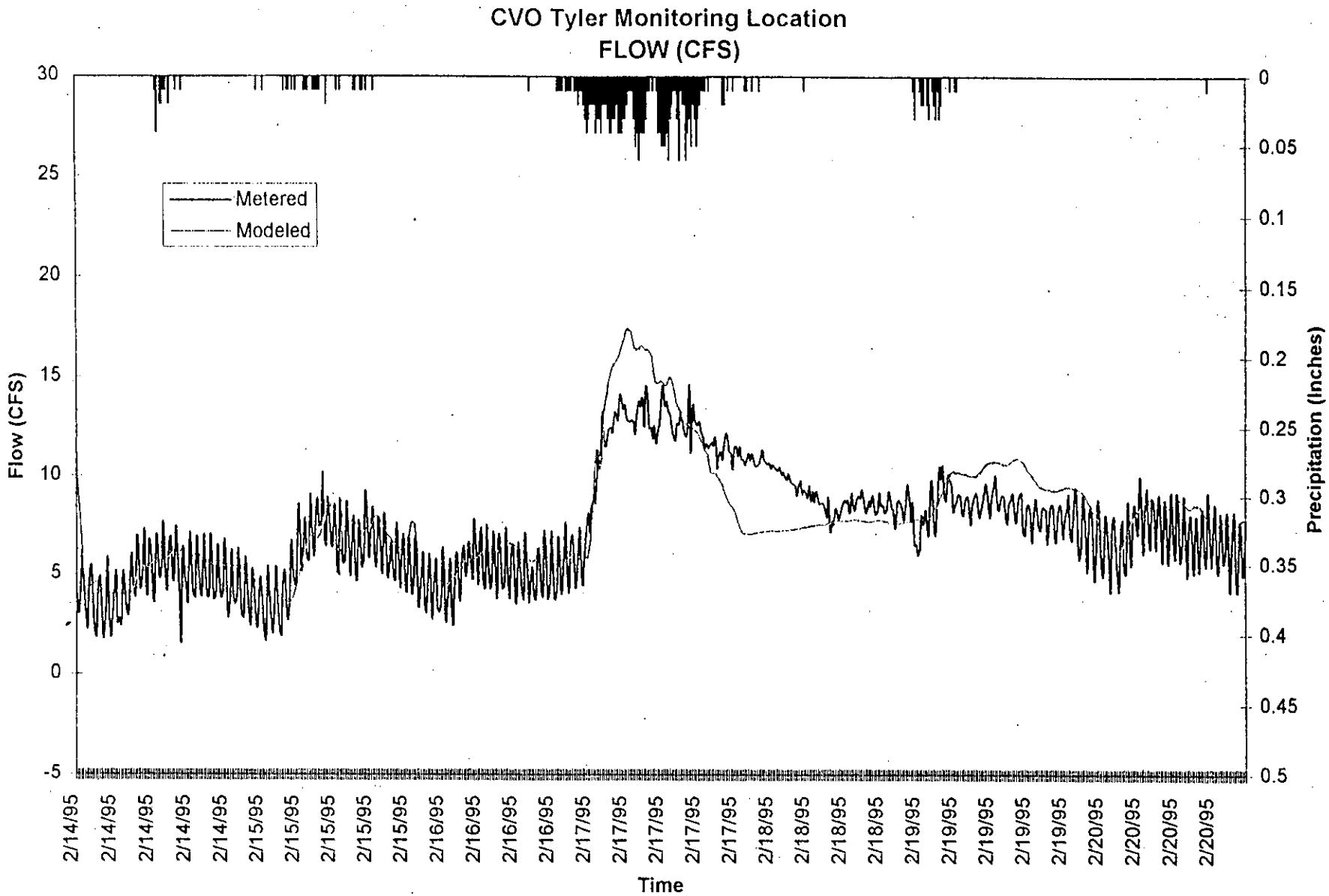

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Figure 7

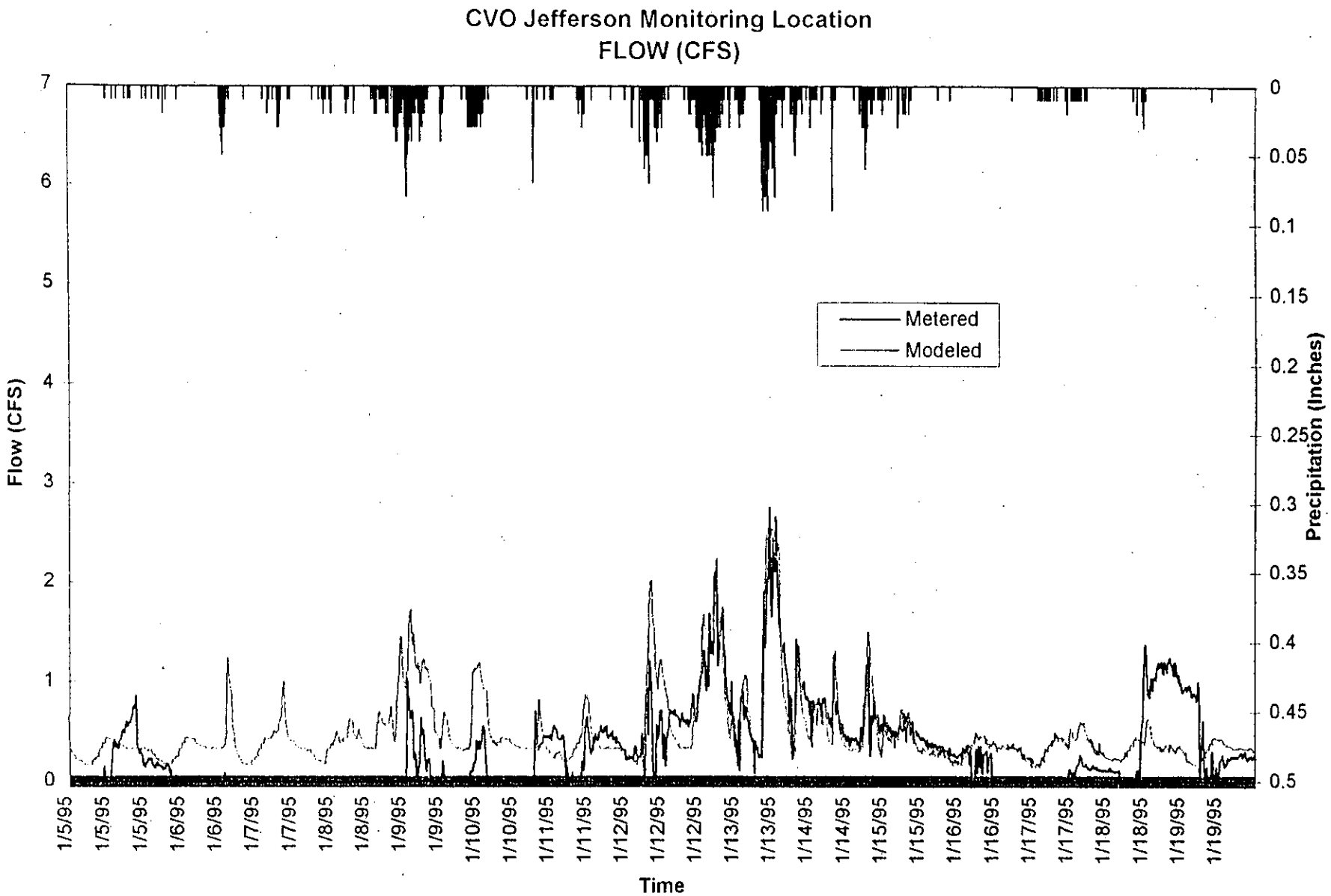

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Figure 8

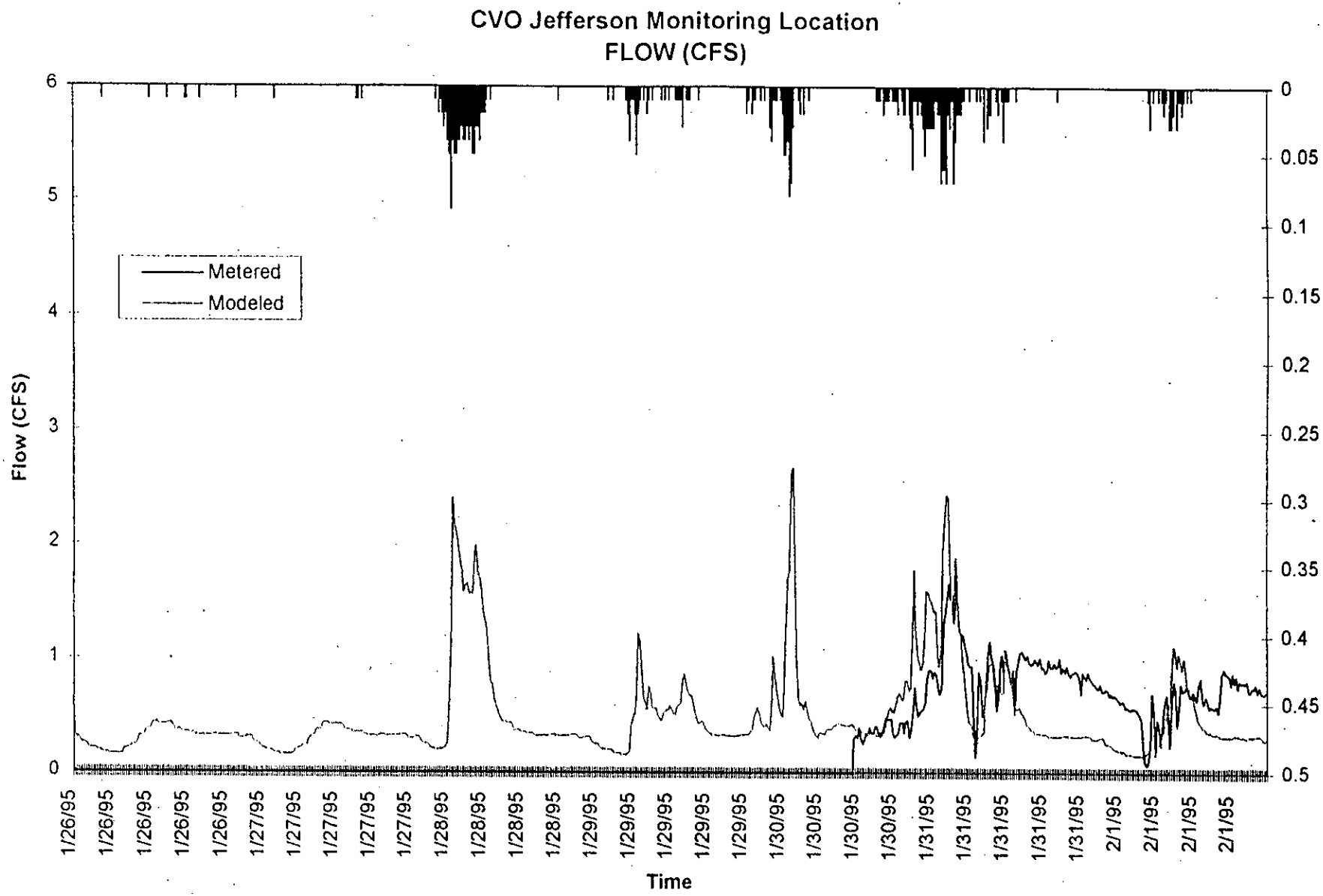

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Figure 9

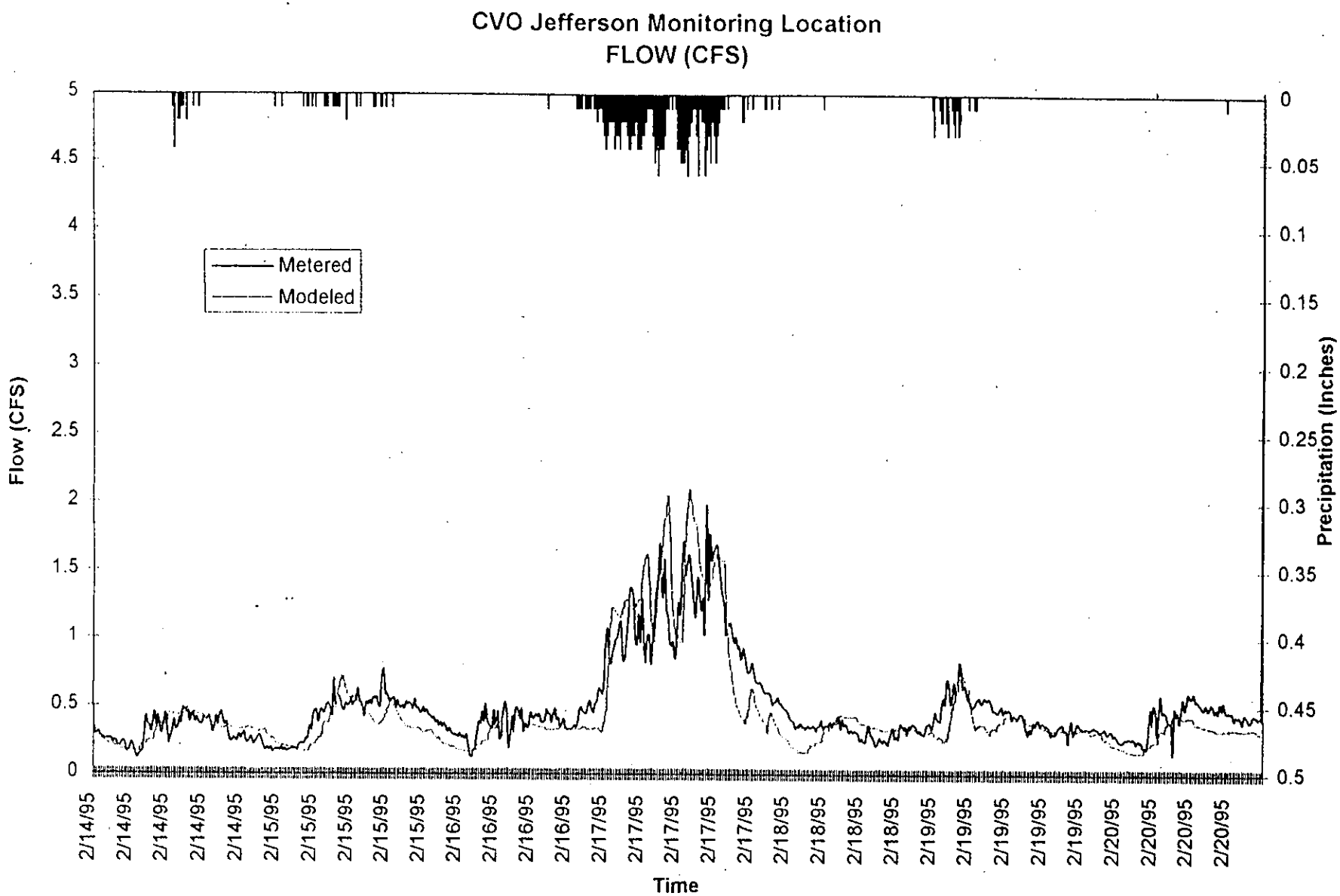

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Figure 10

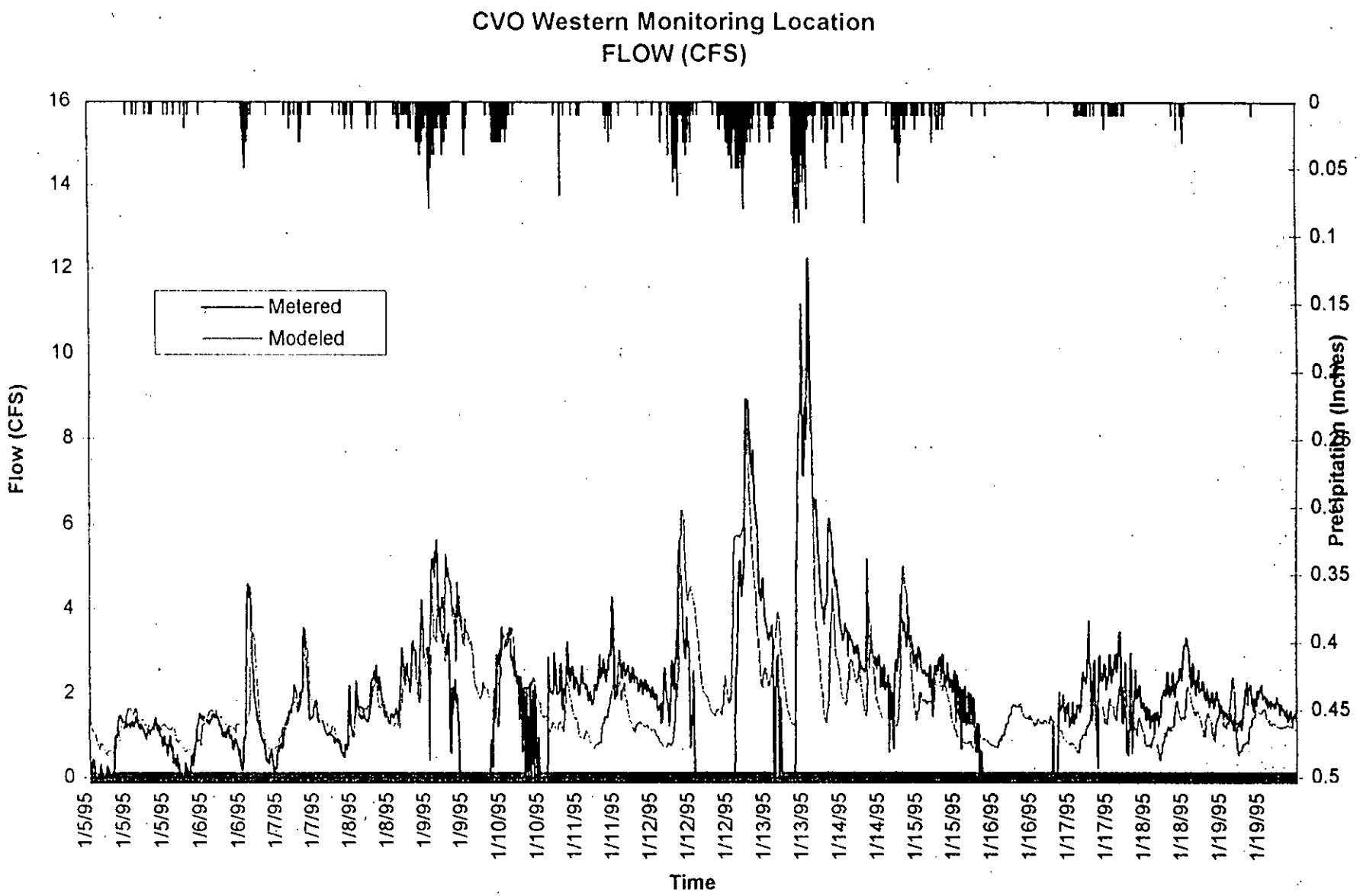
CVO Western Monitoring Location
FLOW (CFS)
Metered
Modeled
Flow (CFS)
Precipitation (Inches)
Time
16
14
12
10
8
6
4
2
0
0
0.05
0.1
0.15
0.35
0.4
0.45
0.5
1/5/95
1/5/95
1/5/95
1/6/95
1/6/95
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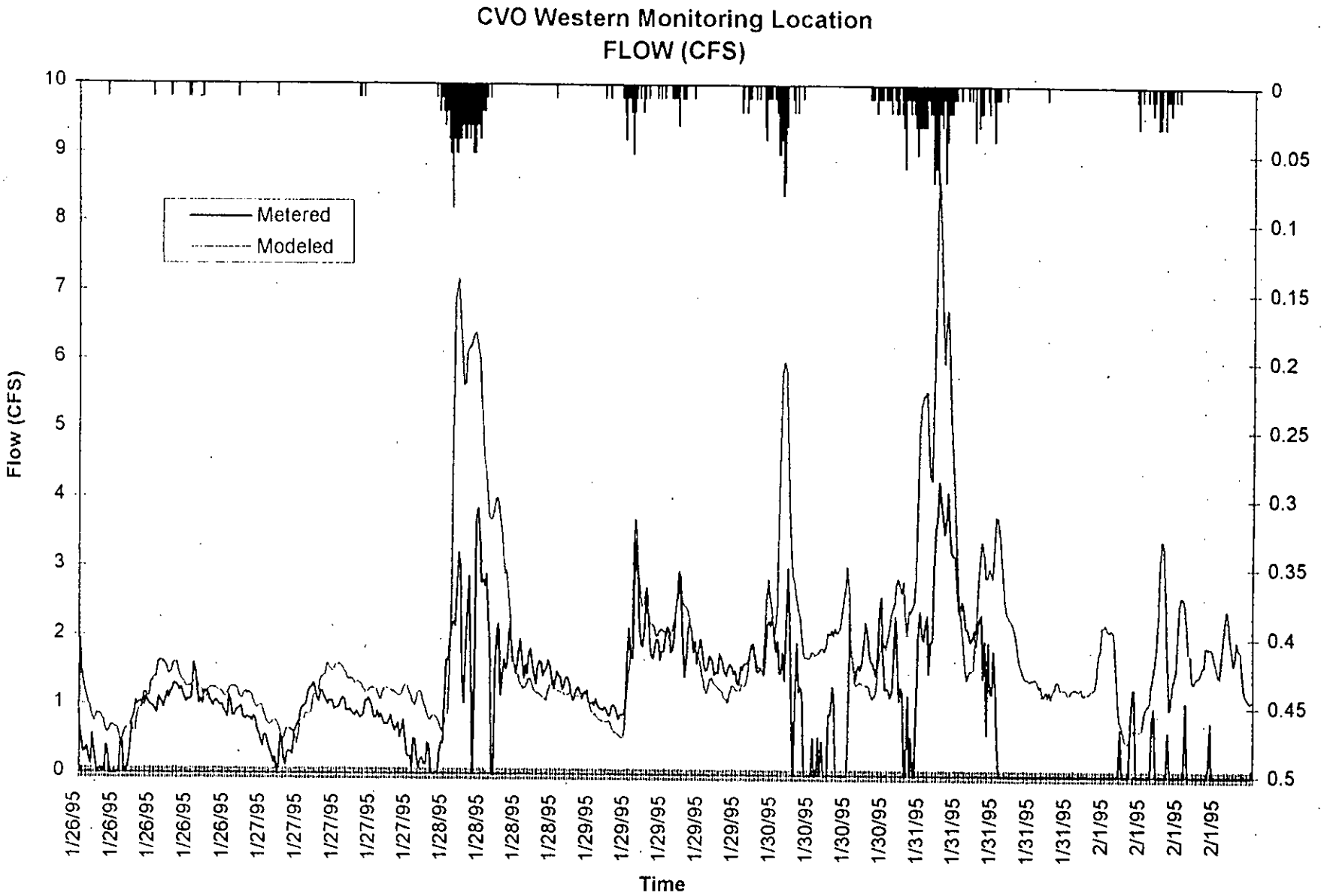
Figure 11



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APPENDIX C

Garfield Basin Smoke Testing Technical Memorandum

TECHNICAL MEMORANDUM

CH2M HILL

PREPARED FOR: Sue Stepan

FROM: Gordon Nicholson
Kim Erickson

PREPARED BY: Jim McWade

DATE: March 7, 1995

PROJECT: OPW36707.B0.2A

Garfield Basin
Smoke Testing

Introduction

During the CSO flow monitoring program conducted in 1993 it became evident that unexplained high peak flows were being generated during rain events in the Garfield Sanitary Sewer Basin. The Garfield Basin is not a combined sewer basin therefore high peak flows should not occur during rain fall.

CH2M HILL was requested to perform a smoke test program in the Garfield Basin to try to identify the sources of infiltration and inflow (I&I). This memorandum will present the results of the smoke testing program that was completed in October 1994.

Smoke testing is used to detect and identify sources of rainfall-dependent infiltration/inflow (RDI/I) in wastewater collection systems. These sources are generally located near the ground surface. RDI/I consists of two components: rainfall-dependent infiltration (RDI) and stormwater inflow (SWI).

RDI is identified as infiltration that slowly percolates into the collection system. Typical sources of RDI include defective service laterals, sewer mains, and manholes. SWI normally reaches a peak shortly after rainfall intensity is greatest and falls off rapidly when rain subsides. Typically sources of SWI include downspouts, cross-connected area drains, and cross-connected catch basins.

Smoke Testing Procedures

During October 1994, CH2M HILL and City of Corvallis field staff conducted a smoke testing program in three subbasins of the Garfield basin. The areas tested total nearly 625 acres containing an approximate 60,000 lineal feet of pipeline.

A public awareness program was conducted both before and during smoke testing to notify businesses and residents of the upcoming program. Printed notices advising residents of the smoke testing program were hand delivered to each home/building connected with the sewer tested. The notices, which contained a "Hotline" number for information, were delivered prior to the actual testing. The Corvallis Fire Department was notified daily during the testing.

A portable gasoline-engine blower located over a manhole forces a nontoxic smoke through the sewerline, as shown in Figure 1. Smoke travels through the isolated sewer sections and escapes through sewer defects that could potentially allow rainwater to enter. Each smoke source is recorded on log sheets and photographed for permanent documentation, see Appendix A for basin maps and logs.

The smoke escapes from the sewer through leaks, breaks, and other deficiencies. Typical defects identified with this source detection technique are shown in Figure 2 and listed below.

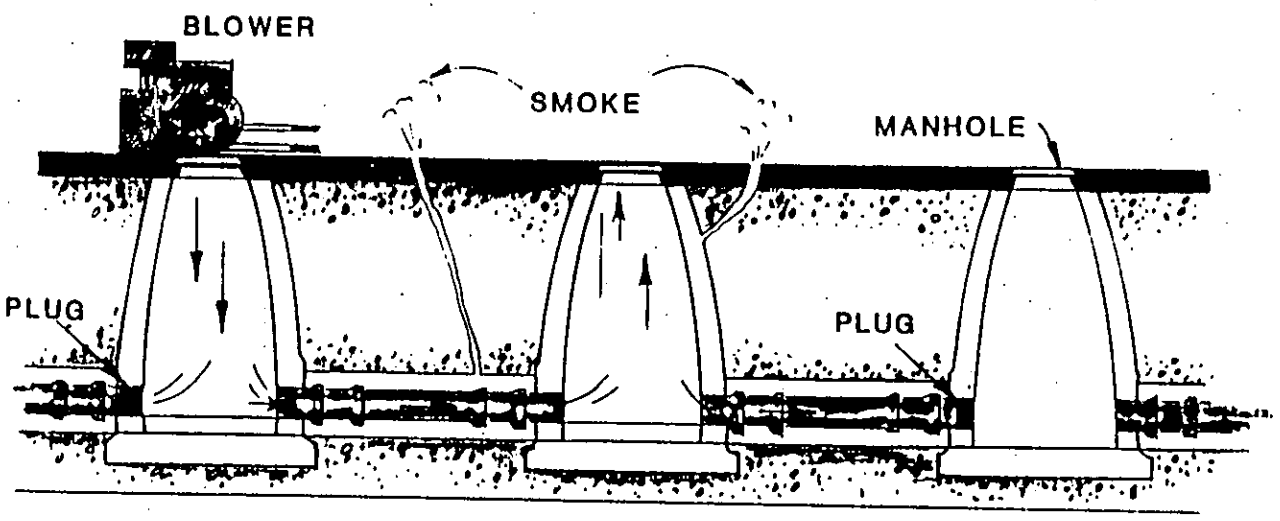
- Downspouts connected to the sanitary system
- Area, yard, and driveway drains connected to the sanitary system
- Cross-connections between sanitary sewers and storm drains
- Structurally damaged sewers, manholes, and service laterals
- Perforated manhole/lamphole covers
- Open cleanouts

Although smoke testing indicates possible I/I sources, interpretation of smoke testing results must take into consideration the following.

- The method is only a source detection technique for inflow and cannot be used to accurately quantify actual I/I flow rates.

- Although smoke may escape from a sewer defect, it does not mean that I/I will enter; it only means that a possible source has been located. Further examination or testing such as dyed water testing and/or television inspection may be necessary to determine if entry is possible and, if so, the approximate rate of entry.

- While a smoke emission is considered evidence of a defect, the converse is not true. If smoke is not observed escaping, it cannot be assumed that the manhole-to-manhole segment tested does not have any defects. Certain conditions may prevent smoke from escaping during the testing period. For example,

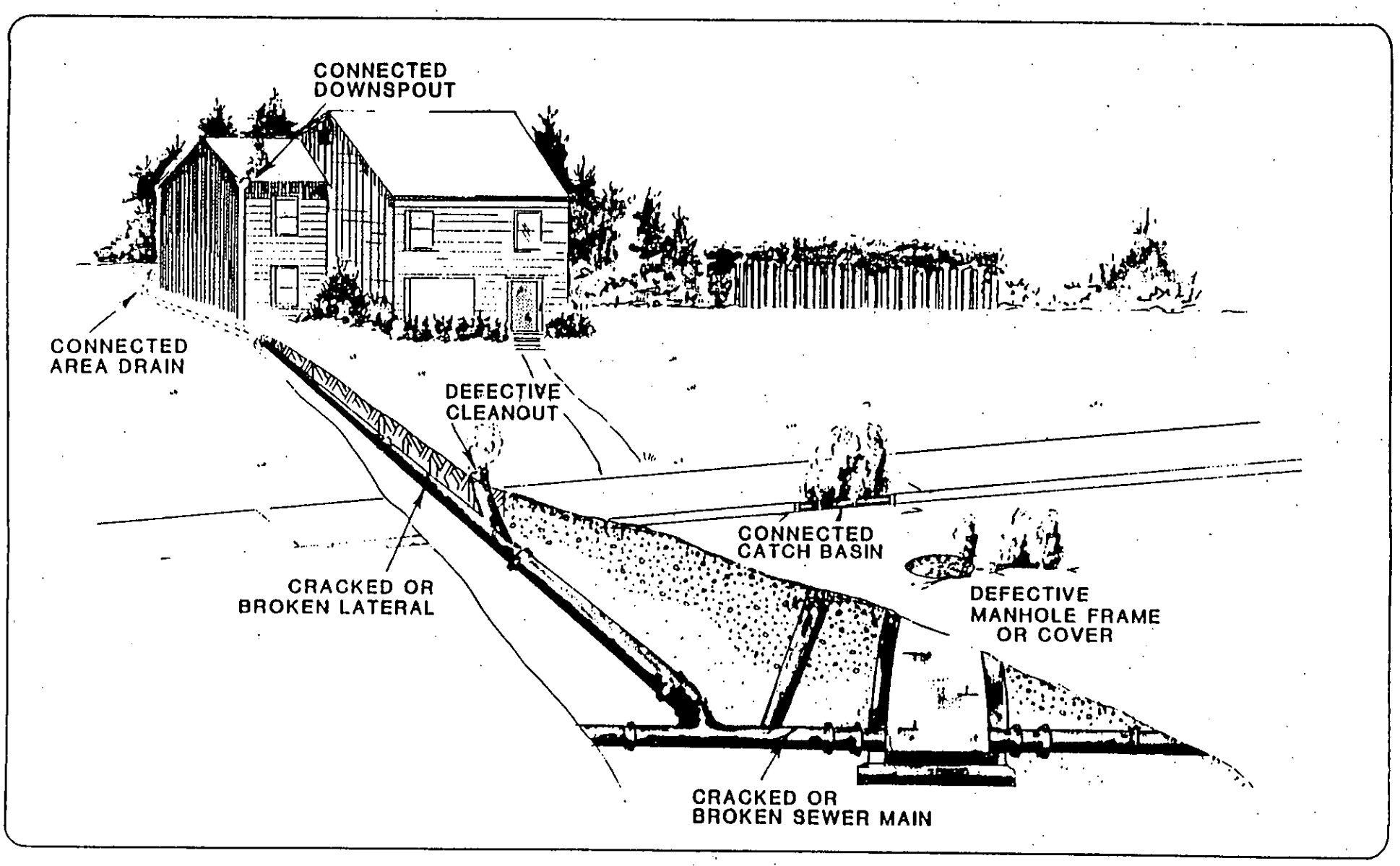


SMOKE TESTING PROCEDURES

FIGURE 1

SMOKE TESTING PROGRAM

CH2M HILL



SMOKE TESTING DEFECTS

FIGURE 2

SMOKE TESTING PROGRAM

severe roots, debris, or sags in the line may prevent smoke from traveling through the line and escaping through cracks or defective joints.

- It cannot be assumed that all of the defects in any section tested have been identified. Physical conditions may vary within portions of any manhole-to-manhole segment: some conditions allowing smoke to escape, some preventing it.

Smoke Testing Results

Detailed smoke test findings are presented in Table 1. Each defect is identified by its street address. The identified defects included manholes, downspouts, service laterals, sewer mains, cleanouts, and catch basins.

Rehabilitation costs are presented in Table 2.

Rehabilitation Procedures

Manholes

During manhole inspection and smoke testing, several manholes were identified as recessed manholes. These manholes can be subject to large quantities of inflow, these manholes should be raised to grade. A number of manholes exhibited infiltration or displayed signs of infiltration in the past. The locations of the infiltration source were recorded. Several manhole frames were noted as not sealed at the riser. Additionally, a large number of the manhole covers have up to 13 holes. These extra holes will allow up to 5 gallons of inflow per cover per storm. Thus, surface water could enter the cracks in the street and flow through the open frame joint. The manholes exhibiting infiltration and inflow and insufficient frame seals are listed in Table 1. Manhole repairs consist of frame sealing, plugging unnecessary lid holes, cone sealing, riser sealing, and base sealing. The cost per manhole to seal the cone, riser, or base varies with manhole depth.

Sewer Mains

The rehabilitation procedure for sewer mains involves a spot repair at the smoke source. Additional television inspection should be performed prior to excavation to identify the exact defect location and the extent of damage. The estimated cost of each spot repair is $2,500 for the first 20 lineal feet, plus $100/lineal foot for each additional foot of repair needed.

Service Laterals

Rehabilitation of service laterals located on private property may require spot repairs or entire replacement depending on the extent and location of the damage. The City's costs, estimated at $100 per defect for service lateral repairs, consist of repair notifications addressed to property owners and follow-up repair verification. The cost to the property

APPENDIX D

Collection System Details and Cost Estimates

Population Distribution for System Modeling

Basin	Growth Population Increment for 60,000 [a]	Total Additional Developed Acres at 60,000	Growth Population Increment for 80,000[a]	Total Additional Developed Acres at 80,000	Growth Population Increment for 120,000 [a]	Total Additional Developed Acres at 120,000	Growth Population Increment for Maximum Buildout [a]	Additional Developed Acres within City Limits @ Maximum Buildout	Additional Developed Acres outside City Limits in UGB @ Maximum Buildout	Total Additional Developed Acres at Maximum Buildout
Lewisburg	202	15	685	52	1,677	127	3,124	-	236	236
Frazier Creek	701	66	2,382	225	5,831	550	10,865	-	1,025	1,025
Crescent Valley	-		-		-		-	-	-	-
Northeast										
CV2	59	3	202	11	494	28				52
End of CV Interceptor	829	47	2,817	160	6,896	392				731
Subtotal	889	51	3,018	172	7,390	421	13,770	405	379	784
North Ninth										
N1	104	12	353	40	864	99				184
N2	144	16	490	56	1,199	137				256
N2A	30	3	101	12	247	28				53
Subtotal	278	32	944	108	2,310	264	4,305	269	224	492
Highland										
H3	30	6	103	19	253	46				86
H2	-		-		-					-
Subtotal	30	6	103	19	253	46	471	19	55	86
Garfield										
G2	45	6	151	20	370	49				91
G3	59	8	202	27	494	65				122
G4	15	2	50	7	123	16				30
G4A	74	10	251	33	614	81				152
G5	297	39	1,008	133	2,469	327				609
G6	15	2	50	7	123	16				30
G7	15	2	50	7	123	16				30
G8	89	12	302	40	741	98				183
Subtotal	608	81	2,066	273	5,058	670	9,425	967	280	1,248
Beca										
B1	19	3	64	12	156	29				54
B2	19	3	64	12	156	29				54
Subtotal	38	7	128	24	312	58	582	107	-	107
Fillmore	651	56	2,213	190	5,418	466	10,096	380	488	868
Future East	66	13	222	45	545	111			206	206
Future South	82	18	279	62	682	151			282	282

Population Distribution for System Modeling

Basin	Growth Population Increment for 60,000 [a]	Total Additional Developed Acres at 60,000	Growth Population Increment for 80,000[a]	Total Additional Developed Acres at 80,000	Growth Population Increment for 120,000 [a]	Total Additional Developed Acres at 120,000	Growth Population Increment for Maximum Buildout [a]	Additional Developed Acres within City Limits @ Maximum Buildout	Additional Developed Acres outside City Limits in UGB @ Maximum Buildout	Total Additional Developed Acres at Maximum Buildout
Lower Oak Creek-BR2A	110	7	372	24	911	59	1,697	61	50	111
Western Blvd.	-		-		-		-	-	-	-
Van Buren	-		-		-		-	-	-	-
Jackson Creek	531	55	1,805	186	4,419	455	8,235	8	840	848
Jefferson	-		-		-		-	-	-	-
Squaw Creek-BR3	754	56	2,561	190	6,269	464	11,681	224	640	865
Country Club-BR4	775	64	2,632	219	6,444	536	12,007	391	607	998
Brooklane										
BR2	45	5	153	16	374	40				74
BR2A-See L. Oak Cr.	-		-		-					-
BR3-See Squaw Cr.	-		-		-					-
BR4-See Country Club	-		-		-					-
Subtotal	45	5	153	16	374	40	696	67	7	74
South Corvallis										
S2	100	5	338	19	828	46				85
S3	365	20	1,239	68	3,033	167				311
S4	51	3	174	10	426	23				44
New line west of existing	2,092	115	7,105	391	17,396	956				1,782
Subtotal	2,607	143	8,857	487	21,684	1,192	40,404	778	1,443	2,221
Marys River	42	1	143	4	350	9	653	**18**	-	**18**
SUBTOTALS	**8,261**	**644**	**28,060**	**2,188**	**68,701**	**5,356**	**128,011**			**9,980**
1995 Population within City Limits	47,485		47,485		47,485		47,485			
1995 Unserved Population outside City Limits within UGB	4,513		4,513		4,513		4,513			
TOTALS	**60,259**		**80,058**		**120,699**		**180,009**			

[a]Incremental increase to total 1995 population within UGB. For example, the forecasted incremental increase in population for probable buildout (120,000) is 68,701. (68,701+47,485+4,513 = approximately 120,000. Dwelling units for future populations were estimated by dividing population by 2.14.

Service Areas and Basins	Existing (1995) Population within City Limits	Growth Population Increment for 80,000 (final distribution)	Growth Population Increment for buildout (final distribution)	Assumed 120k Population Distribution for SDC project ID Purposes
Crescent Valley Service Area				
Lewisburg		685	3,124	685
Frazier Creek		2,382	10,865	2382
Crescent Valley		-	-	0
Northeast		3,018	13,770	3018
Jackson Creek		1,805	8,235	1,805
Subtotal	**2,009**	**7,890**	**35,994**	**7,890**
Garfield Service Area				
North Ninth		944	4,305	4,305
Highland		103	471	471
Garfield		2,066	9,425	9,425
Subtotal	**16,776**	**3,113**	**14,201**	**14,201**
Beca	**1,685**	**128**	**582**	**582**
Brooklane Service Area				
Lower Oak Creek-BR2A		372	1,697	1,697
Squaw Creek-BR3		2,561	11,681	11,681
Country Club-BR4		2,632	12,007	12,007
Brooklane		153	696	696
Marys River		143	653	653
Subtotal	**8,270**	**5,860**	**26,735**	**26,735**
Combined Service Area				
Western Blvd.		-	-	0
Van Buren		-	-	0
Jefferson		-	-	0
Fillmore		1,712	7,810	7,810
Future East		222	1,015	1,015
Future South		279	1,271	1,271
Subtotal	**14,360**	**2,213**	**10,096**	**10,096**
South Corvallis	**4,385**	**8,857**	**40,404**	**8857**
Total Incremental Population Growth	**47,485**	**28,060**	**128,011**	**68,359**
1995 Population		**47,485**	**47,485**	**47,485**
1995 Unserved Population		**4,513**	**4,513**	**4,513**
Total Population		**80,058**	**180,009**	**120,357**

Project Identification and Costs to Serve 120,000 Population

(buildout improvements used in shaded basins, non-shaded basins use 80,000 improvements)

Basin	Projects to Serve 120k Assuming Shaded basins reach maximum buildout	Project Cost
Crescent Valley Service Area		
Lewisburg	No projects	$ -
Frazier Creek	No projects	$ -
Crescent Valley	No projects	$ -
Northeast	Crescent Valley Trunk	$ 2,300,000
Jackson Creek	No projects	$ -
Subtotal		**$ 2,300,000**
Garfield Service Area		
North Ninth	North Ninth Street Diversion	$ 1,570,000
	North Ninth Street Buildout Sewers	$ 1,500,000
Highland	No projects	$ -
Garfield	Garfield Relief Sewers	$ 530,000
	Garfield Buildout Sewers	$ 1,312,500
Subtotal		**$ 4,912,500**
Beca	No projects	$ -
Brooklane Service Area		
Lower Oak Creek-BR2A	No projects	$ -
Squaw Creek-BR3	Squaw Creek Buildout Sewers	$ 2,906,250
Country Club-BR4	Country Club Trunk	$ 1,200,000
	Country Club Buildout Sewers	$ 4,187,500
Brooklane	No projects	$ -
Marys River	Marys River Interceptor	$ 4,480,000
	Brooklane Pump Station	$ 3,810,000
Subtotal		**$ 16,583,750**
Combined Service Area		
Western Blvd.	No projects	$ -
Van Buren	No projects	$ -
Jefferson	No projects	$ -
Fillmore Combined	No projects	$ -
Future East	Fillmore East Pipeline Extension	$ 468,750
Future South	Fillmore South Pipeline Extension	$ 1,093,750
Subtotal		**$ 1,562,500**
South Corvallis	South Corvallis Parallel Interceptor & Diversion	$ 12,410,000
	Marys River Pump Station	$ 4,170,000
Subtotal		**$ 16,580,000**
Total All Basins		**$ 41,938,750**

Project Costs for Buildout Improvements within Basins (in Addition to 80k/120k Projects)

Subbasin	Upstream MH	Pipe Dia (inches)	Pipe Length (feet)	Cost[1]
Garfield	4674030	24	153	$ 50,000
(in addition to Garfield Relief Sewers)	4674032	30	430	$ 162,500
	4674034	30	896	$ 337,500
	4677002	30	214	$ 75,000
	4677026	30	253	$ 87,500
	4677027	30	219	$ 75,000
	4677084	30	278	$ 100,000
	4868049	10	186	$ 25,000
	4868076	12	461	$ 62,500
	4868051	12	147	$ 25,000
	4868050	12	326	$ 50,000
Subtotal			**3,562**	**$ 1,050,000**
Total with 25% Engineering, Legal, Administration				**$ 1,312,500**
Highland	No Projects			
Subtotal				**$ -**
North Ninth	4880055	30	490	$ 187,500
(in addition to North Ninth St. Diversion project)	4880039	30	528	$ 200,000
	4883006	30	393	$ 150,000
	4680028	30	435	$ 162,500
	4880062	30	404	$ 150,000
	4680056	30	560	$ 212,500
	4680057	30	387	$ 137,500
Subtotal			**3,197**	**$ 1,200,000**
Total with 25% Engineering, Legal, Administration				**$ 1,500,000**
Northeast	5283048	48	270	$ 162,500
	5286038	48	266	$ 162,500
	5286037	48	167	$ 100,000
	5286035	48	350	$ 212,500
	5286036	48	96	$ 62,500
	5086017	48	422	$ 250,000
	5086010	48	325	$ 187,500
	5086016	48	435	$ 262,500
	5086011	48	198	$ 112,500
	5086015	48	475	$ 287,500
	5086012	48	323	$ 187,500
	5086014	48	110	$ 62,500
	5086013	48	135	$ 75,000
	4883018	48	179	$ 112,500
	4883016	48	349	$ 212,500
	4883017	48	301	$ 175,000
	4883019	48	292	$ 175,000
	4683001	48	459	$ 275,000
	4483005	48	241	$ 137,500
	4883015	48	362	$ 212,500
	4683002	48	418	$ 250,000
	4683005	48	462	$ 275,000
	4883022	48	323	$ 187,500
	4683004	48	385	$ 225,000
	4683006	48	384	$ 225,000
	4683003	48	460	$ 275,000
	4886001	48	440	$ 262,500
	4483009	48	360	$ 212,500
	5086018	48	432	$ 262,500
Subtotal			**8625**	**$ 6,837,500**
Total with 25% Engineering, Legal, Administration				**$ 8,546,875**
Beca	No Projects			

Project Costs for Buildout Improvements within Basins (in Addition to 80k/120k Projects)

Subbasin	Upstream MH	Pipe Dia (inches)	Pipe Length (feet)	Cost[1]
Subtotal				$ -
Lower Oak Creek	No Projects			
Subtotal				$ -
Squaw Creek	3471034	30	375	$ 137,500
	3471035	30	392	$ 150,000
	3471036	30	162	$ 62,500
	3671040	30	538	$ 200,000
	3668009	21	261	$ 62,500
	3668005	21	221	$ 62,500
	3668002	21	542	$ 137,500
	3668001	21	222	$ 62,500
	3868001	18	493	$ 112,500
	3668010	24	300	$ 87,500
	3671030	24	25	$ 12,500
	3671031	24	240	$ 75,000
	3671032	24	256	$ 75,000
	3671033	24	321	$ 100,000
	3671034	24	317	$ 100,000
	3671038	24	126	$ 37,500
	3671039	30	520	$ 187,500
	3868002	18	503	$ 112,500
	3868003	18	448	$ 100,000
	3868004	18	572	$ 125,000
	3865001	18	440	$ 100,000
	3865002	18	341	$ 75,000
	3668003	21	311	$ 75,000
	3471033	30	205	$ 75,000
Subtotal			**8,131**	**$ 2,325,000**
Total with 25% Engineering, Legal, Administration				**$ 2,906,250**
Country Club	3268002	24	178	$ 50,000
(in addition to Country Club Trunk)	3268001	30	522	$ 187,500
	3465019	24	467	$ 137,500
	3468007	24	109	$ 37,500
	3468006	24	533	$ 162,500
	3468005	24	451	$ 137,500
	3468004	24	459	$ 137,500
	3468002	24	304	$ 87,500
	3468001	30	605	$ 225,000
	3471040	30	423	$ 162,500
	3471038	30	173	$ 62,500
	3474012	36	324	$ 150,000
	3474013	36	310	$ 137,500
	3474016	36	433	$ 187,500
	3474017	36	933	$ 412,500
	3474002	36	345	$ 150,000
	3474001	36	241	$ 112,500
	3465013	24	480	$ 137,500
	3462001	15	380	$ 75,000
	3465030	15	360	$ 62,500
	3465031	15	536	$ 100,000
	3465018	24	380	$ 112,500
	3471007	36	68	$ 25,000
	3471037	30	358	$ 137,500
	3674034	36	362	$ 162,500
Subtotal			**9,735**	**$ 3,350,000**
Total with 25% Engineering, Legal, Administration				**$ 4,187,500**
South Corvallis	Diversion at Cummings	15	1100	$ 200,000
(in addition to Marys River PS, and Parallel Interceptor)	Diversion at Wake Robin	15	1100	$ 200,000
Subtotal			**2,200**	**$ 400,000**

Project Costs for Buildout Improvements within Basins (in Addition to 80k/120k Projects)

Subbasin	Upstream MH	Pipe Dia (inches)	Pipe Length (feet)	Cost[1]
Total with 25% Engineering, Legal, Administration				**$ 500,000**
Marys River	No projects in addition to Marys River Interceptor and Brooklane PS			
Subtotal				**$ -**
Total Cost All Basins				**$ 18,953,125**
[1]Total Project Cost includes 25% Contingency and 25% Engineering, Legal, Administration				
ENR-CCI=6500				

Project Cost Phasing Line Items for Conveyance Improvements within Basins Through 80,000 Population Milestone

Subbasin	Upstream MH	Pipe Dia (inches)	Pipe Length (feet)	Existing Pipeline Cost[1]	60,000 Pipeline Cost[1]	80,000 Pipeline Cost[1]
Garfield	5068020	12	296		$ 43,349.20	$ -
	5068027	12	91		$ 13,326.95	$ -
	5068044	15	214			$ 39,098
	4868047	12	330			$ 48,329
	4868050	12	326	$ 47,742.70		$ -
	4868051	12	147	$ 21,528.15		$ -
	5068063	12	400			$ 58,523
	5068045	12	278			$ 40,698
	4868048	12	143			$ 20,920
Subtotal			**2,224**	**$ 69,000**	**$ 57,000**	**$ 208,000**
Total with 25% Contingency				**$ 86,250**	**$ 71,250**	**$ 260,000**
Total with 25% Engineering, Legal, Administration				**$ 110,000**	**$ 90,000**	**$ 330,000**
Highland	No Projects					
Subtotal				**$ -**	**$ -**	**$ -**
North Ninth	5083032	15	43		$ 7,856.10	
	5083034	15	174		$ 31,789.80	
	5083013	15	411		$ 75,108.70	
	5283018	15	366		$ 66,954.43	
	5283017	15	174		$ 31,846.80	
	Diversion	18	2,800		$ 788,220.00	
Subtotal			**3,969**		**$ 1,002,000**	**$ -**
Total with 25% Contingency					**$ 1,250,000**	**$ -**
Total with 25% Engineering, Legal, Administration					**$ 1,570,000**	**$ -**
Northeast (Crescent Valley Interceptor)	5286039	48	299			$ 175,154
	5286040	48	307			$ 179,841
	5286041	48	237			$ 138,835
	5286042	48	89			$ 52,136
	5286043	48	389			$ 227,876
	5283047	48	199			$ 116,574
	4483006	48	438			$ 256,580
	4483007	48	274			$ 160,509
	4483008	48	274			$ 160,509
Subtotal			**2,506**			**$ 1,468,000**
Total with 25% Contingency				**$ -**	**$ -**	**$ 1,840,000**
Total with 25% Engineering, Legal, Administration				**$ -**	**$ -**	**$ 2,300,000**
Beca	No Projects					
Subtotal						**$ -**

Project Cost Phasing Line Items for Conveyance Improvements within Basins Through 80,000 Population Milestone

Subbasin	Upstream MH	Pipe Dia (inches)	Pipe Length (feet)	Existing Pipeline Cost[1]	60,000 Pipeline Cost[1]	80,000 Pipeline Cost[1]
Lower Oak Creek	No Projects					
Subtotal						$ -
Squaw Creek	No Projects					
Subtotal						$ -
Country Club	3471032	36	89			$ 39,245
	3471006	36	321			$ 141,185
	3471008	36	83			$ 36,331
	3471009	36	288			$ 126,490
	3471010	36	321			$ 140,858
	3471011	36	321			$ 141,055
	3471007	36	313			$ 137,306
Subtotal			**1,735**			**$ 762,000**
Total with 25% Contingency						**$ 950,000**
Total with 25% Engineering, Legal, Administration						**$ 1,200,000**
South Corvallis	Diversion at Tunison	18	1,100			$ 240,845
	SCVO Diversion Pipe (Bore)		11,000			$ 7,700,000
Subtotal			**12,100**			**$ 7,941,000**
Total with 25% Contingency						**$ 9,930,000**
Total with 25% Engineering, Legal, Administration						**$ 12,410,000**
Marys River Interceptor	3677043	48	243			$ 142,349
	3680016	48	847			$ 496,173
Subtotal			**1,090**			**$ 639,000**
Total with 25% Contingency						**$ 800,000**
Total with 25% Engineering, Legal, Administration						**$ 1,000,000**
Total Cost All Basins				**$ 110,000**	**$ 1,660,000**	**$ 17,240,000**
[1]Total Project Cost includes 25% Contingency and 25% Engineering, Legal, Administration. Note: Costs are for pipelines only; pump station costs are shown in other tables.						
ENR-CCI=6500						

APPENDIX E

Present-Worth Cost Estimates for Treatment Facilities

Table E-1

50-Year Present Worth Cost of Headworks Alt. No. 1: Expand the Existing Headworks

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST	FUTURE COSTS
Pump Station Expansion							
New Pumps	Raw Sewage	4 ea	16.7 mgd	20	$140,000	$560,000	$2,031,000
Wetwell Modifications	Demolition	1 LS	allowance	20	$500,000	$500,000	
Headworks Expansion							
Dewatering	Deep wells	1 LS	see attached	20	$75,000	$75,000	
Earthwork/Demolition		1 LS	see attached	20	$200,000	$200,000	$200,000
Concrete/Metals	Cast-in-place	1 LS	see attached	20	$225,000	$225,000	$175,000
Parshall flumes	FRP	3 ea	2-ft/24 mgd	20	$9,500	$28,500	$20,000
Process Total						**$1,060,000**	$2,426,000

MISCELLANEOUS COSTS

Installation	15% of fixed equipment		$84,000
Mobilization	5% of total		$104,227
Subtotal			**$1,248,227**

ALLOWANCES

Sitework/Infrastructure	15% of total		$187,234
Electrical and I&C	17% of total		$212,199
Mechanical	10% of total		$124,823
Contingency	25% of total		$312,057
Construction Cost			**$2,084,539**

Engineering/Legal/Admin	25%	$521,135
Total Capital Cost		**$2,605,674**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Power	201.7 kWh/mgd	1052773 kWh/yr	$0.05 /kwh	$52,639
Labor	0.50 FTE	1040 hrs/yr	$28.00 /hr	$29,120
Maint.	5% of equip	incl. screens		$45,500
Total Annual Cost				**$127,259**

Analysis Parameters

Annual Avg Flow (for O&M)	14.3 mgd
Peak Hour Flow (for sizing)	50 mgd
Discount Factor	5.50%
Projected Life	50 years

Present Worth of Annual Costs	$2,154,682
Total Capital Costs	$2,605,674
Present Worth of Future Costs	$166,828
Total Project Present Worth	**$4,760,356**

50-Year Present Worth Cost of Headworks Alt. No. 2: Construct a New Headworks Facility

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Pump Station						
Wetwell		1 LS	125,000 gal	20	$175,000	$175,000
Pump Station		2,400 SF	2-levels	20	$250	$600,000
New Pumps	Raw Sewage	4 ea	16.7 mgd	20	$140,000	$560,000
						$0
Screenings						
Screenings channels	Cast-in-place Conc	560 CY	3 screens/1 bypass	20	$750	$420,000
Screenings handling area	Slab-on-grade	1,200 SF		20	$200	$240,000
Mechanical Screens	Climber screen	2 ea	4-ft/32 mgd	20	$160,000	$320,000
Bypass Screen	Manually-cleaned	1 ea	6-ft	20	$40,000	$40,000
Screenings Press	Hydraulic	2 ea	30 CF/hr	20	$25,000	$50,000
Screenings conveyor	Shaftless screw	1 ea	12-in/15 CF/hr	20	$40,000	$40,000
Parshall flume	FRP	2 ea	3-ft/32 mgd	20	$12,000	$24,000
						$0
Process Total						**$2,469,000**

MISCELLANEOUS COSTS

Installation	15% of fixed equipment		$149,100
Mobilization	5% of total		$290,900
Subtotal			**$2,909,000**

ALLOWANCES

Sitework/Infrastructure	5% of total		$290,900
Electrical and I&C	15% of total		$872,700
Mechanical	5% of total		$290,900
Contingency	25% of total		$1,454,500
Construction Cost			**$5,818,000**

Engineering/Legal/Admin.	25%	$1,454,500
Total Capital Cost		**$7,272,500**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Power	180.5 kWh/mgd	942119.8 kWh/yr	$0.05 /kwh	$47,106
Labor	0.25 FTE	520 hrs/yr	$28.00 /hr	$14,560
Maint.	5% of equip	incl. screens		$48,500
Total Annual Cost				**$110,166**

Analysis Parameters

Annual Avg Flow (for O&M)	14.3 mgd
Peak Hour Flow (for sizing)	50 mgd
Discount Factor	5.50%
Projected Life	50 years

Present Worth of Annual Costs	$1,865,277
Total Capital Costs	$7,272,500
Total Project Present Worth	**$9,137,777**

Present Worth Cost of Secondary Treatment Alt. No. 1: Trickling Filter/Activated Sludge

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Trickling Filter						
Earthwork		1 LS			$50,000	$50,000
Concrete (Slab)		2,233 cy	160' dia.	50	$200	$446,578
Concrete (Walls)		377 cy	160' dia.	50	$400	$150,720
Distributor Mechanism	Al motor driven	1 ea	160' dia.	20	$175,000	$175,000
Media (8 feet deep)	Plastic	160,768 cu. ft.	160' dia.	20	$4	$643,072
Recycle Pump Station						
Earthwork		1 LS			$15,000	$15,000
Concrete (Slab)		93 cy		50	$200	$18,519
Concrete (Walls)		111 cy		50	$400	$44,444
Pumps	Centrif. pumps	1 ea	11.5 mgd	20	$100,000	$100,000
Change Existing Media						
Media (8 feet deep)	Plastic	0 cu. ft.	Two -160' dia.	20	$4	$0
Process Total						**$1,643,333**

MISCELLANEOUS COSTS

Installation	15% of fixed equipment		$137,711
Mobilization	5% of construction total		$146,428
Subtotal			**$1,927,471**

ALLOWANCES

Sitework/Infrastructure	5% of construction total		$146,428
Electrical and I&C	15% of equipment total		$73,511
Mechanical	10% of equipment total		$49,007
Contingency	25% of construction total		$732,139
Construction Cost			**$2,928,557**

Engineering/Legal/Admin	25%	$732,139
Total Capital Cost		**$3,660,696**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Trickling Filter				
Power	63 hp	409,216 kw-hrs/yr	$0.04 /kwh	$16,369
Labor	2.00 FTE	4160 hrs/yr	$28.00 /hr	$116,480
Maint.	5% of equip			$13,750
Total Annual Cost				**$146,599**

Analysis Parameters

Discount Factor	5.50%
Projected Life	20 years

Present Worth of Annual Costs	$1,751,910
Total Capital Costs	$3,660,696
Total Project Present Worth	**$5,412,605**

Table E-4

Present Worth Cost of Secondary Treatment Alt. No. 2: Roughing Filer/Activated Sludge

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Aeration Basins						
Basins and Blowers	Fine pore	187,200 cu. ft.	156' x 40'	20	$15.18	$2,841,696
Process Total						**$2,841,696**

MISCELLANEOUS COSTS

Installation (included)	0%		$0
Mobilization	5% of construction cost total		$202,978
Subtotal			**$3,044,674**

ALLOWANCES

Sitework/Infrastructure	0% of construction total		$0
Electrical and I&C	0% of equipment total		$0
Mechanical	0% of equipment total		$0
Contingency	25% of construction total		$1,014,891
Construction Cost			**$4,059,566**

Engineering/Legal/Admin	25%	$1,014,891
Total Capital Cost		**$5,074,457**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Aeration Basins				
Power	411 hp	2,684,867 kw-hrs/yr	$0.04 /kwh	$107,395
Labor	2.00 FTE	4160 hrs/yr	$28.00 /hr	$116,480
Maint.	5% of 25% of process subtotal			$35,521
Total Annual Cost				**$259,396**

Analysis Parameters

Avg Annual Solids Loading (for O&M)	475 dry ton/year
Avg Annual Solids Loading (for sizing)	700 dry ton/year
Discount Factor	5.50%
Projected Life	20 years

Present Worth of Annual Costs	$3,099,880
Total Capital Costs	$5,074,457
Total Project Present Worth	**$8,174,337**

Table E-5
Present Worth Cost of Solids Handling Alt. No. 1: Liquid Sludge Hauling

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Facultative Lagoons						
Lagoons	Lined	1 LS	4.4 AC/2.6MG	20	$1,300,000	$1,300,000
Dredge	Mudcat (exist)	1 ea	1000 gpm	15	$135,000	$135,000
Sludge Pumping						
Truck Loading Pumps	Wemco Hydrostal	2 ea	1000 gpm	20	$12,000	$24,000
Sludge Day Tank	Vertical steel	2 ea	50,000 gal	20	$12,500	$25,000
Liquid Sludge Application						
Portable Sludge Pump		1 ea		15	$35,000	$35,000
Portable Mainline		1 ea		15	$10,000	$10,000
Portable Holding Tank		1 ea		15	$50,000	$50,000
Sludge Sprinkler	Traveling Big Gun	1 ea		15	$30,000	$30,000
Semi-Tractors		1 ea		15	$200,000	$200,000
Equipment Hauling Trailers		1 ea		15	$35,000	$35,000
Tanker Trailers		1 ea	6000-gallon	15	$60,000	$60,000
Process Total						**$1,904,000**

MISCELLANEOUS COSTS

Installation	15% of fixed equipment		$7,350
Mobilization	5% of total fixed construction		$74,200
Subtotal			**$1,985,550**

ALLOWANCES

Sitework/Infrastructure	5% of total		$145,768
Electrical and I&C	15% of Sludge Pumping total		$27,600
Mechanical	15% of Sludge Pumping total		$27,600
Contingency	25% of total		$728,839
Construction Cost			**$2,915,357**

Engineering/Legal/Admin	25%	$728,839
Total Capital Cost		**$3,644,196**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Dredging				
Fuel	25.00 gal/hr	77 hrs	$1.20 /gal	$2,298
Labor	1.0 FTEs	800 hrs	$24.00 /hr	$19,200
Maint.	12% of equip			$16,200
Hauling				
Truck Loading	3.1 kWh/DT	1930 dry tons	$0.045 /kWh	$269
Hauling	5% solids	9,190 kgal	$30.00 /kgal	$275,714
Labor	2.0 FTEs	1600 hrs	$28.00 /hr	$44,800
Disposal				
Pump elec	6.2 kWh/DT	1930 dry tons	$0.05 /kWh	$538
Labor	2.5 FTEs	2000 hrs	$24.00 /hr	$48,000
Equipment Maint.	5% of equip.			$21,000
Total Annual Cost				**$428,020**

Analysis Parameters

Avg Annual Solids Loading (for O&M)	1930 dry ton/year
Avg Annual Solids Loading (for sizing)	2580 dry ton/year
Discount Factor	5.50%
Projected Life	20 years

Present Worth of Annual Costs	$5,114,998
Total Capital Costs	$3,644,196
Total Project Present Worth	**$8,759,194**

Table E-6
Present Worth Cost of Solids Handling Alt. No. 2: Mechanical Dewatering

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Dewatering						
Dewatering Equipment	BFP	2 ea	2m/1200lb/hr	25	$240,000	$480,000
BFP Feed Pumps	Progressing Cavity	2 ea	10 - 100 gpm	25	$25,000	$50,000
Belt Washwater Pumps	Centrifugal	2 ea	75 gpm	25	$5,000	$10,000
Dewatering Building	Conc Frame	3200 SF	2 levels	50	$200	$640,000
Sludge Loadout Conveyor	Belt Conveyor	150 ft	24-in/3 ton/hr	25	$500	$75,000
Polymer Prep Equipment	Liquid Mix/Dilution	1 ea		25	$250,000	$250,000
Polymer Feed Pumps	Progressing Cavity	4 ea	50 gpm	25	$15,000	$60,000
Polymer Tanks	FRP	1 ea	6000 gal	25	$10,000	$10,000
Sludge Cake Storage						
Storage Pad	Covered	35300 SF	180 days	20	$17	$600,100
Drying/Hauling/Disposal						
Spreader	Manure spreader	1 ea	12 CY	15	$65,000	$65,000
Dump Trailers	Ram-Eject	2 ea	20 CY	15	$90,000	$180,000
Front-end Loader	Caterpillar	1 ea	3 CY bucket	15	$180,000	$180,000
Process Total						**$2,600,100**

MISCELLANEOUS COSTS

Installation	15% of equipment		$129,000
Mobilization	5% of total		$267,817
Subtotal			**$2,996,917**

ALLOWANCES

Sitework/Infrastructure	5% of total		$267,817
Electrical and I&C	15% of total		$803,450
Mechanical	5% of total		$267,817
Contingency	25% of total		$1,445,333
Construction Cost			**$5,781,333**

Engineering/Legal/Admin	25%	$1,445,333
Total Capital Cost		**$7,226,667**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Dewatering				
Electrical	12.0 kWh/ton	23160 kWh	$0.045 /kwh	$1,042
Polymer	15.0 lb/ton	28950 lb	$2.00 /lb	$57,900
Labor	1.0 FTEs	2080 hrs	$28.00 /hr	$58,240
Maint.	5% of equip			$45,750
Hauling & Disposal				
Truck loading	front end loader	1,930 dry tons	$0.80 /DT	$1,544
Hauling	22% solids	8,773 wet tons	$5.25 /WT	$46,057
In-field application	incl. spreaders & operators	1930 dry tons	$12.00 /DT	$23,160
Total Annual Cost				**$233,693**

Analysis Parameters

Annual Solids Loading (for O&M)	1930 dry ton/year
Annual Solids Loading (for sizing)	2580 dry ton/year
Discount Factor	5.50%
Projected Life	20 years

Present Worth of Annual Costs	$2,792,721
Total Capital Costs	$7,226,667
Total Project Present Worth	**$10,019,388**

Table E-7. Present Worth Cost of Gas Utilization Option No. 4: Resale of Digester Gas

EQUIPMENT ITEM	TYPE	UNITS	SIZE/ CAPACITY	LIFE (years)	UNIT COST	TOTAL PW COST
Gas Conditioning						
H2S Removal	Iron Sponge	1 ea	4-in/30,000 cu ft/d	25	$40,000	$40,000
Particulate filter	Cartridge	1 ea	4-in/50 scfm	25	$12,000	$12,000
Moisture separator		2 ea	50 scfm	25	$5,000	$10,000
Gas Conveyance						
Gas compressor	Turblex	1 ea	50 scfm	20	$35,000	$35,000
Gas pipeline	HDPE	6000 ft	4-in	25	$40	$240,000
Gas Storage						
Storage system	Membrane	1 LS		20	$250,000	$250,000
Process Total						**$587,000**

MISCELLANEOUS COSTS

Installation	15% of equipment		$14,550
Mobilization	5% of total		$55,355
Subtotal			**$656,905**

ALLOWANCES

Sitework/Infrastructure	5% of total		$43,355
Electrical and I&C	10% of total		$86,710
Mechanical	5% of total		$43,355
Contingency	25% of total		$276,775
Construction Cost			**$1,107,100**

Engineering/Legal/Admin	25%	$276,775
Total Capital Cost		**$1,383,875**

ANNUAL COSTS

Item	Usage	Qty	Unit Cost	Total Cost
Gas Conveyance				
Electrical	175.0 kWh/day	63875 kWh	$0.045 /kwh	$2,874
Labor	0.1 FTEs	208 hrs	$28.00 /hr	$5,824
Maint.	5% of equip			$4,850
Gas Conditioning				
Media replacement	every 5 years	240 lbs/yr	$2.00 /lb	$480
Gas Resale				
Revenue	30000 CF/day & operators	10950000 CF	($0.002) /CF	($21,900)
Total Annual Cost				**($7,872)**

Analysis Parameters

Discount Factor	5.50%
Projected Life	20 years

Present Worth of Annual Costs	($94,069)
Total Capital Costs	$1,383,875
Total Project Present Worth Cost	**$1,289,806**

Table E-8
Cost Estimates for "Most Probable" Process Improvements

Unit Process	Total Project Costs			
	Existing	60,000 pop	80,000 pop	Buildout
Influent Pumping/Headworks			$2,606,200	$3,032,500
Primary Treatment		$3,000,000	$1,687,500	$4,375,000
Biological Treatment		$406,200	$3,662,500	$2,250,000
Secondary Clarification		$4,237,500	$2,250,000	$6,356,250
Disinfection		$2,380,000		$833,750
Filtration			$10,856,300	$5,487,500
Tertiary Treatment				
Effluent Pumping			$1,900,000	$350,000
Solids Handling	$4,150,000		$4,300,000	$4,150,000
Odor Control	$0	$968,800	$150,000	$187,500
Non Process Facilities	$179,100	$221,000	$187,500	$312,500
Total Project Cost	**$4,329,100**	**$11,213,500**	**$27,600,000**	**$27,335,000**

Cost estimates include 25 percent construction contingency and 25 percent allowance for engineering, legal, and administrative costs. They do not include seismic improvements. Cost estimates are in 1998 dollars indexed to the Engineering News Record construction cost index of 6,500 for the Seattle area.

APPENDIX F

Seismic Hazard Evaluation Report

Seismic Vulnerability Evaluation *for the* Corvallis Wastewater Reclamation Plant

CORVALLIS
ENHANCING COMMUNITY LIVABILITY
THROUGH IMPROVED WATER QUALITY

June 1998

CH2MHILL
2300 NW Walnut
Corvallis, OR 97330

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Section 1

INTRODUCTION

This report summarizes our evaluation of the seismic vulnerability of the existing Corvallis Wastewater Reclamation Plant (WWRP). The purpose of this study is to identify any vulnerable portions of the plant, identify structural deficiencies, and prioritize the vulnerability of each building or process facility for potential repair or upgrade.

Scope of Work

CH2M HILL conducted a vulnerability assessment of the Corvallis WWRP facilities. The assessment included the following tasks:

- **Geotechnical Site Assessment:** Determine the regional seismicity, estimate the peak ground acceleration for an earthquake with a 10 percent chance of exceedance in 50 years, assess seismically induced ground failure hazards such as liquefaction and lateral spreading, and summarize the geotechnical issues.
- **Review Existing Facility Construction Documents:** Review all available construction documents for the original plant and for subsequent additions.
- **Onsite Visual Inspection:** Walkthrough of site to visually assess the condition of the existing structures and to identify any non-structural hazards.
- **Vulnerability Assessment:** Perform an overview evaluation of all buildings and process facilities for post-earthquake performance based on the information gathered from the construction document review and onsite visual inspection.
- **Prioritization of Plant Facilities:** Based on the results of the vulnerability assessment, rank each building and process facility according to order of magnitude of risk for seismic vulnerability.
- **Corrective Actions:** Based on the results of the vulnerability assessment, suggest corrective actions or further analysis for the vulnerable portions of the structures or nonstructural components that have been identified as being at risk.

Investigation Procedure

The approach for assessing the risk level for each building and process facility was to review the existing drawings and perform an onsite visual inspection of the plant. The drawing review identified the type of construction for all the buildings and process facilities at the plant. The onsite visual inspection verified the construction and assessed the condition of the facilities. In addition to the buildings and facilities, nonstructural elements were inspected for their risk potential. Such nonstructural elements included self-supporting tanks, piping, electrical ductwork, storage racks, etc. Damage to nonstructural elements could present a life safety, health, or economic risk. The visual inspection was cataloged using photographs taken at the time of the inspection. Obviously, only those elements, building systems, and processes that are visible could be evaluated; therefore, the existing drawings were consulted whenever possible to determine the potential hazard of those items not visible during the inspection.

Following the visual inspection, the buildings and process facilities were assigned risk levels for the three primary hazards as described earlier. Based on the assigned risk levels, the buildings and facilities were ranked in order of seismic risk potential.

Plant History

The Corvallis WWRP was constructed in 1953 through 1954. Following the original construction, the plant was upgraded in 1966 and 1978. Table 1-1 summarizes the buildings and process facilities, and lists their date of construction.

TABLE 1-1
Building and Process Facility Construction History

Facility	No.	Type of Facility	Year of Construction
Digester Complex	6200,6300,6400	Building/Process	1953
Primary Clarifier	6700	Process	1953
Primary Digester	6100	Process	1966
Trickling Filters	4010,4020	Process	1966
Secondary Clarifier/Chlorine Contact	4200	Process	1966
Filter Pump Station		Process	1966
Filter Manhole		Process	1966
Secondary Sludge Pump Station	4300	Building	1966/1978
Control Gate Structure	0400	Process	1978
Headworks	2500	Process	1978
Influent Pump Station	2000	Building/Process	1978
Primary Sedimentation	3100	Process	1978
Secondary Clarifiers	5300,5400	Process	1978
Aeration Tanks	5100	Process	1978
Effluent Pump Station	0300	Process	1978
Outfall Diversion		Process	1978
Tunnels and Pipe Chase		Process	1978
Operations and Maintenance Building	1000	Building	1978

The plant is scheduled to undergo another upgrade in the summer 1998; therefore, facilities that are to be demolished in their entirety during the new upgrade were not evaluated. Also, all of the new construction is designed to the criteria in the 1994 Uniform Building Code (UBC), which is the most current seismic design code. Again, any facilities slated for upgrade were not evaluated.

Plant Operation

The WWRP is owned and operated by the City of Corvallis Department of Public Works and serves the entire city, providing secondary treatment of wastewater flows. The plant includes influent pumping and screening following by aerated grit removal and primary sedimentation. The secondary process consists of trickling filters followed by aeration and secondary clarification. Disinfected effluent enters the Willamette River via the outfall diffusers. Solids recovered from the process are anaerobically digested, stored in facultative sludge lagoons, and ultimately land-applied. The plant has a dry weather capacity of 9.7 million gallons per day (MGD) and a hydraulic capacity of 28 MGD.

Power Supply

The main power source for the Corvallis WWRP will be a new 20.8 kV power service being installed by Pacific Power from the Grant Street Substation. However, power will continue to be available from the plant's existing 20.8 kV service, which is supplied by Pacific Power's Peoria feeder from the Buchanan Substation. In addition, an onsite generator provides standby power for a number of nonprocess loads. The process equipment cannot be powered from the standby generator.

The new main service will be located at the northwest corner of the plant site. Medium voltage conductors will be installed in a concrete-encased duct bank from the service location to a pad-mounted switch located nearby. Overcurrent protection is provided at the switch for two feeders, one to a new transformer located at the new Disinfection Building and the second to the rest of the plant. The second feeder will be tapped in a manhole near the new combined sewer overflow (CSO) facility to provide power to the CSO facility through two new pad-mounted transformers. The feeder continues to a new pad-mounted automatic transfer switch located near the existing service transformers where it serves the existing plant. The new service is the preferred source at the transfer switch and the existing service will be the secondary power source. The existing service conductors will be extended to the CSO facility, where it will be possible to supply each transformer through a manual transfer switch located in the transformer.

The existing plant and new equipment supplied from the existing transformers will be provided continued power supply from the existing transformers should the main source fail. If power fails, the new pad-mounted automatic transfer switch will automatically transfer its loads to the secondary power supply. Processes fed from the existing transformers include influent pumping, primary and secondary treatment, digestion, and effluent pumping. The existing building facilities are also supplied from the existing transformers.

The new CSO facility can be supplied power from either the main service or secondary power source by either the manual switches supplied in the transformers, or automatically by the automatic transfer controls provided as part of the switchboard in the CSO facility. The switchboard is used if one transformer is being supplied power by the main service and the other is being supplied power by the secondary power source. The switchboard in the CSO facility provides power influent pumping and screenings removal during high flow conditions.

The new Disinfection Building will normally be provided power from a new pad-mounted transformer fed from the new main service. Should this power supply fail, standby power will be supplied for process requirements through an automatic transfer switch from the existing plant power supply.

In summary, it is possible for all areas of the plant to continue to function normally if either the main power service or the secondary power supply is energized. Should both power supplies be lost, only limited nonprocess loads would continue to operate.

Section 2

SEISMIC HAZARDS

Geotechnical Investigations

No exploratory test holes were drilled as part of this work. The logs of previously drilled test holes were reviewed for this evaluation.

Subsurface Conditions

Logs from previous borings indicate the site is underlain by approximately 10 to 20 feet of Willamette Silt. This material is described as soft to medium stiff brown plastic clayey silt. A thin layer of loose silty sand underlies the silt. This is underlain by dense sand and gravel formations (Linn Gravels). In some places, stiff to hard gray clayey silt has been found between the Willamette Silt and Linn Gravels.

Groundwater Levels

Groundwater levels at the site during summer months are generally between 5 and 10 feet below ground surface (bgs). During wet weather, it is common for the water table in the Willamette Valley to rise to within a few feet of the ground surface.

Geologic Profile

The project site is located in the Willamette Valley physiographic province of Oregon (Orr, et al., 1992). Geology of the Corvallis, Oregon, area has been mapped by the United States Department of Geology and Mineral Industries (DOGAMI). These maps indicate that the upper 50 to 100 feet or more of the geologic profile is primarily Quaternary Lower Terrace Deposits of the Holocene era. This material consists of semi-consolidated cobbles, gravel, sand, silt, clay, and organic matter of variable thickness on lowland terraces above recent alluvium and along usually entrenched tributary rivers and streams.

Geologic mapping indicates that sandstone bedrock of Middle to Late Eocene Age underlies the Quaternary Lower Terrace Deposits. Estimated depth of bedrock is 50 to 100 feet.

Seismic Characterization

Geomatrix Consultants completed a study in 1995 for the Oregon Department of Transportation (ODOT) describing the level of seismic hazard throughout the State of Oregon (Geomatrix Consultants, 1995). This report includes information indicating the location, site, and frequency of occurrence of earthquakes in and near Oregon. Much of the seismic-related information contained in this section is taken from the Geomatrix report.

Historic Seismicity

Historic seismicity in the site vicinity since approximately 1840 has been low to moderate. Jacobson (1986) indicates 14 earthquake occurrences within a 30-mile radius of Corvallis in the time period between 1841 and 1986. One of these events was greater than magnitude 5; two of these events were greater than magnitude 4; and five of these events were greater than magnitude 3.

The most recent regional event of significance was the magnitude 5.6 event that occurred on March 25, 1993, near Scotts Mills, Oregon, about 40 miles northeast of the site. No faults directly coincide with the epicenter of the 1993 Scotts Mills event, but the Mount Angel Fault is the closest known fault and the only candidate for a causative fault for that earthquake (Madin, et al., 1993). Geologic studies in the Pacific Northwest suggest that the record of historic seismicity may not be a useful indicator for expected levels of future seismicity. For this reason, the level of potential ground motion is determined by considering the activity at all seismic sources for the area, some of which have been relatively inactive in the past 160 years of recordings.

Sources of Seismicity

On the basis of geologic evidence gathered recently from coastal areas in Washington and Oregon, scientists believe that very large earthquakes have originated offshore from Oregon and Washington, along the principal tectonic feature of the Pacific Northwest. This feature involves an active subduction zone located off the western coast, where the Juan de Fuca plate subducts beneath the North American plate along the Cascadia margin. The presence of this Cascadia subduction zone west of Oregon provides the second to two primary seismic sources that can affect ground motion in the state. The other primary source involves crustal earthquakes occurring within the North American plate.

Cascadia Subduction Zone Sources

There are two types of subduction zone sources: the Juan de Fuca-North America *plate interface* earthquakes, and *intraslab* earthquakes. Plate interface earthquakes are usually thrust faults at depths of less than 30 km and are associated with the largest earthquakes observed worldwide. Intraslab earthquakes occur within the subducting Juan de Fuca plate and are deeper than interface earthquakes. Intraslab earthquakes are typically normal faults.

Crustal Sources

The Scotts Mills earthquake was an example of a crustal earthquake. These crustal earthquakes are further categorized as occurring on *discrete fault sources* where repeated earthquakes have occurred in the geologic past on specific geologic features, or within *regional areal source zones* where earthquakes have been observed and will likely occur again but have not been associated with any specific geologic features.

Previous geologic mapping of the project's region shows few identified faults in the Corvallis area (Walker and MacLeod, 1991; Rogers, et al., 1991). The nearest fault is the Corvallis fault, located approximately 2 miles northwest of the site. This fault has shown evidence of post-Eocene movement. Geomatrix (1995) reports four additional faults within approximately 30 miles of the project site. Table 2-1 lists these faults and their approximate distance from the project site.

TABLE 2-1
Closest Sources of Seismicity, and Distance from Site

Fault	Approximate Distance to Site, Miles (Direction)
Corvallis Fault	2 (NW)
Salem Hills Structure, Waldo Hills Fault	21 (NE)
Mill Creek Fault	19 (NE)
Owl Creek Fault	3 (E)

Source: Table 2-6 and Plates 1 and 2, Seismic Design Mapping, State of Oregon. Geomatrix Consultants, 1995.

The project site is situated within the southern Willamette Valley zone as outlined in the Geomatrix report (1995). It is a zone characterized by low seismic activity and few mapped Quaternary faults. Because of uncertainties and limitations in knowledge, the normal approach is to assume that crustal earthquakes could occur in any region where those earthquakes will have a magnitude of 6 to 6.5 and be located in the upper 15 to 30 km (Geomatrix, 1995).

Ground Acceleration and Earthquake Magnitude

Results of Geomatrix's statewide seismic design map project suggest that the peak ground acceleration on rock for the project site will be approximately 0.20g where g is the acceleration of gravity. This acceleration is for a 500-year return period (mean annual frequency of exceedance of 0.002)—that is, a 10 percent chance of exceedance in 50 years—and takes into account contributions from all seismic sources.

The relative contribution from each of the two types of seismic sources is as follows:

- The fraction of the hazard resulting from crustal earthquakes is approximately 25 percent, meaning that roughly 25 percent of the total number of annual events that exceed 0.20g are anticipated to be crustal earthquakes and 75 percent are anticipated to be subduction zone earthquakes.
- The magnitude of the crustal earthquakes that exceed 0.20 g is expected to be approximately 5.0. The magnitude of intraslab subduction zone earthquakes is anticipated to be approximately 6.0. For interlace subduction zone earthquakes, the magnitude is anticipated to be approximately 8.5 (Geomatrix, 1995, Fig. 4-6a).

Procedures that have been adopted in the 1997 edition of the Uniform Building Code (UBC) were used to determine soil profile types and site coefficients. Based on blow counts and the generally stiff subsurface conditions recorded during the geotechnical exploration, the site was classified as Soil Profile Type S_D.

Earthquake Induced Hazards

The following section briefly defines and discusses the susceptibility of the site to various types of earthquake-induced hazards.

Landslide - The project site and surrounding area are relatively flat and are not subject to landsliding or ground slumping.

Liquefaction- Liquefaction refers to the loss of shear strength that saturated soil deposits can experience during undrained cyclic loading, such as earthquake loading. Cyclic liquefaction occurs when pore pressures within soil increase to the point where the effective stress in the soil approaches zero. When this happens, the shear strength of the soil decreases and permanent deformations within the soil can accumulate to large values (greater than 5 percent strain). The susceptibility of a soil deposit to liquefaction is a function of the degree of saturation, soil grain size, relative density, percent fines, age of deposit, plasticity of fines, earthquake ground motion characteristics, and several other factors. Liquefaction can significantly affect both settlement and slope stability.

The soils most susceptible to liquefaction at the site are the soft to medium stiff silts in the depth range of 10 to 20 feet that have SPT N-values generally below 10 blows per foot (bpf). Fortunately, these deposits also contain some degree of clay materials as indicated by their high liquid limit. Clayey deposits generally do not liquefy as easily as sands or nonplastic silts. Based on the cohesive, fine-grained nature of this upper silt material, it appears that there is a low probability that the silt could liquefy. The probability that the underlying materials would liquefy is also low because of the depth and density of the granular deposits and the depth, cohesion, and particle size of the fine-grained soils.

A 3- to 5-foot-thick layer of loose sand is present in some areas that could be susceptible to liquefaction. However, this layer overlies a dense sand and gravel layer that would likely prevent significant amounts of pore pressure from building in the thin, loose sand layer.

Tsunami - A tsunami is a great sea wave produced by sub-marine earth movement. Because of its landlocked location, the treatment facility is not considered subject to damage or flooding from tsunamis.

Seiche - Seiche refers to the oscillation of the surface of a lake or landlocked sea that varies in period. The closest body of water to the project site is the Willamette River. The Willamette is not a contained body of water and is sufficiently far away to not be considered a seismic hazard.

Subsidence - Subsidence, or the downward settling of the ground surface, may be caused by natural geologic processes such as compaction, subsurface mining, or pumping of oil or water. The project site is not considered subject to seismic subsidence.

Fault Rupture - The potentially active faults that are present in the Corvallis vicinity are not adjacent to nor do they underlie the treatment facility. Even though these faults could be the source of an earthquake, the probability of a fault rupture near the site is low.

Section 3

ASSESSMENT CRITERIA AND CODES

The purpose of this section is to assess the level of seismic vulnerability for each building and process facility and their components.

Assessment Criteria

It is important to differentiate between life safety, health hazard, and economic loss; therefore, the following definitions will be used throughout this report to define the level of risk associated with each building and process facility at the plant:

- **Life Safety:** Failure of, or damage to, a building system, nonstructural secondary building component; or nonstructural process or supply system represents a direct threat to human life. For example, complete collapse of a building represents a direct threat to the occupants of the building; also, a release of chlorine into the atmosphere represents a direct threat to the occupants of the plant and nearby residents.
- **Health Hazard:** Partial failure of, or damage to, a building system, nonstructural secondary building component; or nonstructural process or supply system represents a threat to health but is not an immediate threat to life. For example, discharge of untreated waste into adjacent streams or the Willamette River would represent a threat to human health but not an immediate threat to life.
- **Economic Loss:** Localized failure of, or damage to, a building system, nonstructural secondary building component, or nonstructural process or supply system does not represent a threat to life or health. However, loss of function results in economic loss. For example, an expensive piece of equipment that may be damaged during an earthquake may not represent a threat to life or health, but the cost to replace the equipment could be significant.

All facilities at the plant will be evaluated based on the above criteria. A level of risk for each of the criteria will be assigned to each facility based on three levels: High, Moderate, or Low.

Building Codes

Current building codes are established to provide minimum design levels for new construction to minimize potential loss of life during an earthquake. The minimum design levels required by the codes are meant to prevent collapse or damage to the structure that could jeopardize human life; however, the minimum design levels do not necessarily limit damage to a structure. Structures subjected to code level forces will likely be damaged during an earthquake and may require substantial retrofitting to allow occupancy, or they may have to be razed.

At the time of their design, the structures comprising the Corvallis Wastewater Reclamation Plant met the most current code prescribed forces; however, as our understanding of earthquakes and structural behavior has increased, the codes have adopted newer and more stringent earthquake design levels and detailing requirements. As such, many of the structures were designed for lower than current earthquake forces and will be subject to forces significantly larger than assumed during the original design. For example, in 1990 western Oregon was upgraded from UBC Zone 2B to Zone 3. The result

of this upgrade is a 50 percent increase in the ground acceleration associated with the design earthquake.

The most current code for western Oregon is the 1994 UBC; however, in October 1998 the 1997 UBC will become the legally binding code. The seismic provisions for the 1997 UBC have been completely revised. The result of these revisions is higher seismic design loads and more stringent structural detailing requirements for the same code seismic zones. The 1997 UBC will be the legally binding code until approximately the year 2004, when the State of Oregon intends to adopt the new 2003 International Building Code (IBC). Indications are that the seismic provisions for the IBC will be significantly different from those of the UBC, and again could result in higher seismic design forces and more stringent structural detailing. Because of the evolutionary nature of design codes, it may be neither practical nor economically feasible to bring the current facilities up to current code force and detailing levels. Alternately, the International Congress of Building Officials publishes the Uniform Code of Building Conservation. This code specifically addresses retrofit and upgrade of existing facilities. As such, the design seismic forces required by the Uniform Code of Building Conservation should be used for any future retrofit or upgrade. Discussions with the City of Corvallis Building Department will be necessary to decide which building code will be enforced for future upgrades or retrofits.

For the purposes of this report, the vulnerability assessment is based on the provisions of the 1994 UBC.

Section 4

VULNERABILITY ASSESSMENT

The following section outlines the expected performance of the plant during an earthquake and the consequences for the plant of failures. The evaluation assumes that the plant will be subjected to a UBC prescribed earthquake; the prescribed earthquake equates to an effective peak acceleration of 0.30 g with a 10 percent chance of exceedance in 50. The value of 0.30 g at the ground surface is a code-required acceleration and is higher than the peak ground bedrock acceleration of 0.20 g as indicated in Chapter 2 of this report. The evaluation is limited to the buildings and process facilities at the Corvallis WWRP. Facilities and processes that are part of the overall wastewater treatment system for the City of Corvallis but that physically exist outside of the plant site have not been evaluated for expected performance or consequences of failure.

Each building and process facility and its components have been assigned levels of risk for each of three criteria: Life Safety, Health Hazard, and Economic Loss. The levels of risk assigned to each of the criteria are: High, Moderate, and Low. Based on these levels of risk, each building and process facility has been ranked according to its seismic vulnerability. The visual inspection and seismic evaluation was conducted in a chronological fashion, starting with the oldest construction and ending with the most recent.

Digester Complex

The digester complex is comprised of two digester tanks and a digester control building (see photos 3 and 4 in Appendix A). The digester control building shares common walls with the digester tanks. The construction of the digester tanks is conventionally reinforced, cast-in-place concrete walls and foundation with a floating wood roof. The exterior of the tanks is brick veneer. The digester control building is a two-story concrete building with exterior brick veneer. The bottom story is below grade. There are two associated structures near the digester complex that will be demolished during the combined sewer overflow project in the summer of 1998. Those facilities are the storage building (see photo 1) and the flair (see photo 2). Because these facilities are to be demolished, they were not evaluated.

Digester Tanks (Facilities 6200 and 6300)

Life Safety:	Moderate
Health Hazard:	Moderate
Economic Loss:	Moderate

Tanks that hold fluid are subject to the effects of liquid sloshing during an earthquake event. Sloshing causes the fluid in the tank to be accelerated laterally and the top surface of the fluid oscillates, causing an increase and decrease in water level on the walls. The resulting effect is a net increase in pressure on one side of the tank and a net decrease in pressure on the opposite wall of the tank. This type of unbalanced loading can cause localized increases in stress in the tank walls. Because the tank was designed without taking sloshing during an earthquake into consideration, the exterior tank walls could become overstressed and crack and leak; however, because of their geometry, circular abovegrade tanks have performed well in earthquakes. If cracking and/or leakage occurs, repair of the

tank would be necessary to continue operation. Such repair could include epoxy injection of cracks or local repair and replacement of damaged areas. Also, any leaked contents would represent a health hazard.

Secondary to the tank construction, the brick veneer also represents a seismic hazard. In reviewing the existing drawings, it does not appear that the brick veneer is anchored to the concrete tank walls. Even if anchors were used, brick anchors for facilities of this age have frequently suffered damage from corrosion due to rainwater or tank leakage, and do not function as intended. Unanchored brick might fall off the tank walls during an earthquake and represents a falling hazard to people near the tank during the event.

Digester Control Building (Facility 6400)

Life Safety: High
Health Hazard: Low
Economic Loss: High

The main earthquake resisting structural system of the digester control building is concrete shearwalls. The shearwalls were designed for forces less than those currently required by the UBC. As such, the walls will be subject to increased stresses and overturning. Overstress of the concrete shear walls will cause localized cracking but probably not failure of the shear walls. Any cracking can be repaired using epoxy crack injection.

The concrete beams supporting the first floor and roof levels between the digester tanks are supported on steel tube columns. The tube columns are adequate for vertical support of the beams only. Any movement of the building that causes the beams to displace laterally will induce bending in the columns and will likely overload the columns. Such lateral displacements are highly likely during an earthquake. If the columns were to become overloaded and buckle, the building floor or roof levels could collapse.

The brick cladding on the exterior is a seismic hazard. As in the case of the digester tanks, the veneer does not appear to be attached to the concrete building and would likely fall away from the structure (see brick discussion above).

Several pieces of equipment in the building were not anchored to prevent movement during an earthquake. Boilers number 6521 and 6511 are not anchored to the concrete floor slab (see photos 5 and 6 in Appendix A). Also, the support frame for Tank T-6525 is not anchored to the concrete slab (see photo 7). In either case, the lack of anchorage means that the equipment will move or fall over in an earthquake. If the contents of the boilers or tanks spill, they would represent a hazard to the occupants of the facility.

Primary Clarifier (Facility 6700)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The primary clarifier is a below grade, circular, conventionally reinforced, cast-in-place concrete tank (see photo 9). Associated with the primary clarifier is a parshall flume and a grit basin (see photo 8);

both of these secondary facilities will be demolished during the combined sewer overflow project in the summer of 1998 and as such were not evaluated.

Because the primary clarifier holds liquid, the tank is subjected to the effects of sloshing as described in the digester tank section. In this case, because the tank is below grade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. Historically, belowgrade tanks perform well during an earthquake with little or no damage; therefore, there is little likelihood that the tank would be severely damaged during an earthquake. If the tank was damaged, the damage would be limited, localized cracking and leakage. If cracking occurs, the tank could be quickly repaired using epoxy injection.

Primary Digester (Facility 6100)

Life Safety: Moderate
Health Hazard: Moderate
Economic Loss: Moderate

The primary digester is an abovegrade, conventionally reinforced, cast-in-place circular tank (see photo 10). The roof structure is a cast-in-place concrete roof slab supported by cast-in-place concrete columns cast integrally with the exterior tank walls. The exterior concrete is covered with a brick veneer.

The primary digester is subject to sloshing effects during an earthquake. The sloshing can cause overstress of the tank walls and cracking. If cracking occurred, repair of the tank would be necessary to continue operation. Such repair could include epoxy injection of cracks or local repair and replacement of damaged areas.

There is a wheeled gantry crane stored on the roof of the primary digester (see photo 11). The gantry crane is chained to the handrail surrounding the roof slab; however, there is nothing to prevent the crane from falling over the railing. If the crane must remain on the roof slab, the crane should be anchored to prevent it from moving around or falling off the roof in an earthquake. Chaining the gantry crane to the handrail is not sufficient to prevent movement or to prevent the crane from falling off the structure.

Trickling Filters (Facilities 4010 and 4020)

Life Safety: Moderate
Health Hazard: Moderate
Economic Loss: Moderate

The trickling filters are abovegrade, conventionally reinforced, cast-in-place concrete circular tanks (see photo 12). The tanks are open-topped tanks. The tank has filter media and a center mechanism (see photo 13). The media consists of various sizes of river run rock and sand.

Because of the filter media, the tank is not subjected to the same sloshing effects as typical fluid-holding basins. However, the mass of the filter media in the interior of the tank will move laterally in an earthquake. The movement of the filter media will cause local stress increases in the tank walls. Also, the lateral acceleration of the filter media will cause an overturning movement on the foundation and potential instability of the tank. The localized stress increases would cause cracking of

the tank walls and some leakage of the contents. The cracks could be repaired using epoxy injection or local repair and replacement.

Secondary Clarifier/Chlorine Contact Basin (Facility 4200)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The secondary clarifier/chlorine contact basin is a belowgrade, circular, conventionally reinforced, cast-in-place concrete tank (see photo 14).

Because the secondary clarifier is a liquid-holding structure, the tank is subjected to the effects of sloshing as described in the digester tank section. In this case, because the tank is belowgrade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. Historically, belowgrade tanks perform well during an earthquake with little or no damage; therefore, there is little likelihood that the tank would be severely damaged during an earthquake. If the tank were damaged, the damage would be limited, localized cracking and leakage. If cracking occurs, the tank could be quickly repaired using epoxy injection.

Filter Pump Station

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The filter pump station is a belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic (water-holding) structure (see photo 15). Because it is a hydraulic structure, the pump station will be subject to sloshing loads; however, because of the surrounding soil effects, sloshing should not cause significant damage to the basin. Historically, belowgrade hydraulic structures perform well in an earthquake. Any damage from an earthquake would be minor cracking and leakage. The cracking could be repaired using epoxy injection.

Adjacent to the filter pump station, there are two abovegrade pipes running parallel to each other. The top pipe is supported off of the bottom pipe with a small saddle (see photo 16). This type of support only provides vertical support for the piping. There is potential for the top pipe to move off of the lower pipe during an earthquake because of the lack of lateral restraint.

Secondary Sludge Pump Station (Facility 4300)

Life Safety: Moderate/High
Health Hazard: Low
Economic Risk: Moderate

The secondary sludge pump station is an abovegrade building (see photo 17). The building was constructed in 1967 and was expanded in 1975. The original structure was constructed using concrete masonry unit walls and cast-in-place concrete roof slab. The exterior of the original structure was clad with brick veneer. The addition was constructed using cast-in-place concrete walls and roof slab. The exterior of the new addition was clad with precast concrete panels. During the expansion, the brick

veneer of the original portion of the structure was not removed but was covered with precast concrete panels. It appears that the brick veneer was left in place during the expansion (see photo 18).

The original structure and the addition are both shear wall buildings. Both concrete masonry and cast-in-place concrete shear walls require special detailing to handle earthquake forces, especially high-tension forces that occur at the ends of the shear walls as a result of overturning. In both cases, the wall construction for the original building and the addition was designed for a lower force level than is currently required by the UBC; therefore, the walls could be subjected to higher than designed overturning loads and tension forces. Also, the addition of precast concrete panels over the exterior brick of the original structure has significantly increased the overall mass of the structure, which causes a significant increase in the actual seismic forces that the structure will be subjected to. The net result of the lower design forces and the increase in overall structure mass is a significant increase in seismic demand. The shear walls of the structure could become overstressed and crack significantly during an earthquake.

The connections of the precast panels to the structure must be evaluated. The connections at the original structure were designed for lower force levels than current codes and could fail during an earthquake. Also, the panels must support the brick veneer that was left in place from the original construction. The integrity of the ties attaching the brick veneer should be evaluated. Falling precast panels and brick veneer represent a life safety hazard to anybody near the facility during the event.

The original portion of the building is used as an electrical room (room 4330). Any piece of equipment greater than 400 pounds or any equipment that represents a hazard, such as overhead or wall-mounted equipment, must be anchored to the building slab. The anchorage of the electrical equipment in the original portion of the building should be verified.

Outside of the building is a chemical storage tank (T4360) and concrete containment curb (see photo 32). The tank is anchored directly to the foundation and any contents leaking from the tank would be contained within the structure; therefore, the tank and the containment curb appear adequate for earthquake loads.

Control Gate Structure (Facility 0400)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The control gate structure is a belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic structure. Because it is a hydraulic structure, the structure will be subject to sloshing loads; however, because of the surrounding soil effects, sloshing should not cause significant damage to the basin. Historically, belowgrade, hydraulic structures perform well in an earthquake. Any damage caused by an earthquake would likely be minor cracking and leakage. The cracking could be repaired using epoxy injection.

Headworks (Facility 2500)

Life Safety: Low
Health Hazard: Moderate
Economic Loss: Moderate

The headworks is a partially belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic structure (see photo 19). Because it is a hydraulic structure, the structure will be subject to sloshing loads. Because portions of the structure are abovegrade, those portions would be subject to stress increases caused by sloshing. As such, the upper portions may suffer some damage during an earthquake; however, the damage would likely be minor cracking and leakage. All damage could be repaired using epoxy injection. Leaked contents would represent a health hazard to anyone coming into contact with the spillage.

The anchorage of the bar screens should be evaluated for current seismic force levels, because damage to the bar screens represents a significant economic loss.

Influent Pump Station (Facility 2000)

Life Safety: Moderate
Health Hazard: Moderate
Economic Loss: Moderate

The influent pump station consists of an abovegrade building and a belowgrade pump station (see photo 20). The abovegrade building construction is cast-in-place concrete walls and cast-in-place concrete roof slab and beams; the building shares a common wall with the headworks facility. The belowgrade pump station is a rectangular, conventionally reinforced, cast-in-place concrete facility with half of the facility a dry well providing pumping and the other half a wet well.

The abovegrade building is a concrete shear wall structure with exterior precast panel cladding. The wall construction for the building was designed for a lower force level than that currently dictated by the UBC; therefore, the walls could be subjected to higher than designed overturning loads and tension forces. The shear walls of the structure could become overstressed and crack during an earthquake.

The connections of the precast panels to the structure must be evaluated. The connections for the existing structure were designed for lower force levels than current codes and could fail during an earthquake. Falling precast panels represent a hazard to anybody near the facility during the event.

On the ground level of the building, there is a motor room (room 2400) (see photo 21). There is a piece of newer equipment that is not anchored to the floor slab (see photo 22). It is recommended that the anchorage of all the equipment at ground level be verified. Also, on the roof of the structure is a piece of HVAC equipment (UH 2605) that does not appear to be anchored to the roof slab (see photo 23), and should be anchored.

The belowgrade portion of the influent pump station is a rectangular structure. The wet well portion of the structure that contains water is likely to be subject to sloshing. Those portions of the structure that have soil on the opposite side of the wall will likely see little effect from the sloshing; however, the divider wall between the wet well and the pumps will be subject to localized stress increases. The increases in stress in the divider wall could cause cracking and leakage. If leakage were to occur at the first level belowgrade, the piping tunnels for the plant could become flooded. The spillage of material would be a health hazard for anyone coming into contact with the material.

Primary Sedimentation (Facility 3100)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The primary sedimentation tank is a belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic structure (see photo 24). Because the primary sedimentation tank is a liquid-holding structure, the tank is subjected to the effects of sloshing. In this case, because the tank is belowgrade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. Historically, belowgrade tanks perform well during an earthquake with little or no damage; therefore, there is little likelihood that the tank would be severely damaged during an earthquake. If the tank were damaged, the damage would likely be minor cracking and leakage. If cracking and leakage occurred, the tank could be quickly repaired using epoxy crack injection.

Secondary Clarifiers (Facilities 5300 and 5400)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The secondary clarifiers are belowgrade, circular, conventionally reinforced, cast-in-place concrete tanks (see photo 25).

Because the secondary clarifiers are liquid-holding structures, the tanks are subjected to the effects of sloshing. In this case, because the tanks are belowgrade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. Historically, belowgrade tanks perform well during an earthquake with little or no damage; therefore, there is little likelihood that the tank would be severely damaged during an earthquake. If a tank was damaged, the damage would be limited, localized cracking and leakage. If cracking occurs, the tank could be quickly repaired using epoxy injection.

At the center mechanism of clarifier 5400, there is control equipment that is not anchored to the walkway bridge (see photo 26). This equipment could move or fall over during an earthquake. The equipment should be attached to the walkway bridge.

Adjacent to each clarifier are RAS pumps (P5321, P5322, P5421, and P5422) with a cast-in-place concrete cover structure (see photo 27). The cover has two parallel concrete walls and is open on the other sides. The roof structure is framed with wide flange steel beams framed directly into the two walls supporting a lightweight ribbed decking. There is no direct connection between the roof decking and the concrete walls. The lateral force resisting system for the cover is shear walls in one direction and a moment frame in the other direction. The lack of connection between the walls and the roof decking does not allow transfer of the roof loads directly into the concrete shear walls. The load must be transferred into the walls by out-of-plane bending of the supporting roof beams; however, this is likely not detrimental to the structure as the loads are small due to the relatively lightweight roofing. In the direction of the moment frame, the connections between the concrete walls and steel beams should be evaluated for the higher loads mandated by the current UBC.

The piping in the RAS pump area is supported with a combination of stacked pipe supports and floor-mounted supports (see photo 28). One of the floor-mounted supports is not attached to the concrete slab, and the stacked support provides no resistance against lateral forces. During an earthquake, there is a possibility that the upper piping would come free of the stacked support and potentially damage the piping.

Aeration Tanks (Facility 5100)

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The aeration tanks are a belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic structures (see photo 29).

Because the aeration tanks are a liquid-holding structure, the tanks are subjected to the effects of sloshing. In this case, because the tanks are belowgrade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. There is little likelihood that the tank would be severely damaged during an earthquake. If the tank were damaged, the damage would likely be minor cracking and leakage. If cracking occurs, the tank could be quickly repaired and back in service.

In between the tanks, there is a dry channel containing the air piping (see photo 30). There appears to be no lateral bracing for the air piping; as such, the piping could possibly be damaged during an earthquake.

Adjacent to the aeration tanks is the WAS pump cover (facility 5200) (see photo 31). The cover has two parallel concrete walls and is open on the other sides. The roof structure is framed with wide flange steel beams framed directly into the two walls supporting a lightweight ribbed decking. There is no direct connection between the roof decking and the concrete walls. The lateral force resisting system for the cover is shear walls in one direction and a moment frame in the other direction. The lack of connection between the walls and the roof decking does not allow transfer of the roof loads directly into the concrete shear walls. The load must be transferred into the walls by out-of plane-bending of the supporting roof beams; however, this is likely not detrimental to the structure as the loads are small because of the relatively lightweight roofing. In the direction of the moment frame, the connections between the concrete walls and steel beams should be evaluated for the higher loads mandated by the current UBC.

Effluent Pump Station (Facility 0300)

Life Safety: Moderate
Health Hazard: Low
Economic Loss: Moderate

The effluent pump station is a belowgrade, conventionally reinforced, cast-in-place concrete, rectangular hydraulic structure. There is also an abovegrade cover over the pumps (see photo 33).

Because the effluent pump station is a liquid-holding structure, the structure is subjected to the effects of sloshing. In this case, because the structure is belowgrade, the increase in pressure caused by

sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. There is little likelihood that the tank would be severely damaged during an earthquake. If the tank were damaged, the damage would likely be minor cracking and leakage. If cracking occurs, the tank could be quickly repaired.

The abovegrade cover has two parallel concrete walls and is open on the other sides. The roof structure is framed with wide flange steel beams framed directly into the two walls supporting a lightweight ribbed decking (see photo 34). There is no direct connection between the roof decking and the concrete walls. The lateral force resisting system for the cover is shear walls in one direction and a moment frame in the other direction. The lack of connection between the walls and the roof decking does not allow transfer of the roof loads directly into the concrete shear walls. The load must be transferred into the walls by out-of-plane bending of the supporting roof beams; however, this is likely not detrimental to the structure as the loads are small because of the relatively lightweight roofing. In the direction of the moment frame, the connections between the concrete walls and steel beams should be evaluated for the higher loads mandated by the current UBC.

On the top slab of the pump station, there is a piping support that is not anchored to the concrete slab (see photo 35). If this support were to tip over in an earthquake, the piping could potentially be damaged.

Adjacent to the effluent pump station is a transformer pad (see photo 36). The transformers (TFR102, TFR103, TFR104, and TFR105) are not anchored to the slab (see photo 37). The lack of anchorage will allow the transformers to move and possibly tip in an earthquake. Movement or tipping of the transformers is a threat to life safety for anyone in the area.

Outfall Diversion Structure

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The outfall diversion structure is a belowgrade, conventionally reinforced, cast-in-place concrete rectangular hydraulic structure

Because the diversion structure is a liquid-holding structure, the structure is subjected to the effects of sloshing. In this case, because the tanks are belowgrade, the increase in pressure caused by sloshing is offset by the additional restraint provided by the soil adjacent to the tank walls. There is little likelihood that the tank would be damaged during an earthquake. If the tank were damaged, the damage would likely be minor cracking and leakage. If cracking occurs, the tank could be quickly repaired.

Tunnels

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

The tunnels are belowgrade, cast-in-place concrete tunnels. Plant piping is run through the tunnels on a system of pipe racks. The majority of piping is laterally supported against movement in an

earthquake; however, a couple of piping runs are not laterally supported. The heat piping and the circulated sludge piping in the tunnels are not strapped to prevent lateral movement and could be damaged in an earthquake (see photos 38 and 39). Also, there are storage racks in the tunnels that are not anchored to the walls (see photo 40). In an earthquake, the shelving could tip or the contents could fall. Falling debris represents a life safety hazard to anyone in the area at the time of the earthquake.

Miscellaneous Site Structures

Diesel Fuel Building

Life Safety: High
Health Hazard: Moderate
Economic Loss: High

To the north of the effluent pump station, there is a small prefabricated metal building housing a diesel fuel tank (see photo 41). The building in anchored to curbs that form a containment area around the tank. The tank is not anchored to the concrete slab supporting the tank (see photo 42). Lack of tank anchorage will allow the tank to move and potentially tip over in an earthquake. Movement of the tank and spillage of the fuel is a direct threat to life safety if the spilled contents were to ignite or the tank to explode. Also, the building is very near the Operations and Maintenance building; any fire or explosion caused by tank support failure could possibly damage this building.

Garage Structure

Life Safety: Low
Health Hazard: Low
Economic Loss: Low

To the north of the effluent pump station and next to the diesel fuel building, there is a small, prefabricated metal garage (see photo 43). At the time of the visual inspection the garage was unoccupied. The building is anchored to the foundation structure, and represents little seismic threat.

Operations and Maintenance Building (Facility 1000)

Life Safety: High
Health Hazard: High
Economic Loss: High

The Operations and Maintenance building is a one-story building complex. The complex is comprised of two independent buildings joined together by a breezeway (see photos 44, 45, and 46). The eastern building portion houses the operations and laboratory. The western building portion houses chlorine storage, a maintenance bay, a blower room, and an electrical/control room. Both buildings are a combination of cast-in-place bearing walls and cast-in-place and/or precast concrete columns. The roof structure is cast-in-place concrete beams and precast concrete double tee roof beams. The exterior of the structure is clad with precast concrete panels. Interior partition walls are non-bearing concrete masonry unit walls. The lateral system for each portion of the structure is the cast-in-place concrete walls acting as shear walls. The columns and beams supporting the roof structure are not part

of the lateral force resisting system. The interior masonry partitions also are not part of the lateral force resisting system.

The Operations and Maintenance building was designed to a lower seismic force level than the current UBC requires. As such, the concrete shear walls will be subjected to loads greater than was anticipated during design. Also, the possibility for the shear walls to overturn will increase because of the increased load demand. Depending on the level of overstress in the walls, the concrete could crack and there could be localized failure of the walls. The columns that support the roof beams, though not part of the lateral force system, must also be able to handle the movements of the structure during an earthquake. The columns should be designed with ductility (flexibility) in order to resist the seismic movement. Ductility is provided by confining the column concrete with tightly spaced ties, especially at the top and bottom of the column. The original design drawings do not show tie spacings that would provide the necessary ductility for the columns. The columns, therefore, are not able to move laterally in an earthquake and remain ductile. There is a possibility for column damage and potential column failure during an earthquake. The columns can be retrofitted and reinforced externally to provide ductility.

The roof structure for each portion of the complex is constructed using cast-in-place concrete beams supporting precast concrete double tees. The double tees must act monolithically to form a diaphragm to transmit earthquake forces to the building shear walls. The original design drawings indicate that the double tees are all connected to form a diaphragm; however, the force levels used to design the connections are lower than those currently required by the UBC. As such, the connections should be evaluated to determine if there is enough capacity to resist the new, higher earthquake diaphragm forces. If the connections were to fail during an earthquake, the roof structure would no longer act as a continuous diaphragm. If the diaphragm was compromised during an earthquake, all the seismic loading would not be transferred to the shear walls and the force not transferred to the shear walls would be transmitted to elements of the structure not designed to resist earthquake forces. If overloaded, these elements could become damaged and fail.

Along with the construction of the buildings, the geometry of the building could have an adverse affect on the performance of the structure. The building, in plan, is laid out in an "L" shape. The operations portion of the building is separated from the maintenance portion of the building by a one-inch gap. The two portions of the building might move independently during an earthquake. Depending on the response of each portion of the structure, the two building portions would likely not move in unison; therefore, there is a likelihood that there would be pounding between the two building portions at the separation. A one-inch gap does not appear to provide sufficient separation to prevent this pounding. As such, there will likely be localized damage at the joint when the two buildings come into contact during the earthquake. Because the roof structures of the building portions are in the closest proximity, the damage will likely be limited to localized damage of the roofs of each building portion.

There are also several non-building hazards related to the maintenance portion of the building. In the chlorine room (room 1200), the chlorine storage cylinders sit in a cradle on the floor (see photos 47 through 49). The cradles only prevent movement of the cylinders laterally in one direction. There is no restraint for the cylinders moving laterally along their long axis. Obviously, if there is movement of the cylinders during an earthquake, there is the potential for the cylinders to rupture or for the chlorine piping to disengage from the tanks and leak chlorine gas. A chlorine gas release directly endangers the occupants of the facility and nearby residents. It should be noted that the gaseous chlorine system is to be removed as part of the Combined Sewer Overflow improvement in the summer of 1998. Also in the chlorine room, there is an HVAC unit (UH1225) that is not braced

laterally (see photo 50). This piece of equipment could fall during an earthquake because there is no lateral restraint to resist seismic forces.

In the maintenance bay (room 1300), there is an HVAC unit (UH1350) that is not braced laterally (see photo 51). Again, there is a possibility of this equipment falling during an earthquake. In the maintenance bay, there are materials stored above an interior office (see photo 52). The materials stored above the office area appear to be very light, but any items stored above the office could possibly fall off in an earthquake. Any item stored above the office should be stored in such a way as to prevent falling in an earthquake.

In the blower room (room 1100), the blowers (B1111, B1112, B1113, and B1114) are attached to the concrete equipment pads with steel anchor bolts; however, there are no nuts on any of the bolts (see photos 53, 54 and 55). The blowers could come free of the anchor bolts and move during an earthquake. Nuts should be added to the anchor bolts. The vertical blower piping and horizontal ducting is not braced laterally (see photo 56). The lack of lateral bracing will allow the piping and ducting to move horizontally during an earthquake and potentially fall. Along the north wall of the blower room is a series of storage racks (see photo 57). The racks have heavy items stored on the shelves, and are not anchored to the wall. The racks could tip during an earthquake.

In the electrical/control room (room 1100, see photos 58 and 59), the anchorage of all the free-standing electrical and control equipment should be verified. Several pieces of equipment do not appear to be anchored to the concrete slab. In particular, the blower control panel and the fiber optic control panel are not anchored to the concrete slab (see photos 60 and 61). Obviously, lack of anchorage could allow the equipment to tip in an earthquake, possibly damaging the equipment or interrupting operation.

Section 5

PRIORITIZATION

The following section ranks each building and process facility relative to every other building and process facility in terms of its seismic vulnerability. The major factor involved in ranking the facilities is life safety. Those facilities that have a high vulnerability in the life safety category are the most susceptible in an earthquake and represent an immediate threat to human life. No cost estimate was performed to determine the relative cost for upgrading or retrofitting the facilities. A cost estimate could be done in the next phase of work, which should also include calculations and the preliminary upgrade design

Table 5-1 ranks each facility from high to low in terms of our engineering opinion of their seismic vulnerability. The ranking is based on the findings of the visual inspection and drawing review.

TABLE 5-1
Seismic Prioritization of Buildings and Process Facilities

Rank	Facility No.	Facility	Life Safety Risk	Health Hazard Risk	Economic Loss Risk
1	1000	Operations and Maintenance	High	High	High
2		Diesel Fuel Building	High	Moderate	High
3	6400	Digester Control Building	High	Low	High
4	4300	Secondary Sludge Pump Station	Moderate/High	Low	Moderate
5	6200,6300	Digester Tanks	Moderate	Moderate	Moderate
6	6100	Primary Digester	Moderate	Moderate	Moderate
6	4010,4020	Trickling Filters	Moderate	Moderate	Moderate
7	2000	Influent Pump Station	Moderate	Moderate	Moderate
8	0300	Effluent Treatment Facility	Moderate	Low	Moderate
9	2500	Headworks	Low	Moderate	Moderate
10	5300,5400	Secondary Clarifiers	Low	Low	Low
11	6700	Primary Clarifier	Low	Low	Low
11	4200	Secondary Clarifier/Chlorine Contact Basin	Low	Low	Low
11		Filter Pump Station	Low	Low	Low

TABLE 5-1
Seismic Prioritization of Buildings and Process Facilities

Rank	Facility No.	Facility	Life Safety Risk	Health Hazard Risk	Economic Loss Risk
11	0400	Control Gate Structure	Low	Low	Low
11	3100	Primary Sedimentation	Low	Low	Low
11	5100	Aeration Tanks	Low	Low	Low
10		Outfall Diversion	Low	Low	Low
10		Tunnels	Low	Low	Low
11		Garage	Low	Low	Low

Section 6

RECOMMENDATIONS

This section presents our recommendations for corrective actions or further analysis for each of the at-risk buildings or process facilities at the Corvallis WWRP. This section is meant to serve as a starting point for any future upgrades or retrofits. The facilities are listed in order of importance from high risk to low risk. No attempt was made to rank the structures based on cost of upgrade or replacement; therefore, it is recommended that prior to proceeding with any of the following corrective actions, a cost estimate for design and construction should be performed . The cost estimate should be based on further analysis and code requirements.

Operations and Maintenance (Facility 1000)

Item: Lateral Force System

Action: A complete analysis of the structural lateral forces resisting system should be performed at the Building Department-mandated code force levels. Particular attention should be paid to those columns and roof beams that are not part of the lateral force system but must deflect with the building in an earthquake. Also, the separation of the building segments should be verified as sufficient to handle expected lateral displacement.

Item: Precast Panel Connections

Action: The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Item: Chlorine Tank Storage

Action: Chlorine storage tanks should be anchored or stored in a manner that prevents any horizontal movement during an earthquake. This corrective action should be implemented immediately because the presence of chlorine poses the greatest direct risk to life safety at the plant. This action is necessary only until the removal of the gaseous chlorine system during the Combined Sewer Overflow improvements in the summer of 1998.

Item: HVAC Units (UH1225)

Action: All hanging HVAC units shall be braced laterally to prevent horizontal movement.

Item: Blowers and Piping (B1111, B1112, B1113, and B1114)

Action: Provide nuts on all blower anchor bolts. Laterally brace all blower piping and ductwork to prevent horizontal movement.

Item: Electrical/Control Equipment

Action: Verify anchorage of all electrical and control equipment in excess of 400 pounds or supported overhead or off of walls. Provide anchorage for any equipment that lacks it.

Diesel Fuel Building

Item: Diesel Fuel Tank

Action: Anchor tank to concrete slab to prevent movement and tipping during an earthquake.

Digester Complex

Item: Digester Tanks (Facilities 6200 and 6300)

Action: Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls.

Item: Digester Control Building (Facility 6400)

Action: A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels. Particular attention should be paid to the steel columns and concrete roof beams that are not part of the lateral force system but must deflect with the building in an earthquake.

Item: Brick Veneer

Action: Provide retrofit anchorage of the brick veneer to the existing building and tank walls.

Item: Equipment Anchorage

Action: Provide anchorage for boilers 6521 and 6511 and Tank T-6525. Verify anchorage of all equipment in excess of 400 pounds or that represents a hazard.

Secondary Sludge Pump Station (Facility 4300)

Item: Lateral Force System

Action: A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels.

Item: Precast Panel Connections

Action: The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Item: Equipment Anchorage

Action: Verify anchorage of all equipment in excess of 400 pounds or that represents a hazard

Primary Digester (Facility 6100)

Item: Tank Sloshing

Action: Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls.

Item: Brick Veneer

Action: Provide retrofit anchorage of the brick veneer to the existing building and tank walls.

Item: Gantry Crane

Action: Remove gantry crane from the roof or provide a means to prevent movement or rolling of the crane during an earthquake.

Trickling Filters (Facilities 4010 and 4020)

Item: Tank Walls

Action: Perform structural analysis of the existing tank for the lateral acceleration of the filter media in an earthquake. The analysis should evaluate potential cracking or overloading of the walls.

Influent Pump Station (Facility 2000)

Item: Lateral Force System

Action: A complete analysis of the structural lateral forces resisting system should be performed at the current code-mandated force levels.

Item: Precast Panel Connections

Action: The precast panel connections should be analyzed for adequacy at the current code-mandated force levels.

Item: Electrical/Control Equipment

Action: Verify anchorage of all electrical and control equipment in excess of 400 pounds. Provide anchorage for any equipment that lacks it.

Item: Roof-mounted HVAC Equipment (UH2605)

Action: Verify anchorage of roof-mounted HVAC equipment. Provide anchorage for any equipment that lacks it.

Item: Wet Well Divider Wall

Action: The wet well divider wall should be analyzed for the effects of sloshing. The analysis should evaluate potential cracking or overloading of the divider wall.

Effluent Treatment Facility (Facility 0300)

Item: Electrical Transformers (TFR102, TFR103, TFR104, and TFR105)

Action: Anchor all transformers to the concrete pad.

Item: Pump Cover

Action: Evaluate existing steel moment connections for increased seismic loads.

Item: Pipe Support

Action: Anchor pipe support on top slab to concrete.

Headworks (Facility 2500)

Item: Basin Walls

Action: Perform structural analysis of existing tank for earthquake sloshing effects. The analysis should evaluate potential cracking or overloading of the walls

Item: Bar Screens

Action: Evaluate anchorage of bar screens for increased seismic loads.

Secondary Clarifiers (Facilities 5300 and 5400)

Item: Control Equipment at Center Pier of Clarifier 5400

Action: Anchor all control equipment at center pier of clarifier.

Item: RAS Pump Cover

Action: Evaluate existing steel moment connections for increased seismic loads.

Item: RAS Piping

Action: Modify existing pipe supports to prevent damage to piping during an earthquake.

Filter Pump Station

Item: Large Diameter Piping

Action: Modify existing pipe supports to prevent damage to piping during an earthquake.

Aeration Tanks (Facility 5100)

Item: Air Piping

Action: Provide lateral bracing for air piping.

Item: WAS Pump Cover

Action: Evaluate existing steel moment connections for increased seismic loads.

Tunnels

Item: Heat Piping and Circulated Sludge Piping

Action: Provide strapping to prevent lateral movement of piping during an earthquake.

Section 7

GENERAL CONDITIONS

This report has been prepared for the exclusive use of the City of Corvallis and CH2M HILL for the specific application to the seismic vulnerability assessment of the Corvallis Wastewater Reclamation Plant in Corvallis, Oregon. This evaluation has been performed in accordance with generally accepted engineering practices. No other warranty, express or implied, is made.

The information contained in this report is based on a site investigation and data obtained from previous geotechnical explorations on the plant site that indicate subsurface conditions only at the specific location and time drilled, and only to the depth penetrated. The subsurface conditions described herein do not necessarily reflect strata variations that may exist at the site.

If anything in the nature of the project changes significantly from what is described in this report, the recommendations contained in this report should not be considered applicable unless the changes and conclusions in this report are reviewed and found to be applicable, in writing, by CH2M HILL. CH2M HILL is not responsible for any claims, damages, or liability associated with interpretation or reuse of the subsurface data for engineering analysis without written authorization from CH2M HILL.

References

Converse, Davis and Associates, Inc. 1974-1977. Various Geotechnical investigations for the Corvallis WWRP.

Geomatrix Consultants. January 1995. *Seismic Design Mapping, State of Oregon.* Prepared for Oregon Department of Transportation.

Jacobson, R.S. 1986. Map of Oregon Seismicity. Oregon Department of Geology and Mineral Resources Geologic Map Series GMS-49, Scale 1:100,000.

Madin, I.P. Priest, G.R. Mabey, M.A., Yelin, T.S., and Meier, D. 1993. March 25,1993, Scotts-Mills Earthquake – Western Oregon's Wake-up Call. Oregon Geology, Vol. 55, No. 3, pp. 51-57.

Orr, E.L., Orr W.N., and Baldwin, E.M. 1993. The Geology of Oregon. Kendall/Hunt Publishing Company: Dubuque, Iowa.

Walker, G.W. and N.S. MacLeod. 1991. Geologic Map of Oregon. U.S. Geological Survey, Scale 1:500,000.

Photo 1: Storage Building. To be demolished during CSO construction.

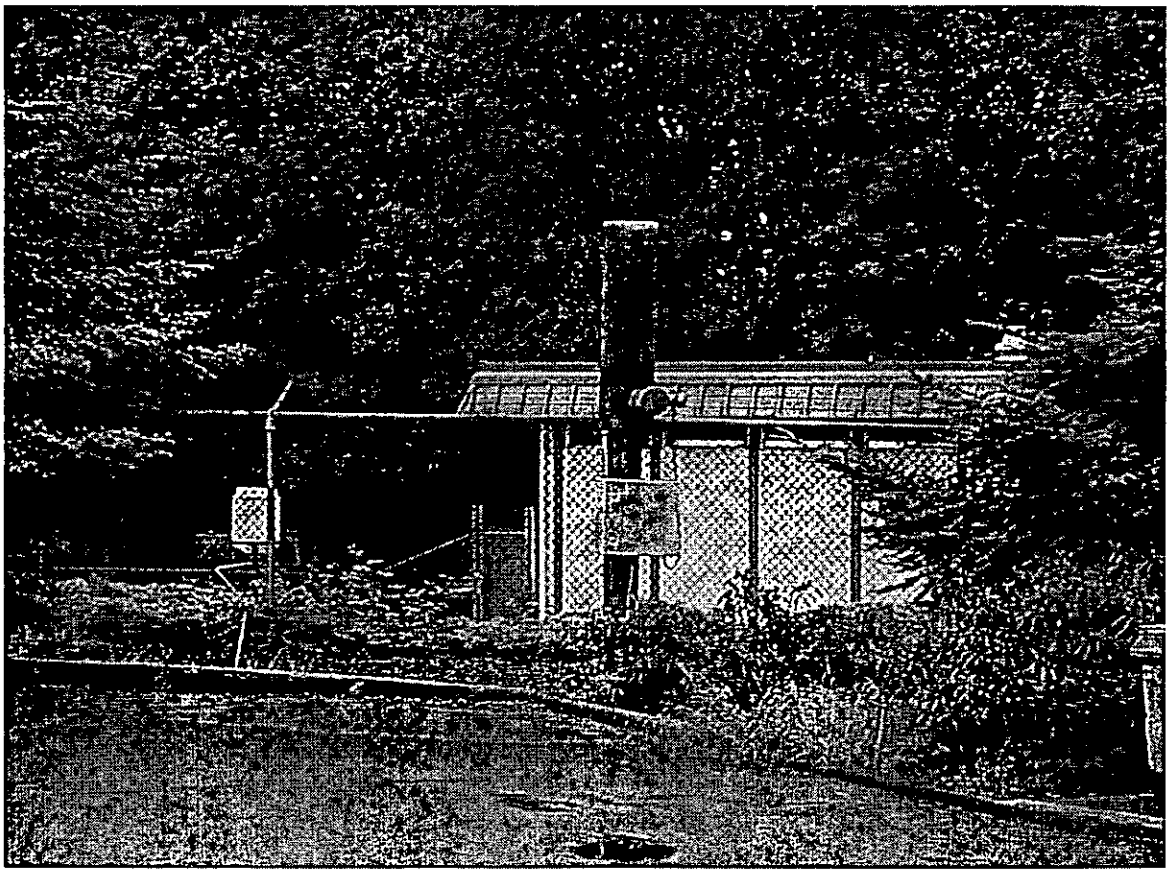

Photo 2: Existing Flair. To be demolished during CSO construction.

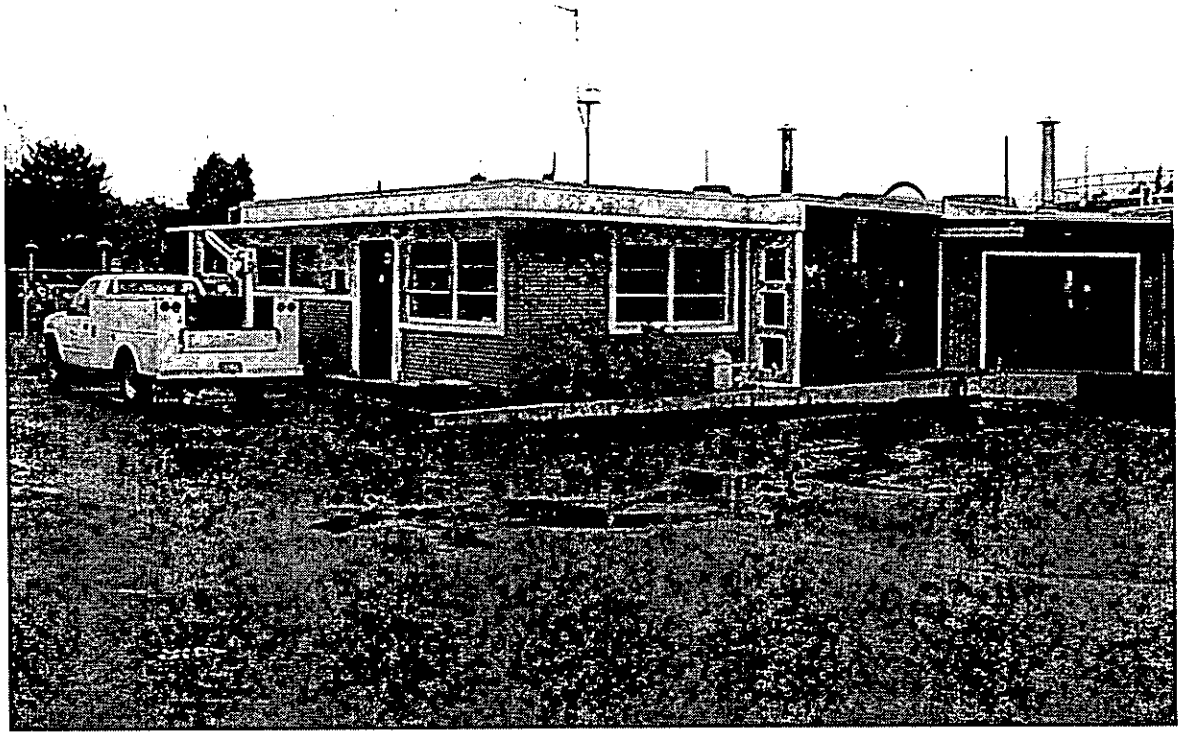

Photo 3: Digester Control Building, Upper Level.

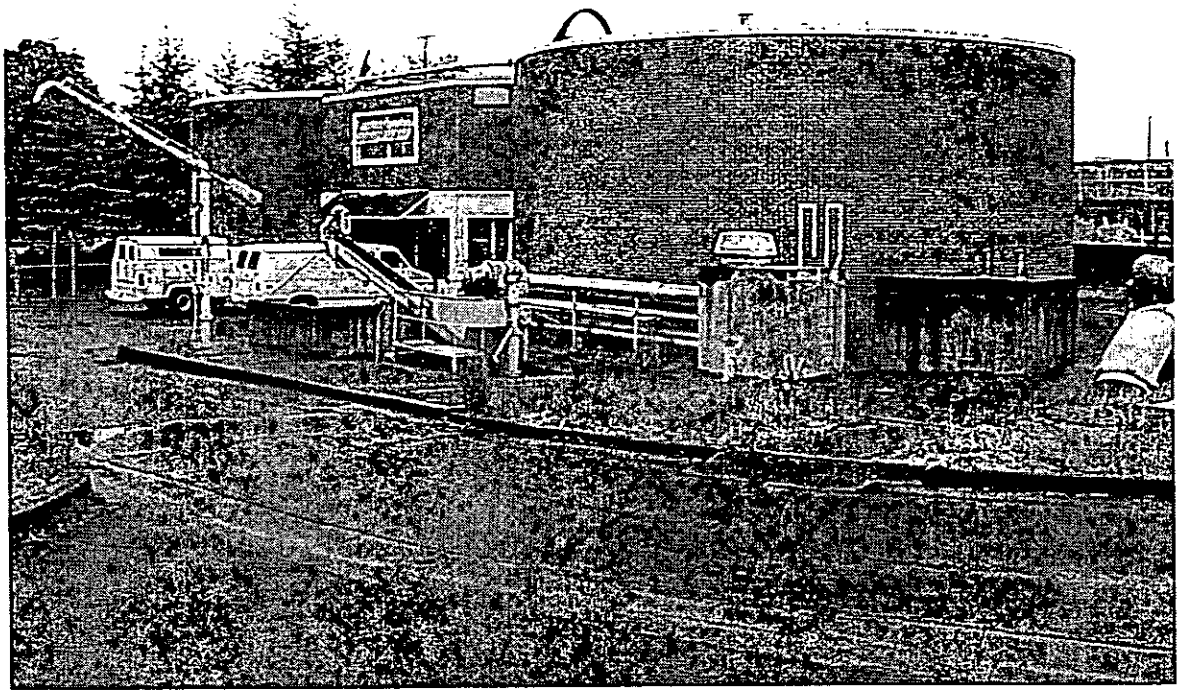

Photo 4: Digester No. 1 and No. 2 and Digester Control Building.

Photo 5: Digester Control Building – Boiler No. 6521. No anchorage to concrete slab.

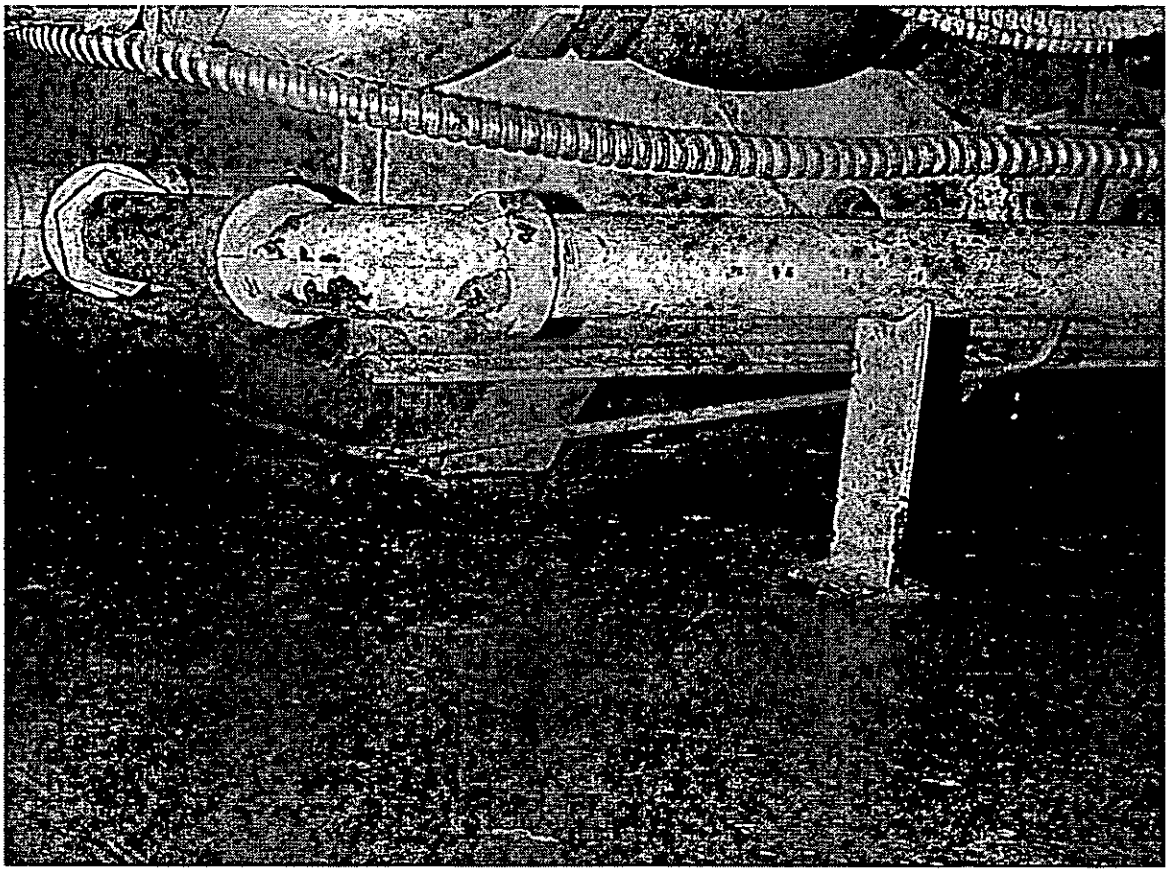

Photo 6: Digester Control Building – Boiler No. 6511. No anchorage to concrete slab.

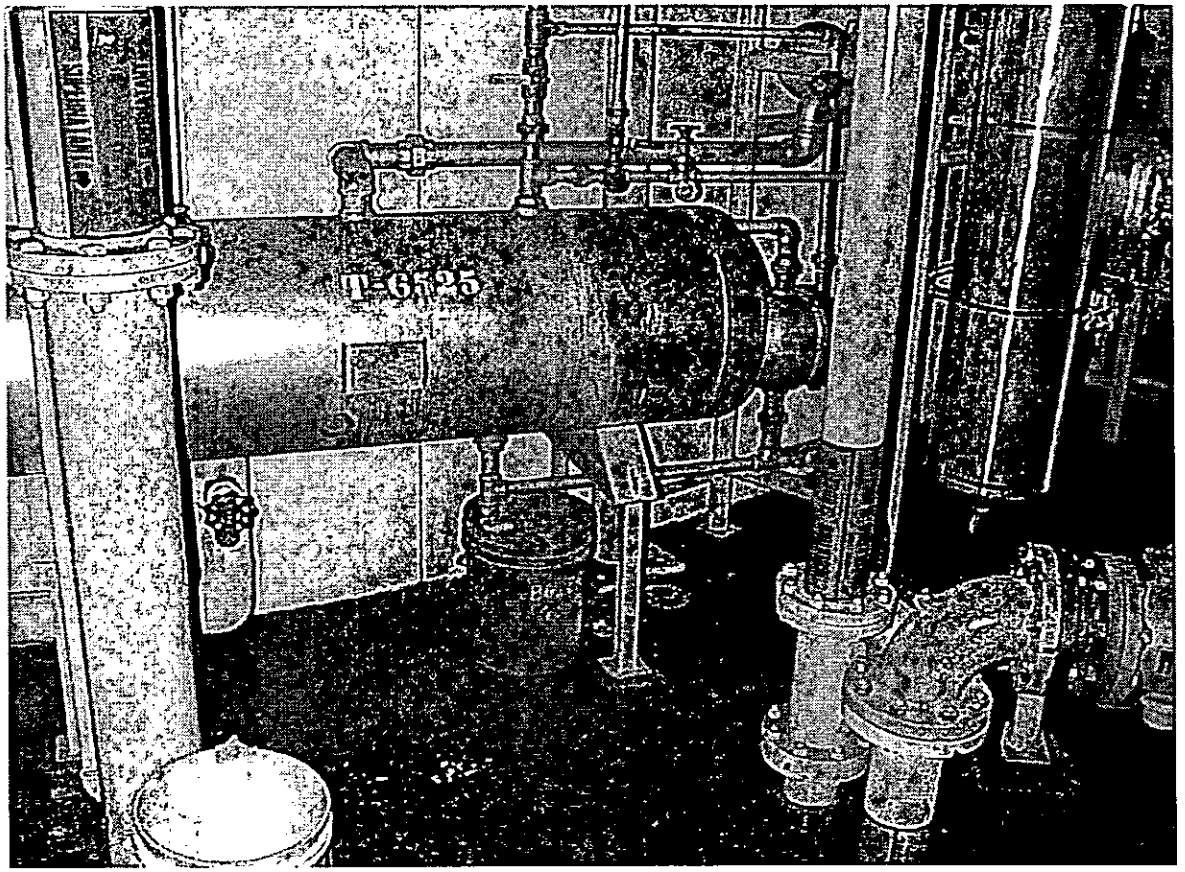

Photo 7: Digester Control Building – Tank T-6525. No anchorage to concrete slab.

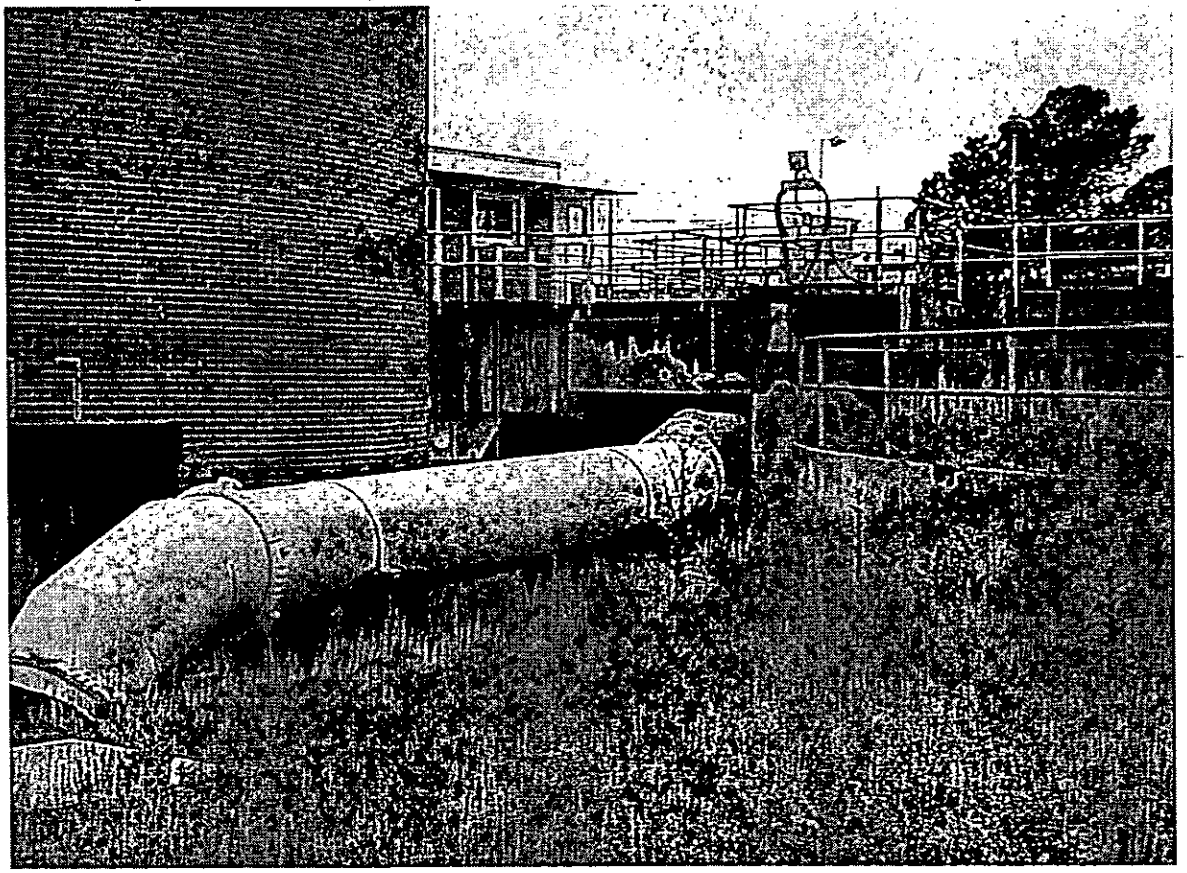

Photo 8: Parshall Flume and Piping Between Digester Control Building and Primary Clarifier. Flume and piping to be removed during CSO construction.

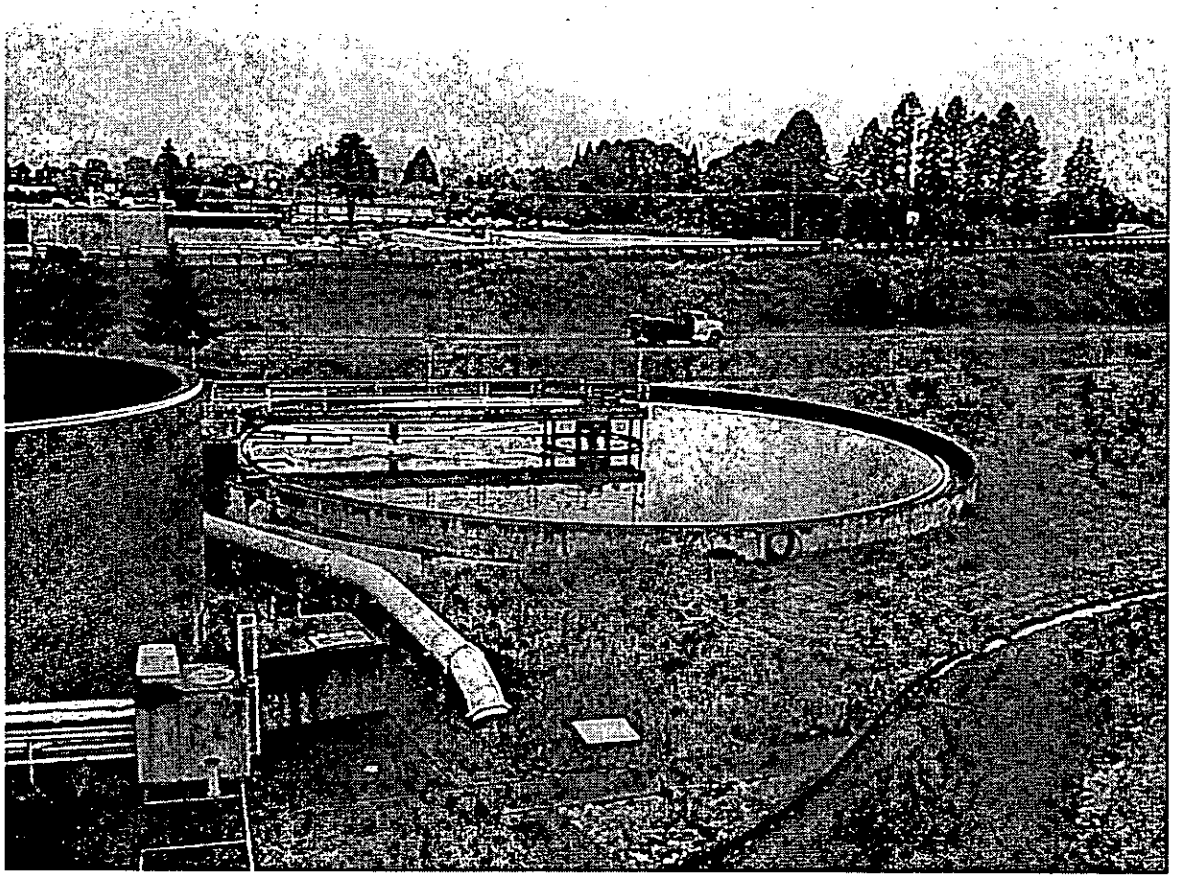

Photo 9: Primary Clarifier.

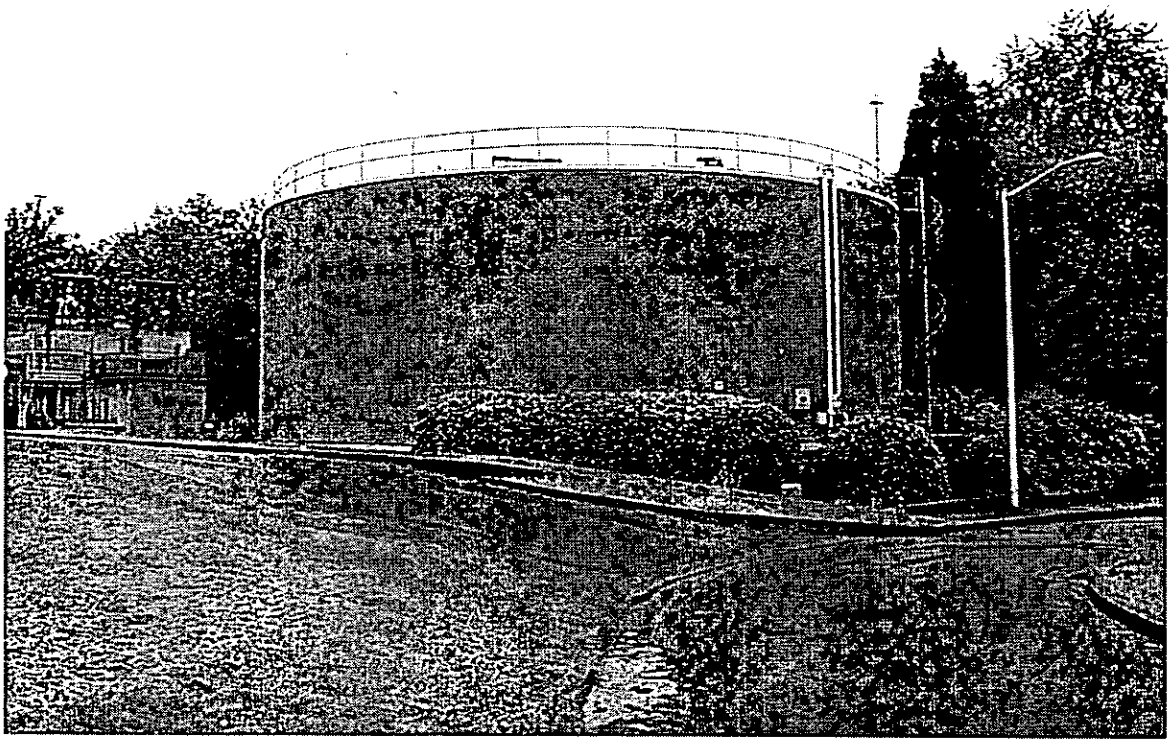

Photo 10: Primary Digester.

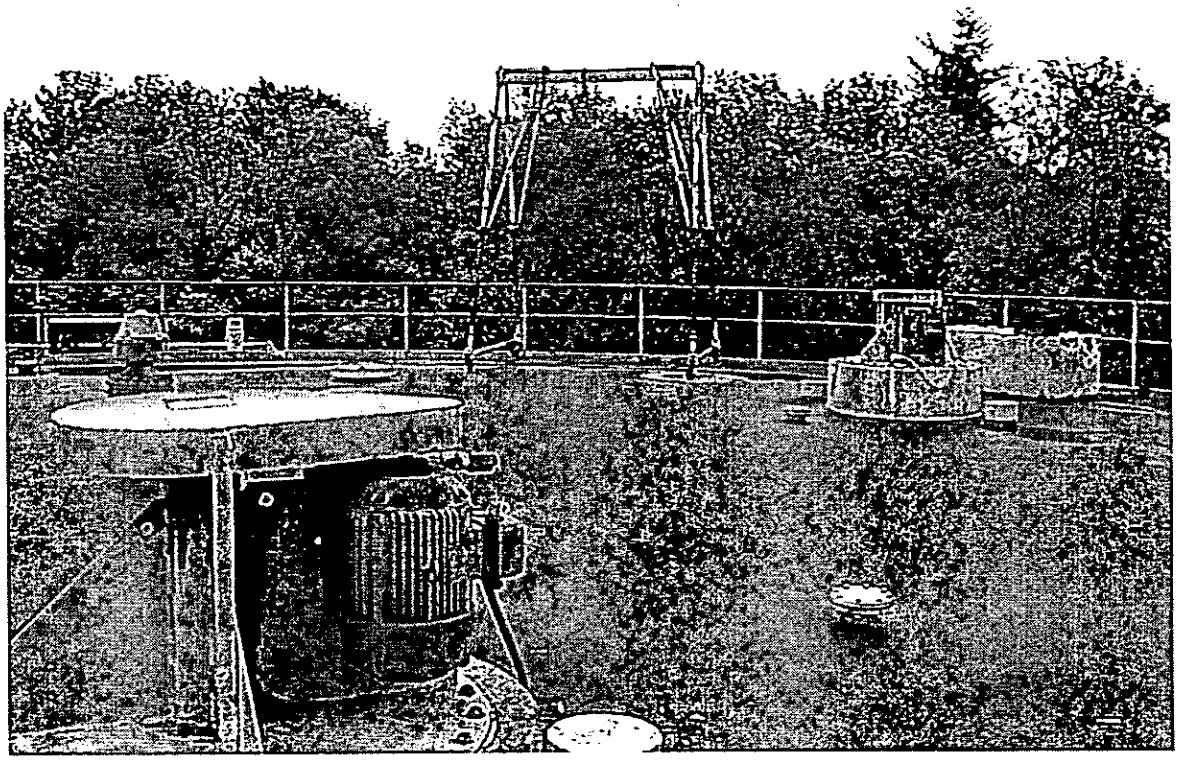

Photo 11: Primary Digester. Gantry Crane on Roof.

Photo 12: Trickling Filters.

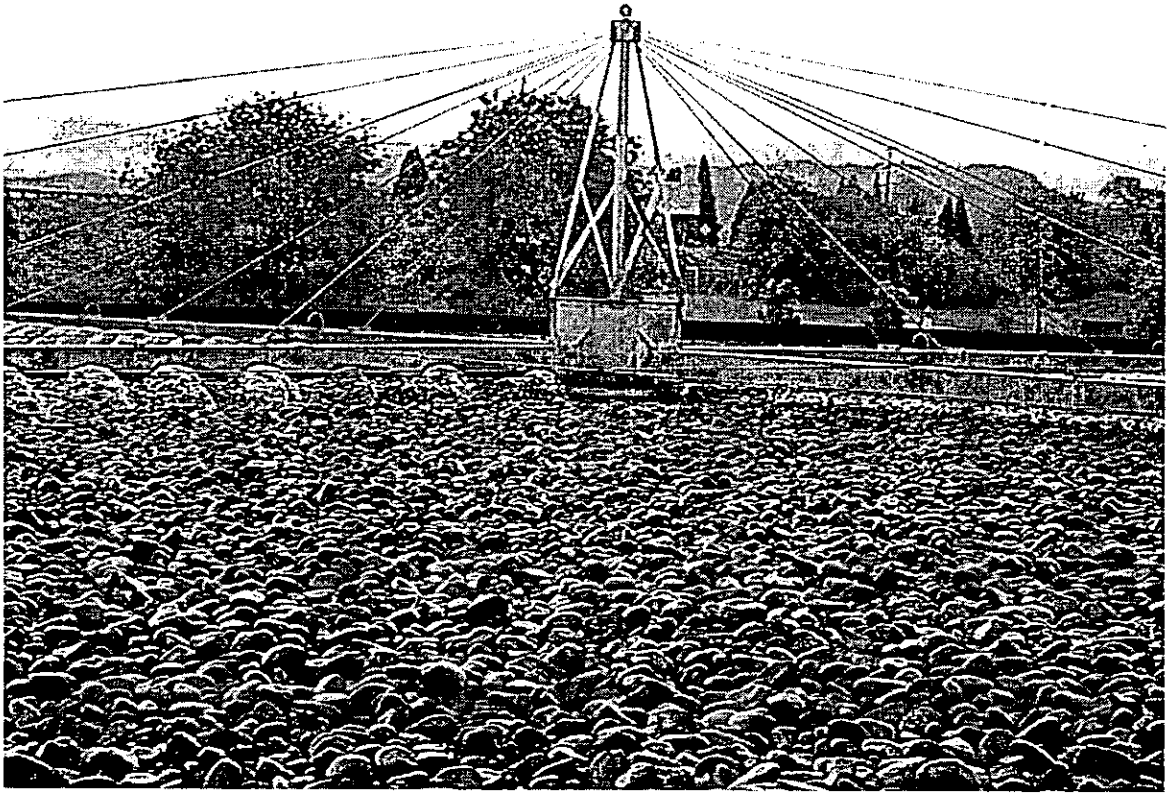

Photo 13: Trickling Filter. Filter Media and Center Mechanism.

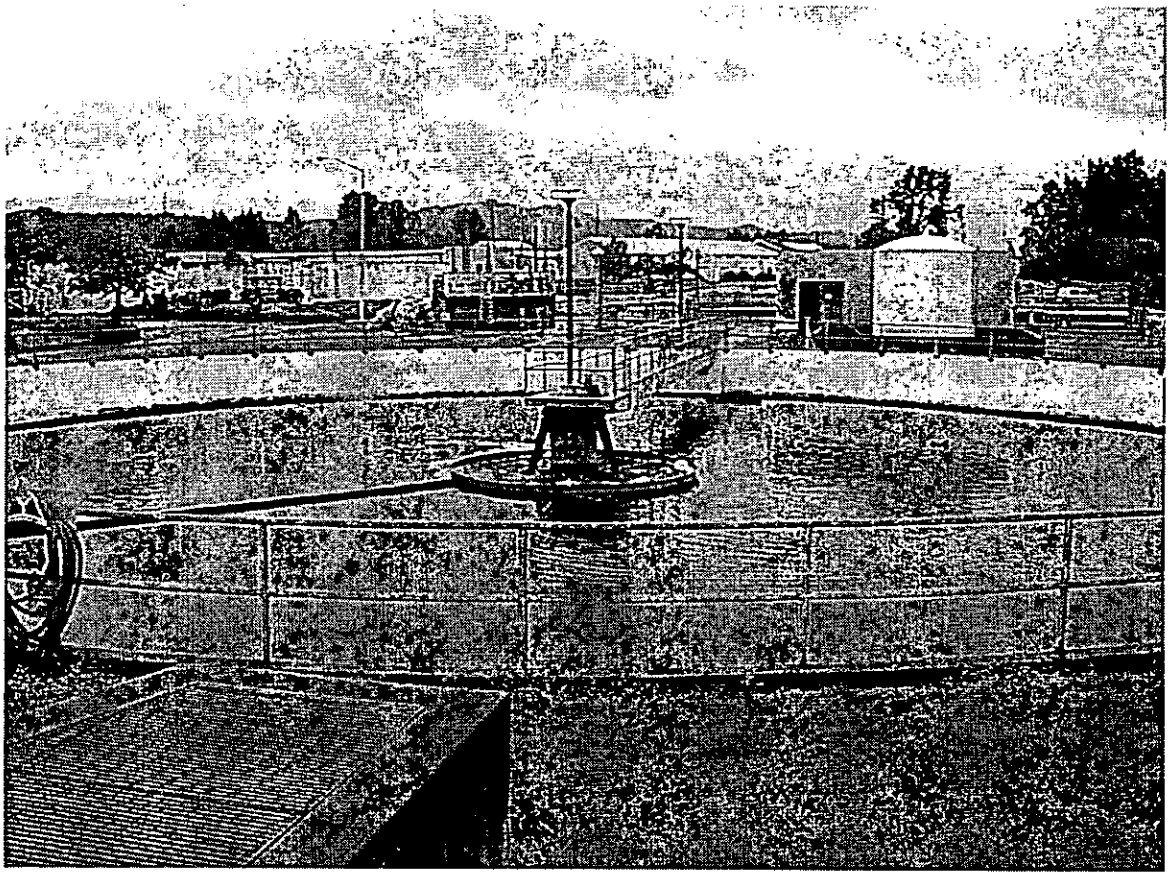

Photo 14: Secondary Clarifier/Chlorine Contact Basin.

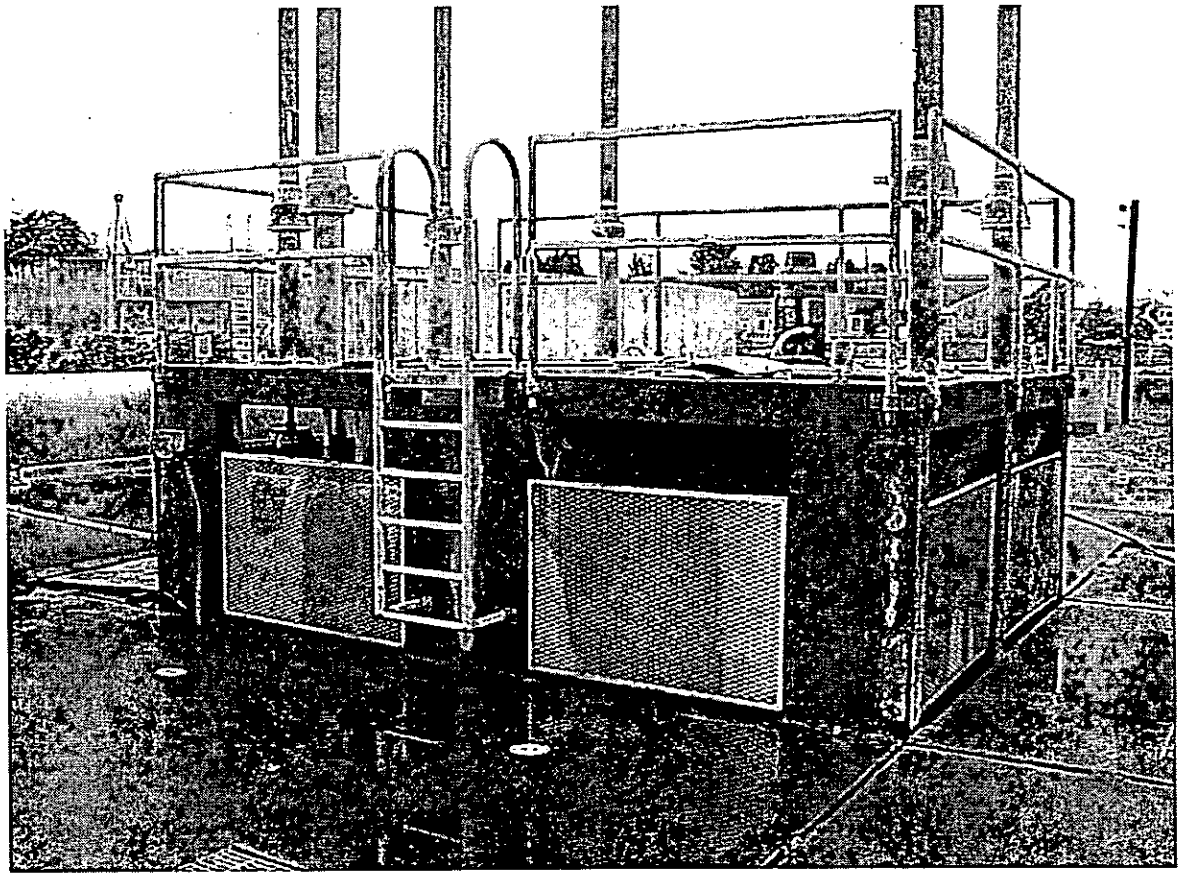

Photo 15: Filter Pump Station.

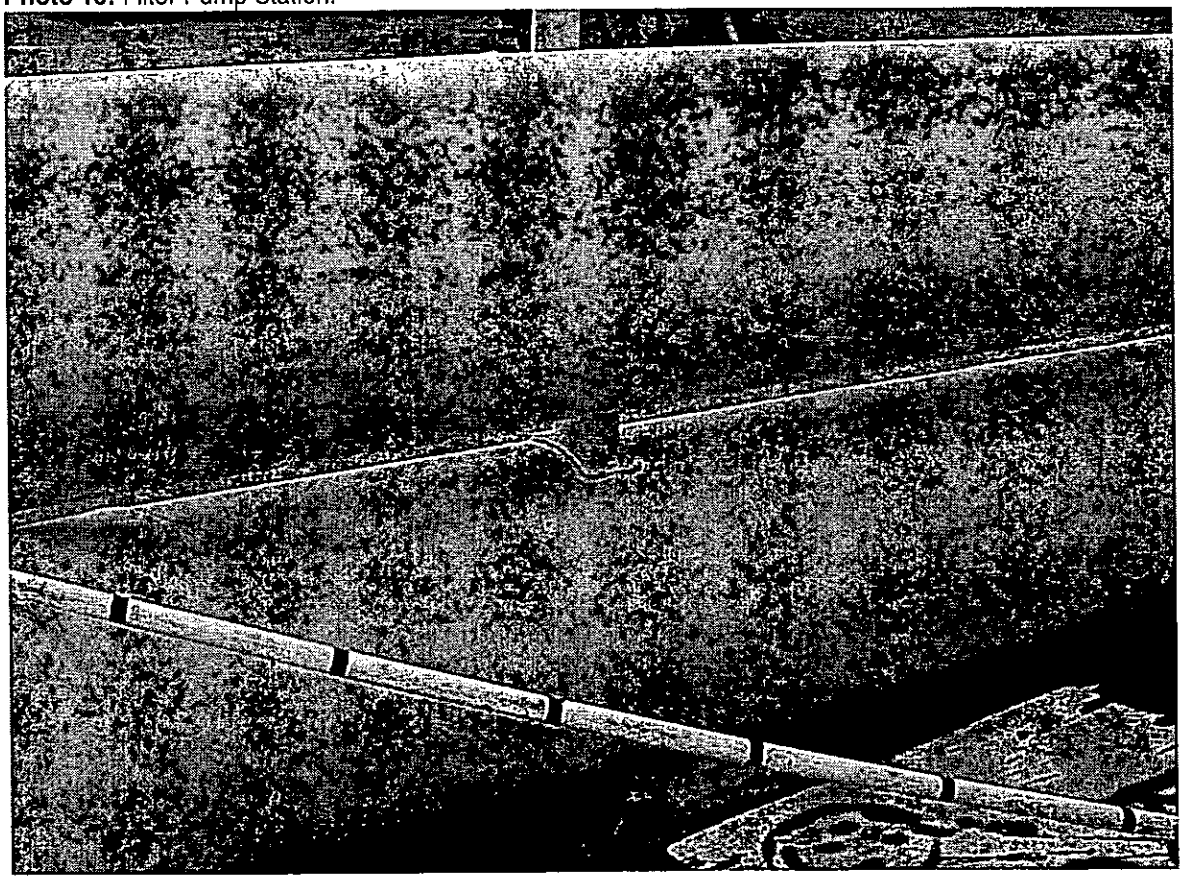

Photo 16: Filter Pump Station. Stacked pipe support.

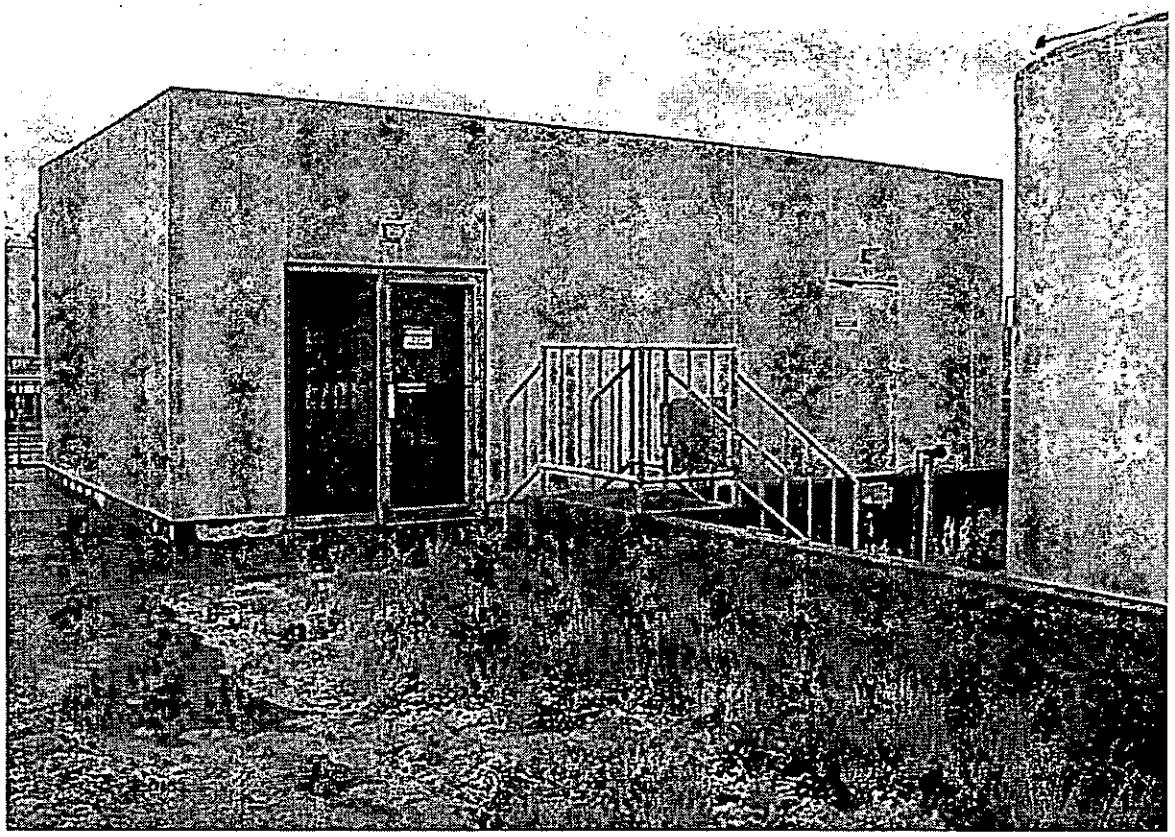

Photo 17: Secondary Sludge Pump Station.

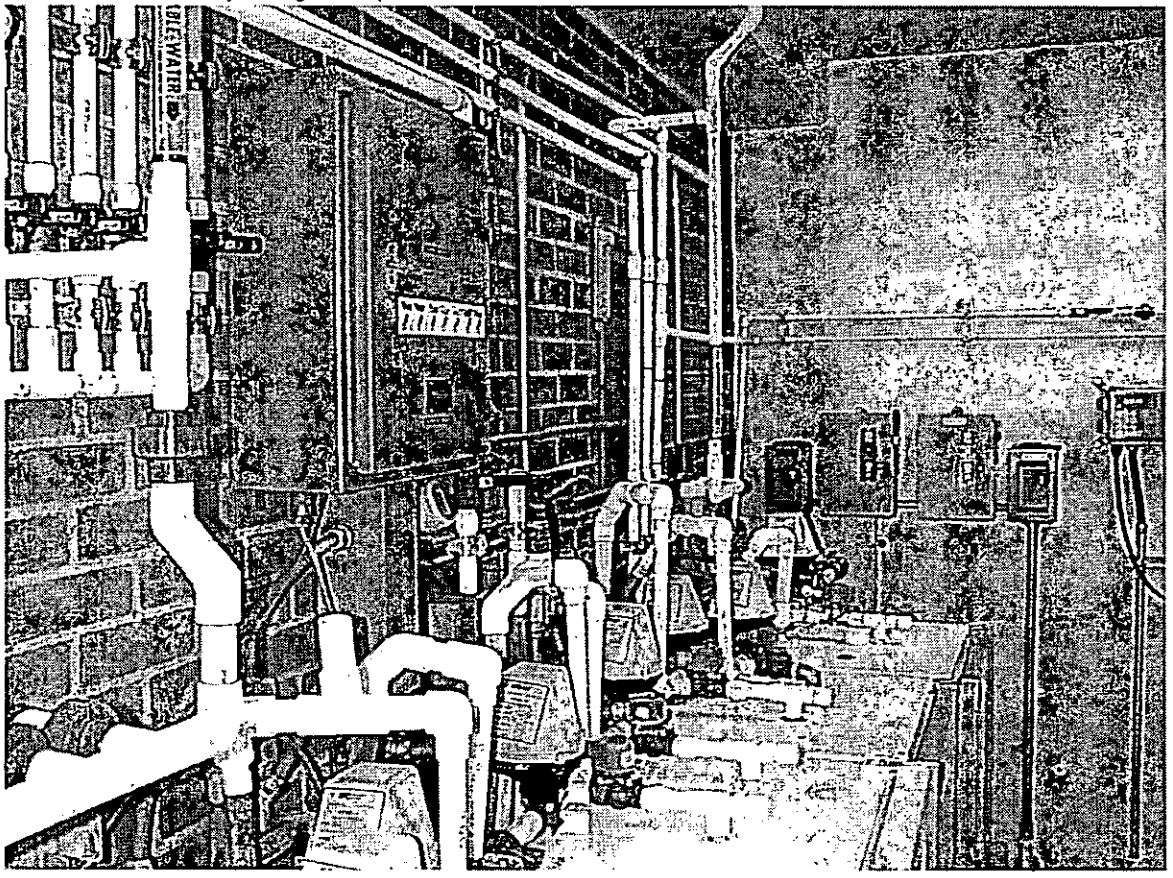

Photo 18: Secondary Sludge Pump Station. 1975 Addition – Notice brick veneer from original construction.

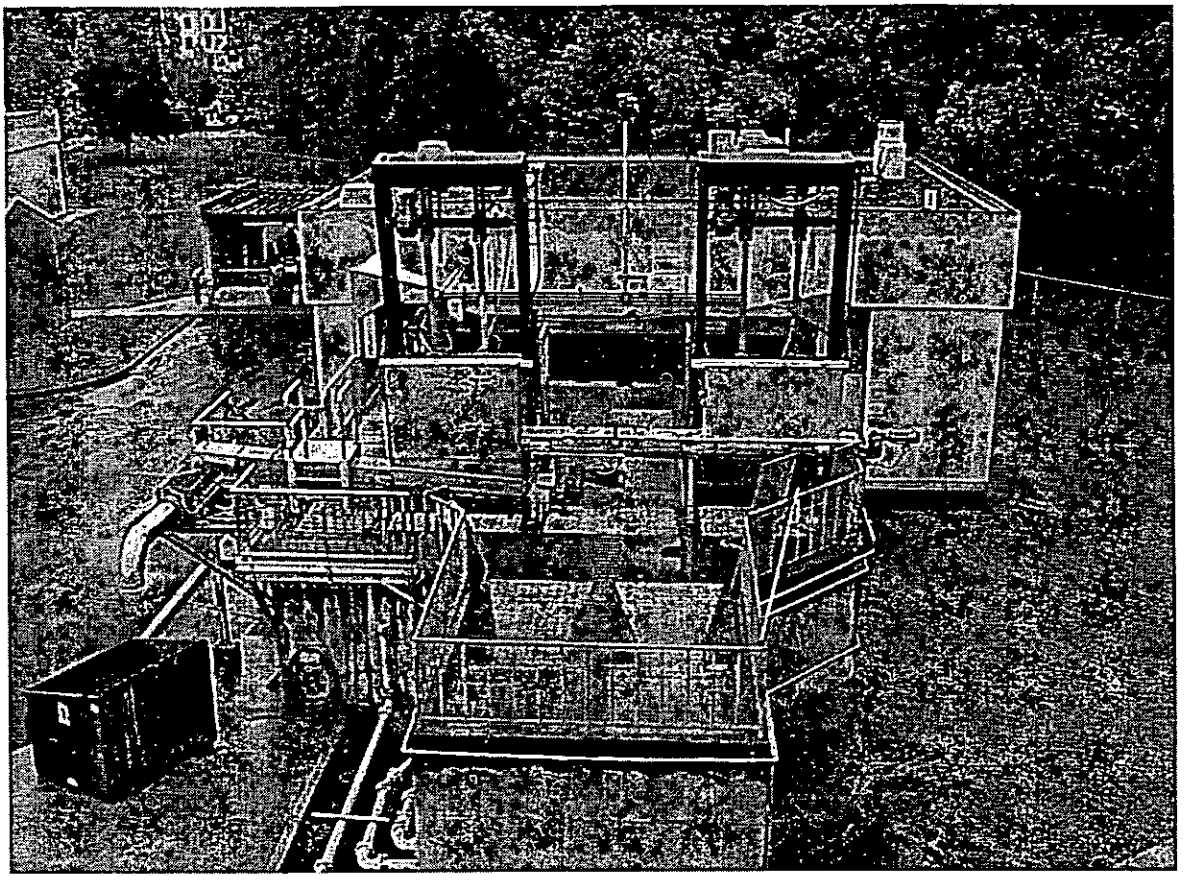

Photo 19: Headworks. Influent Pump Station in background.

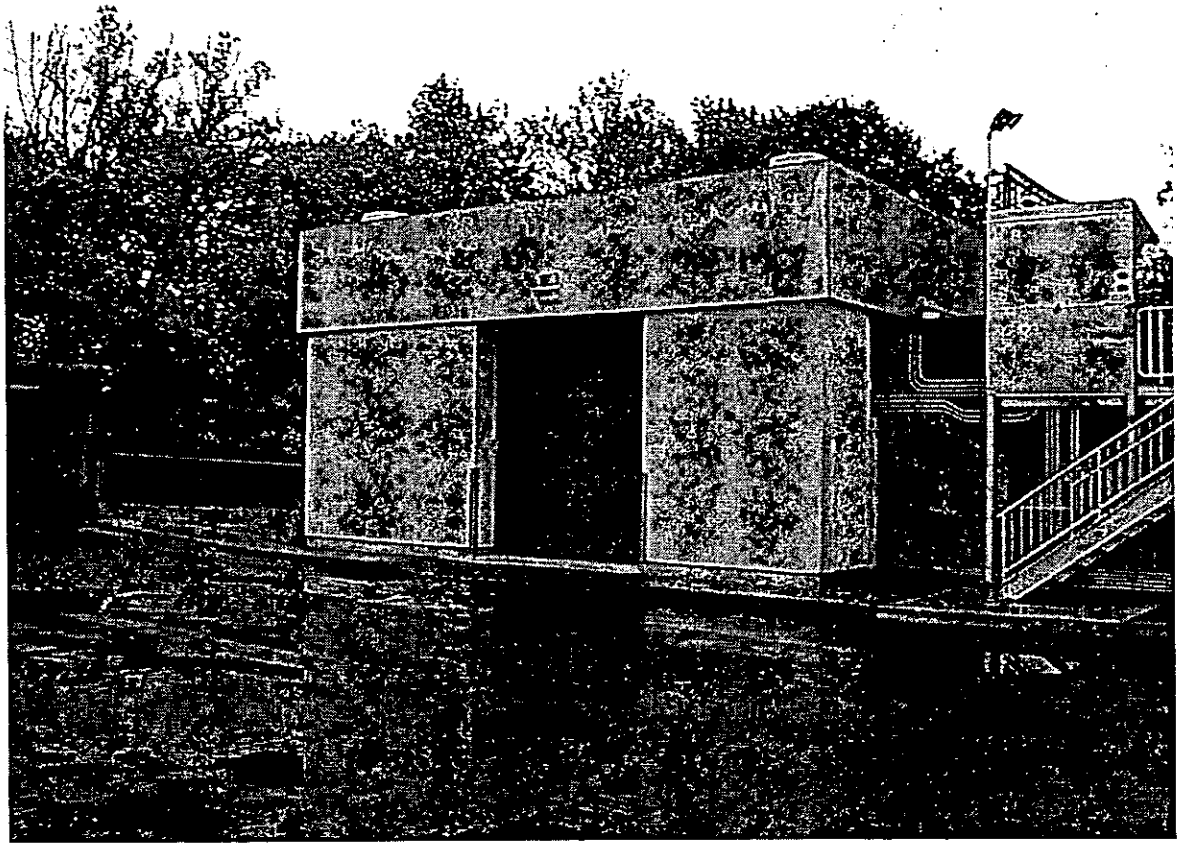

Photo 20: Influent Pump Station.

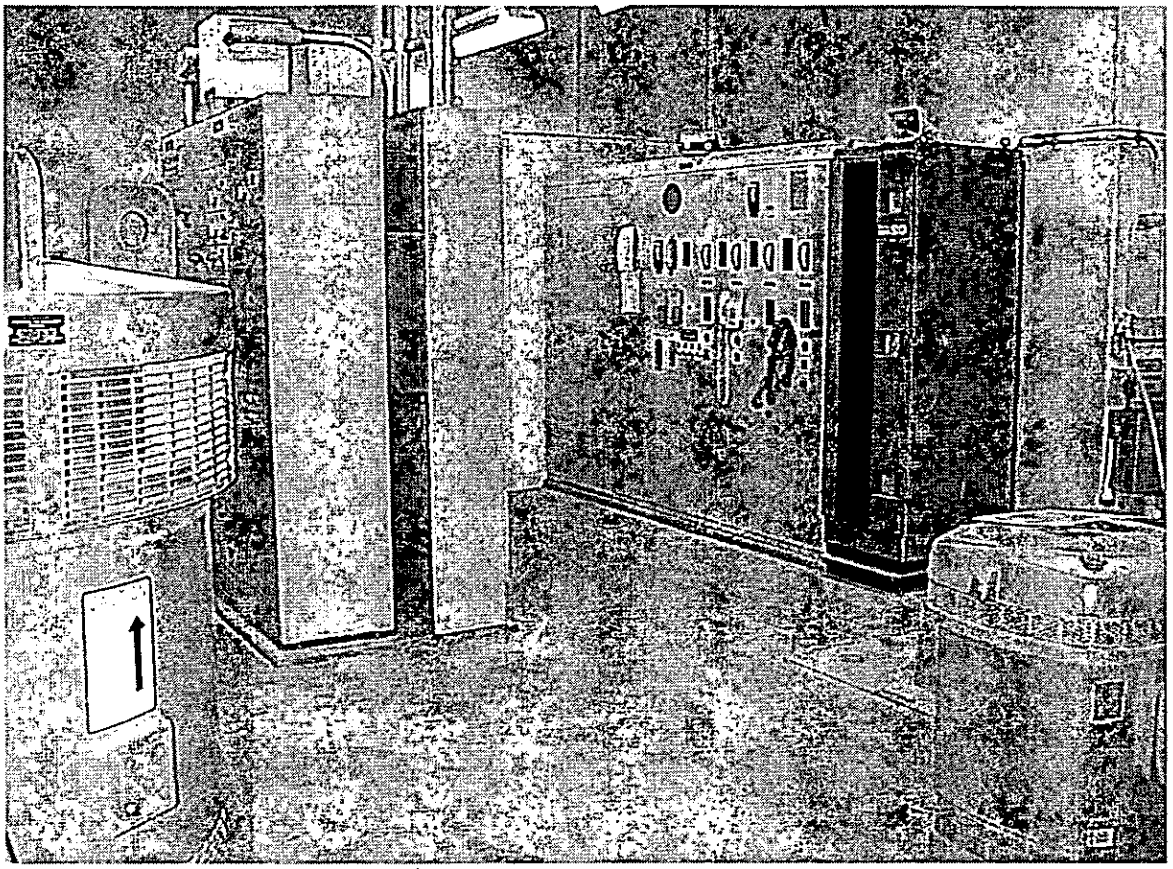

Photo 21: Influent Pump Station. MCCs at ground level.

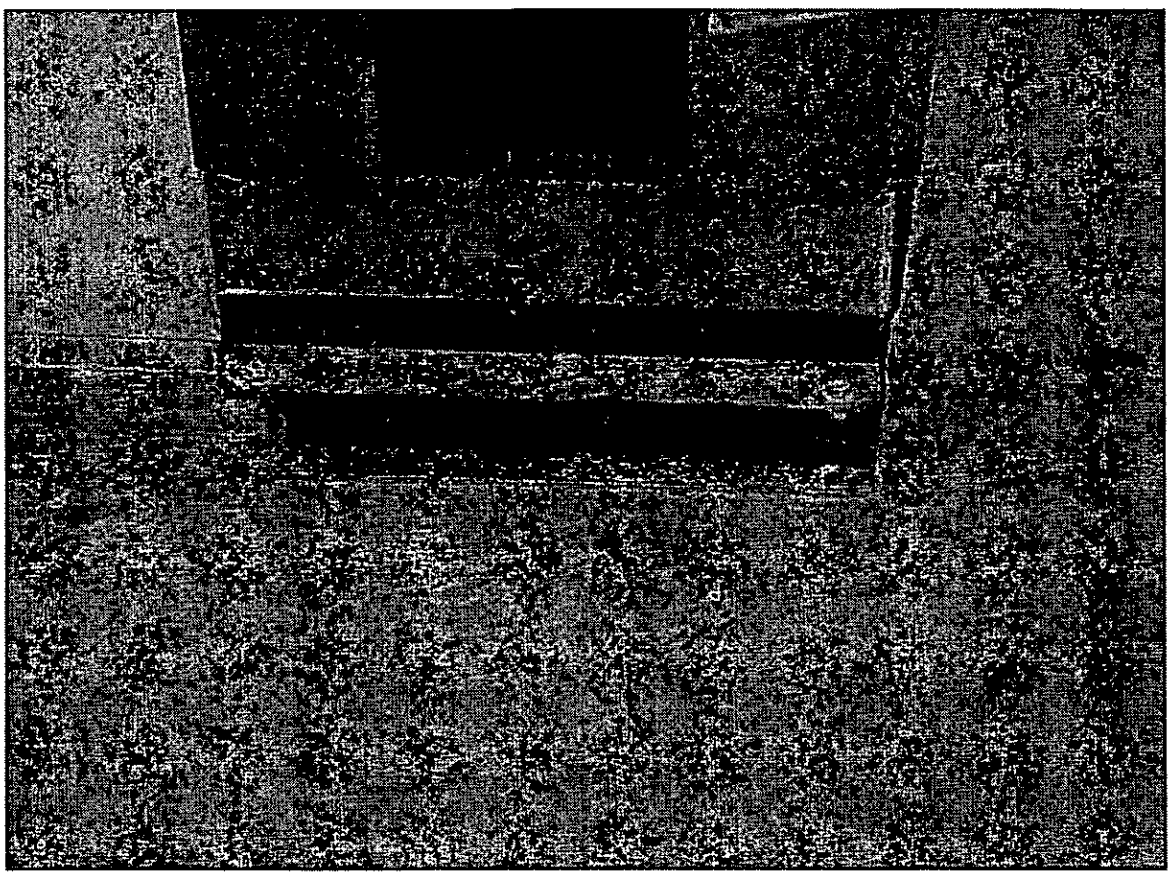

Photo 22: Influent Pump Station. No anchorage of control equipment.

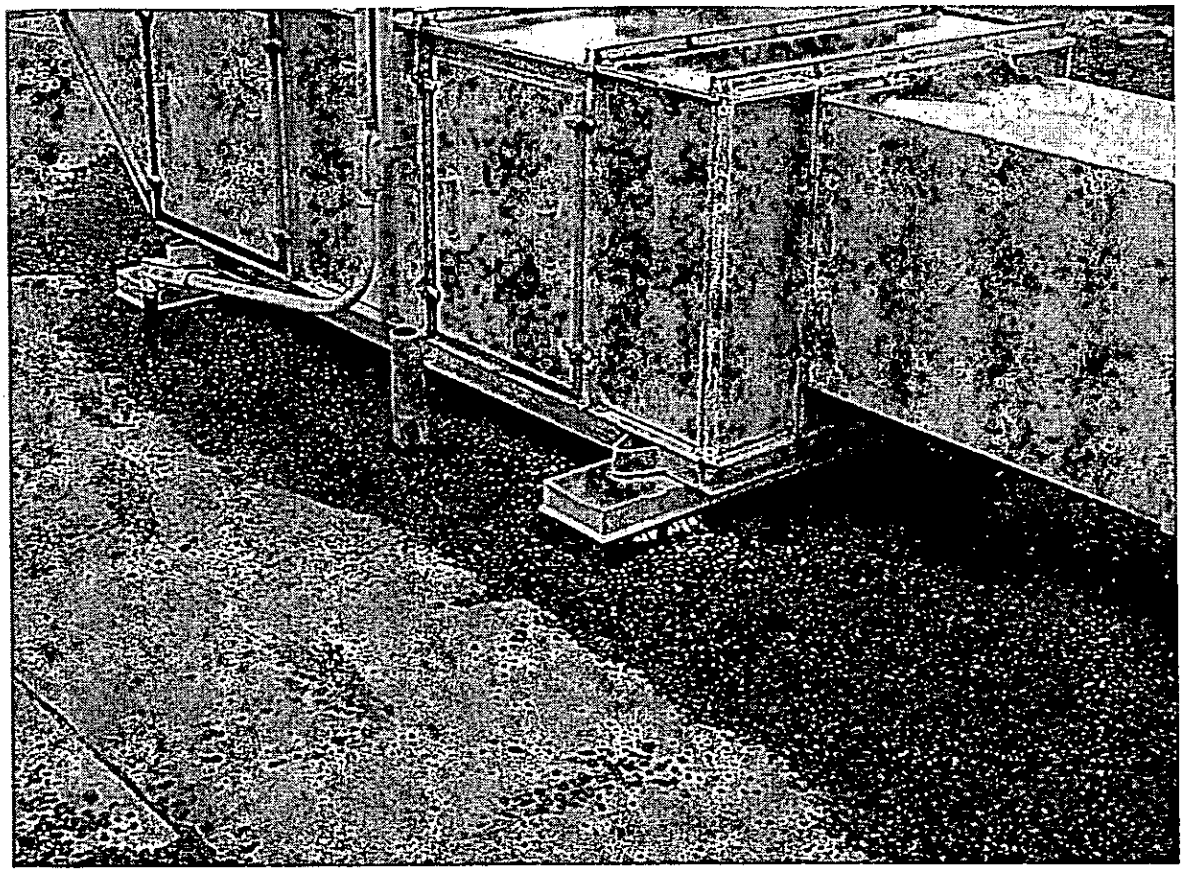

Photo 23: Influent Pump Station. HVAC equipment on roof unanchored.

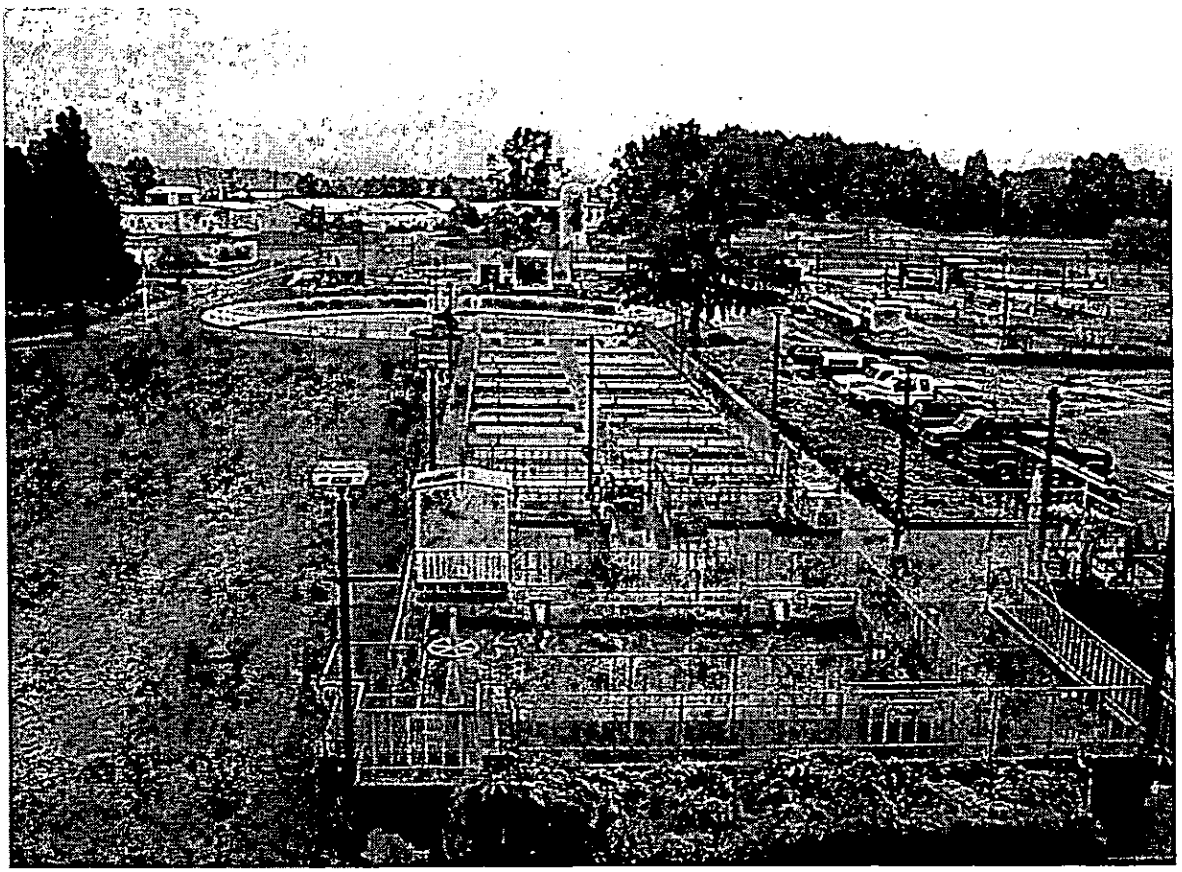

Photo 24: Primary Sedimentation.

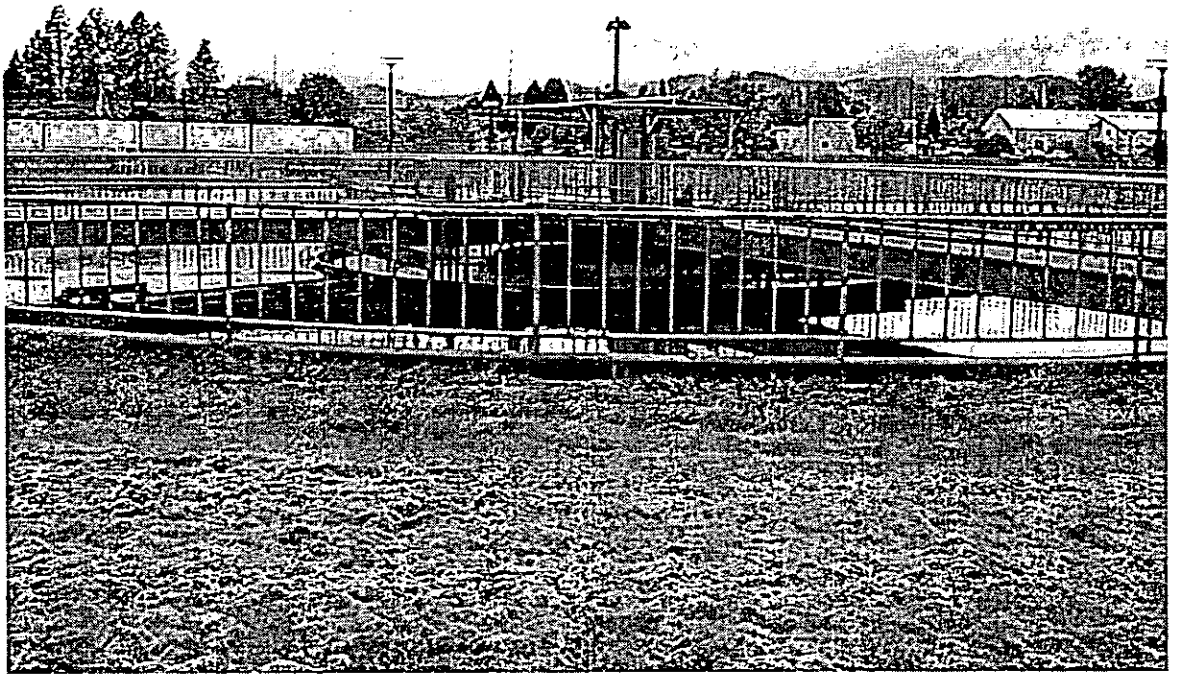

Photo 25: Secondary Clarifier.

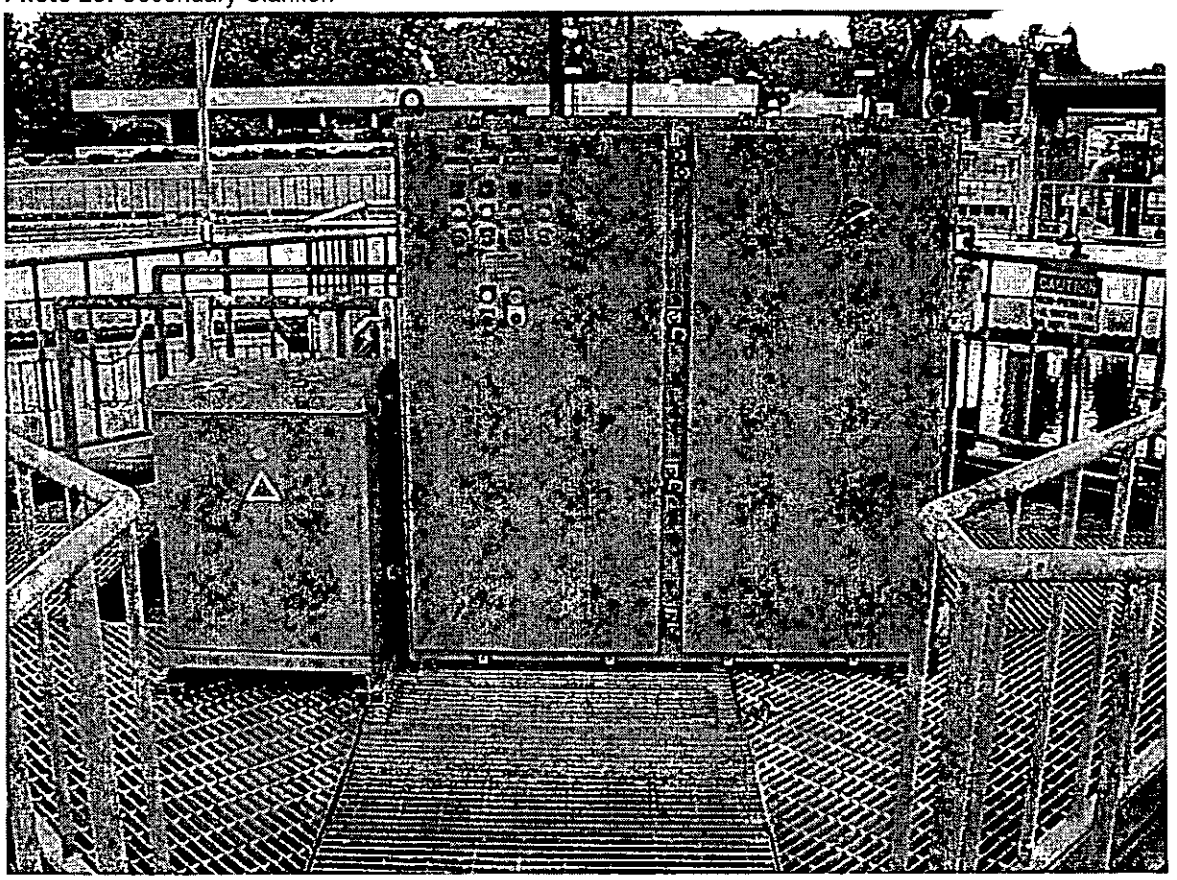

Photo 26: Secondary Clarifier. Control equipment at center pier is not anchored.

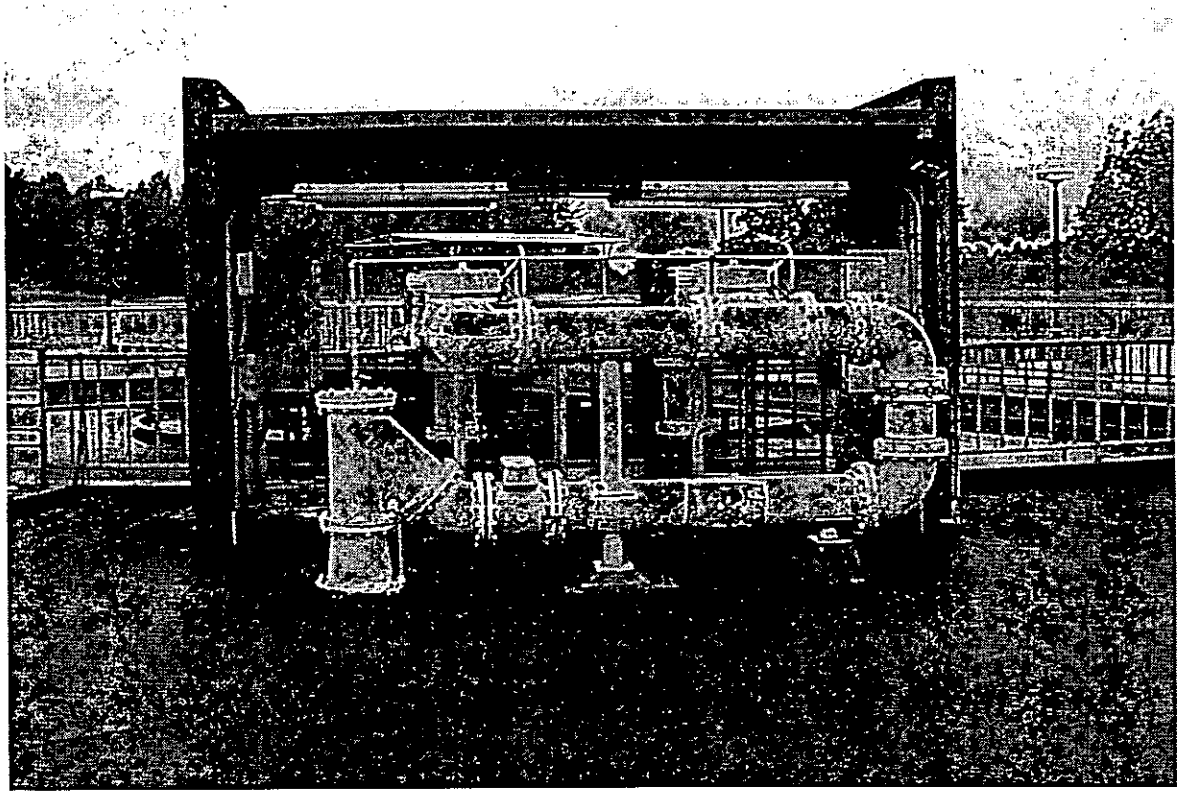

Photo 27: Secondary Clarifier. RAS pump and piping cover.

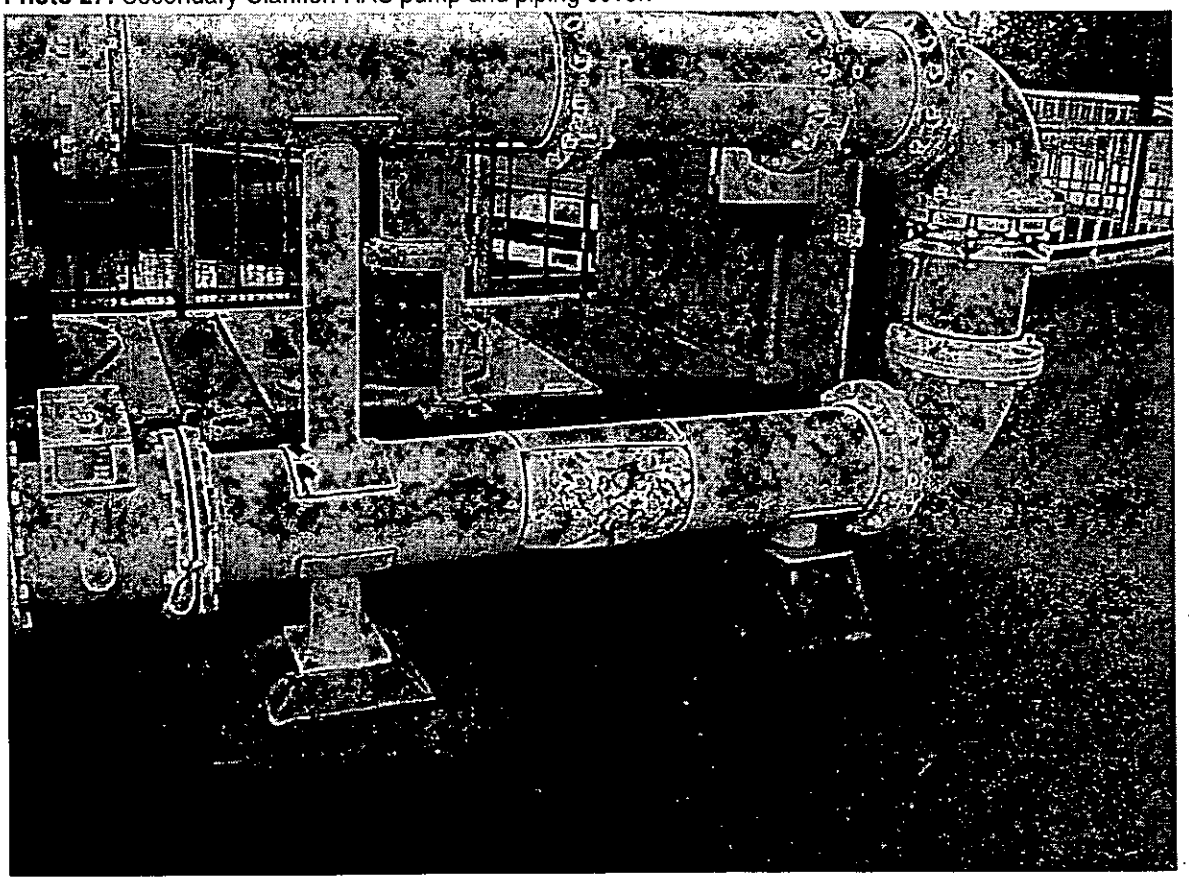

Photo 28: Secondary Clarifier. RAS pump and piping cover. Stacked pipe supports and pipe supports not anchored to concrete.

Photo 29: Aeration Tanks.

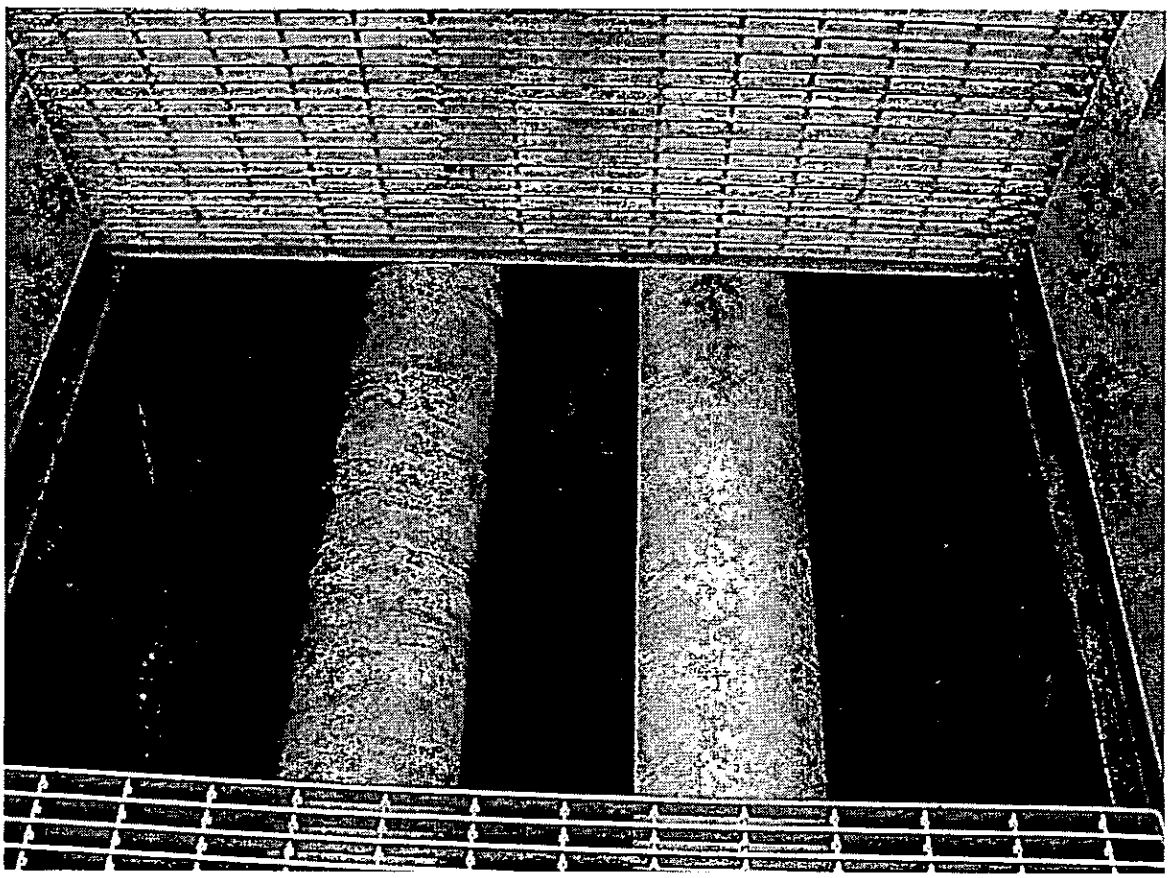

Photo 30: Aeration Tanks. Air piping not braced.

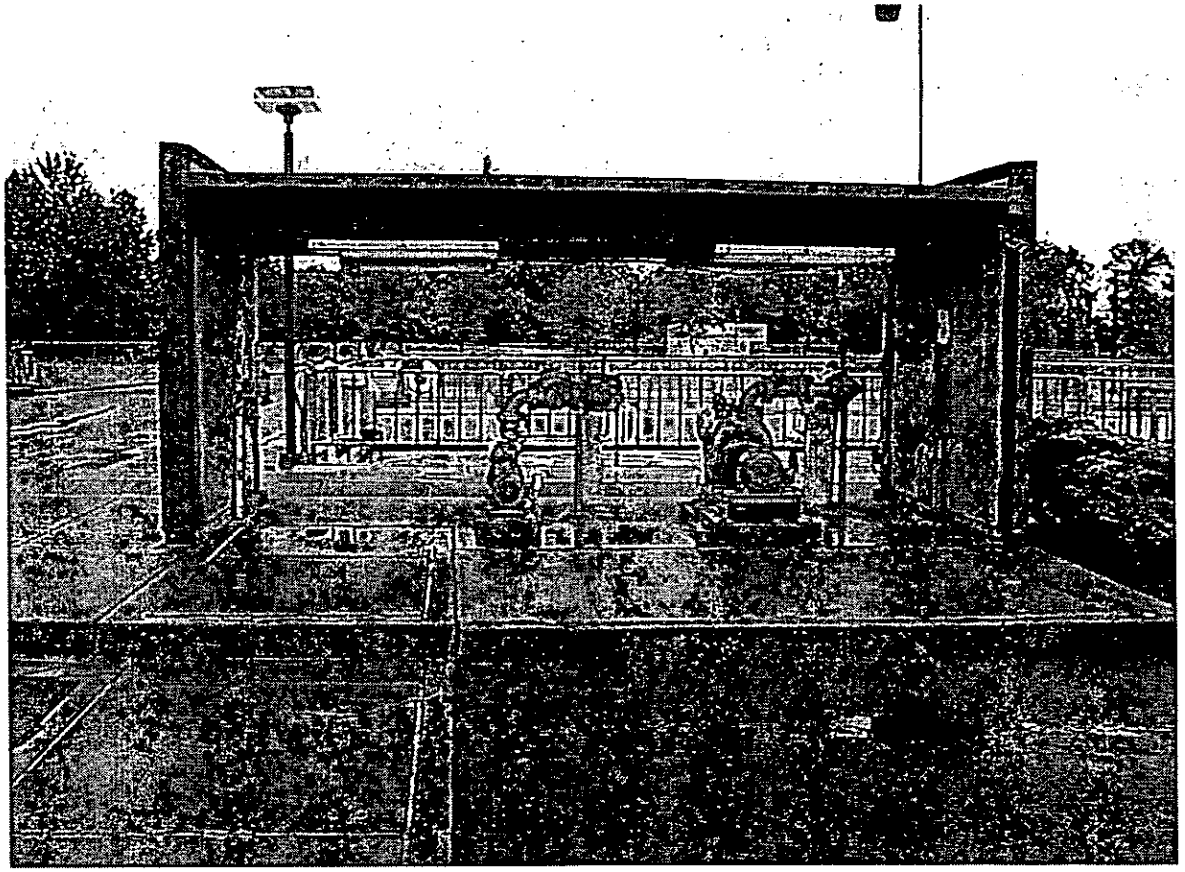

Photo 31: Aeration Tanks. WAS pump cover.

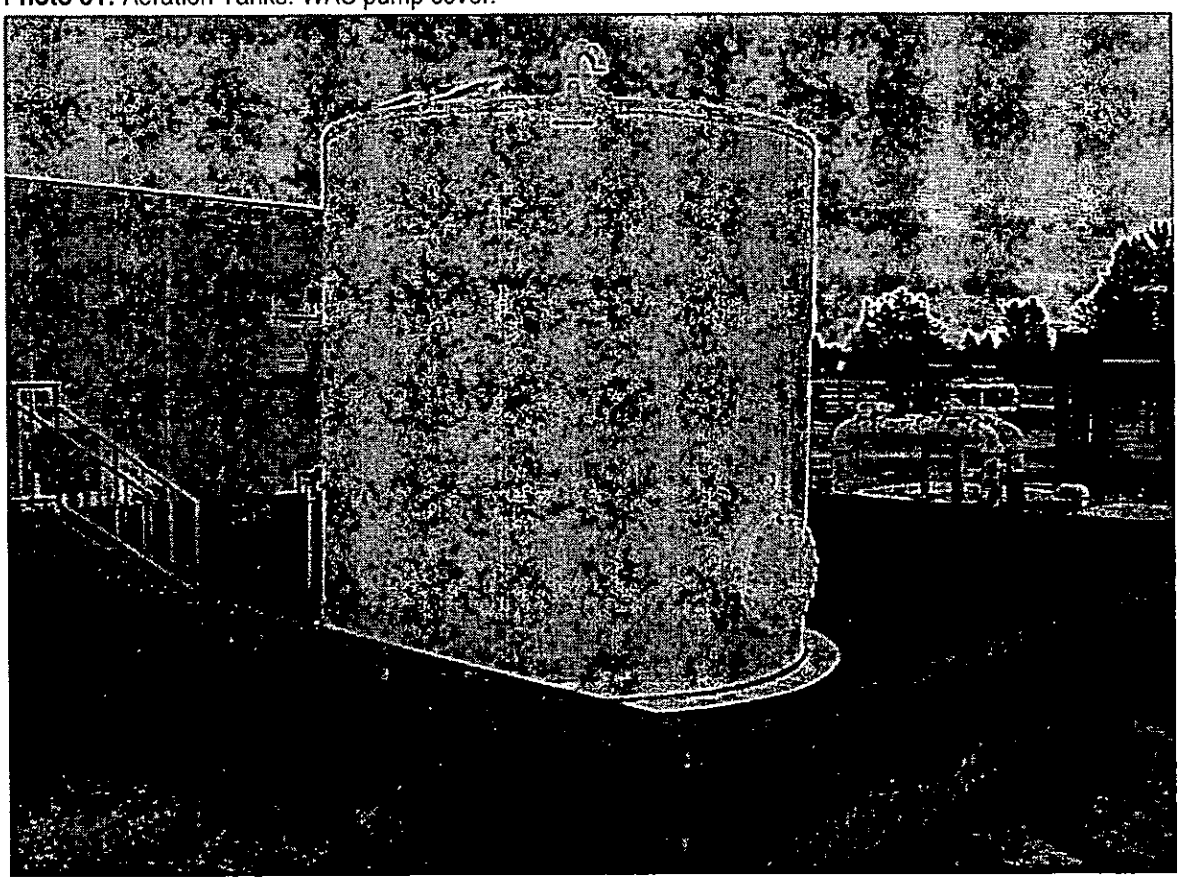

Photo 32: Sludge Storage Pump Station. Chemical tank and containment slab.

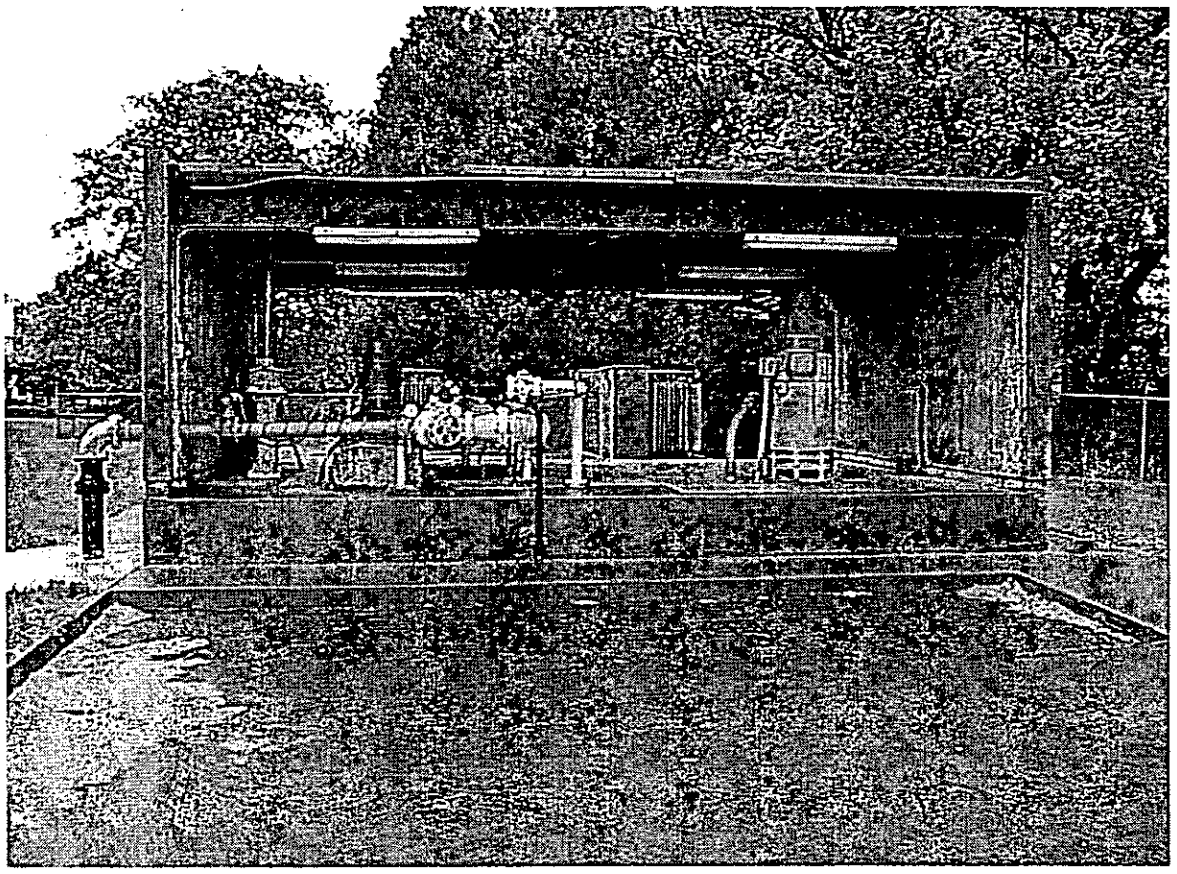

Photo 33: Effluent Pump Station and Cover. Note transformer pad in background.

Photo 34: Effluent Pump Station. Typical beam to concrete wall connection.

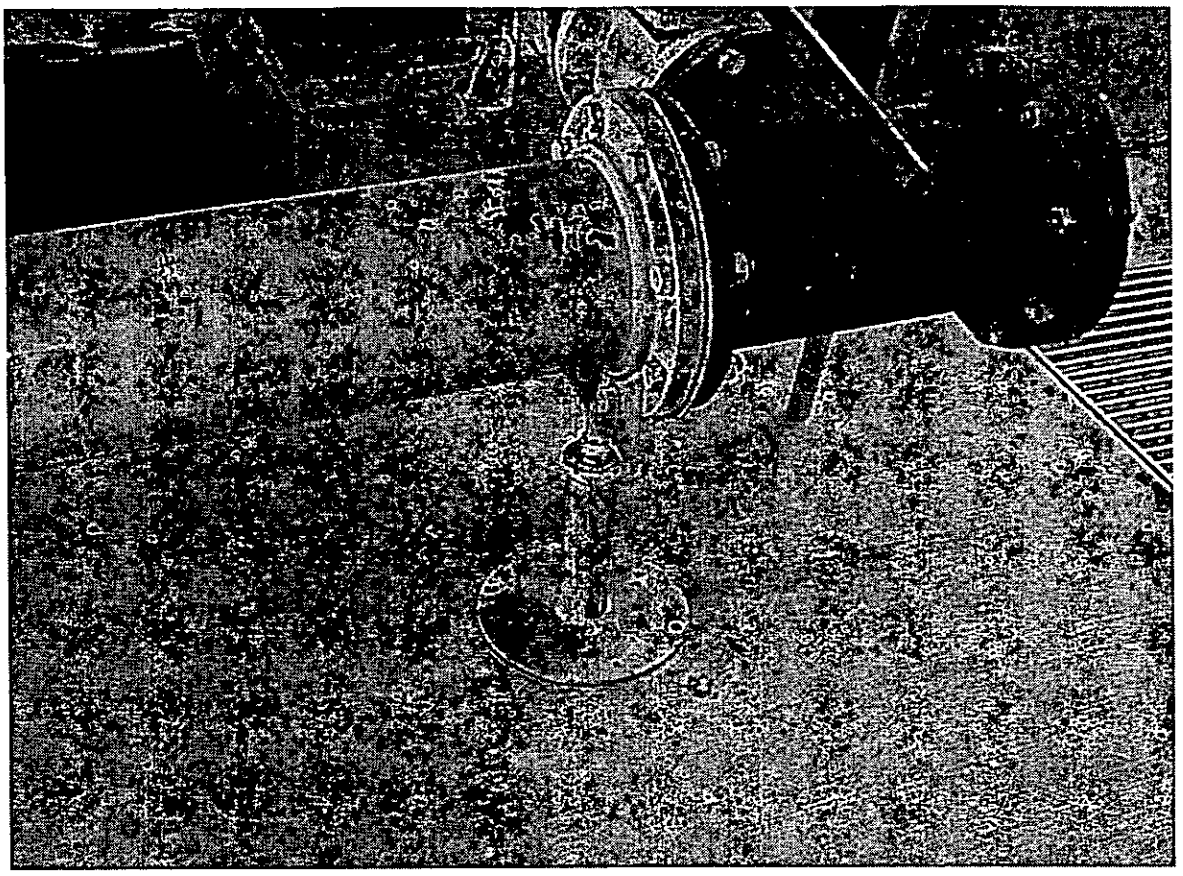

Photo 35: Effluent Pump Station. Pipe support not anchored to concrete slab.

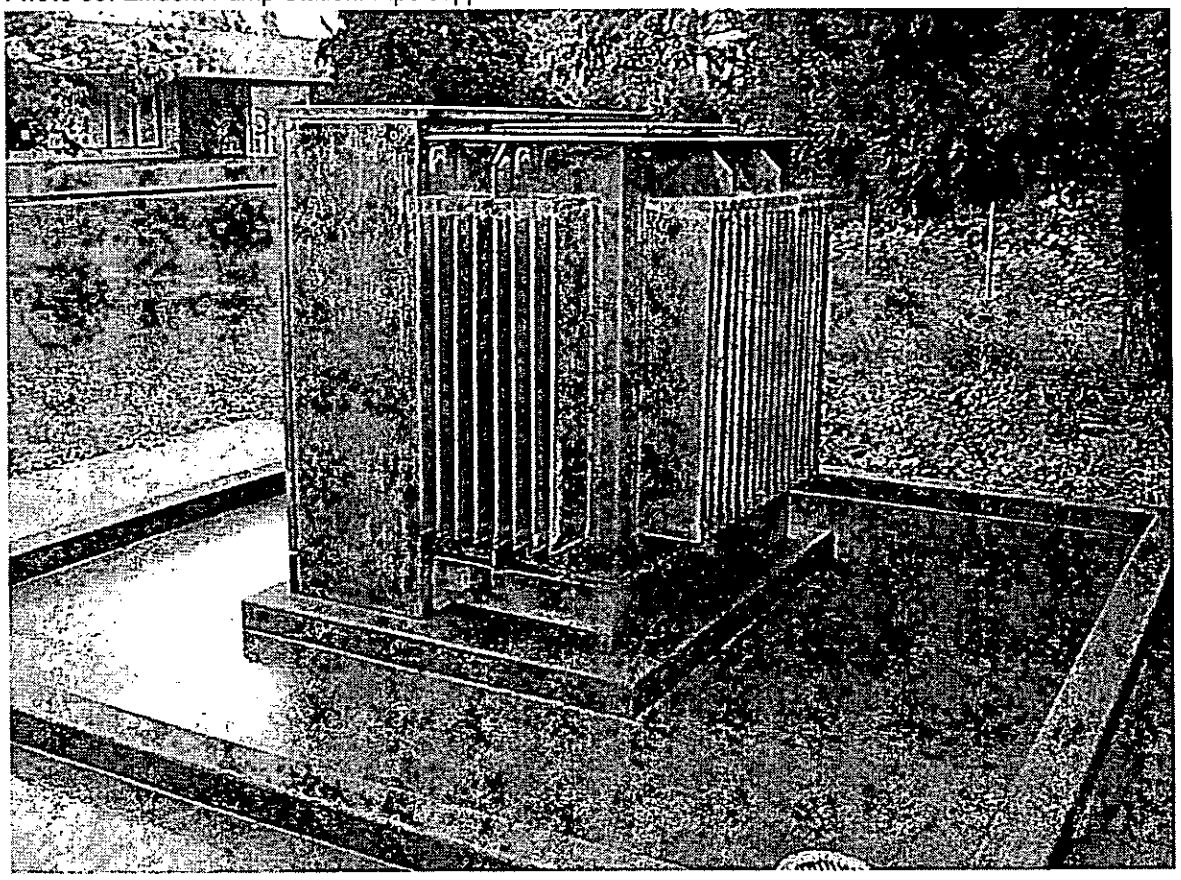

Photo 36: Transformer Pad at Effluent Pump Station.

Photo 37: Transformer Pad. Lack of anchorage for transformer.

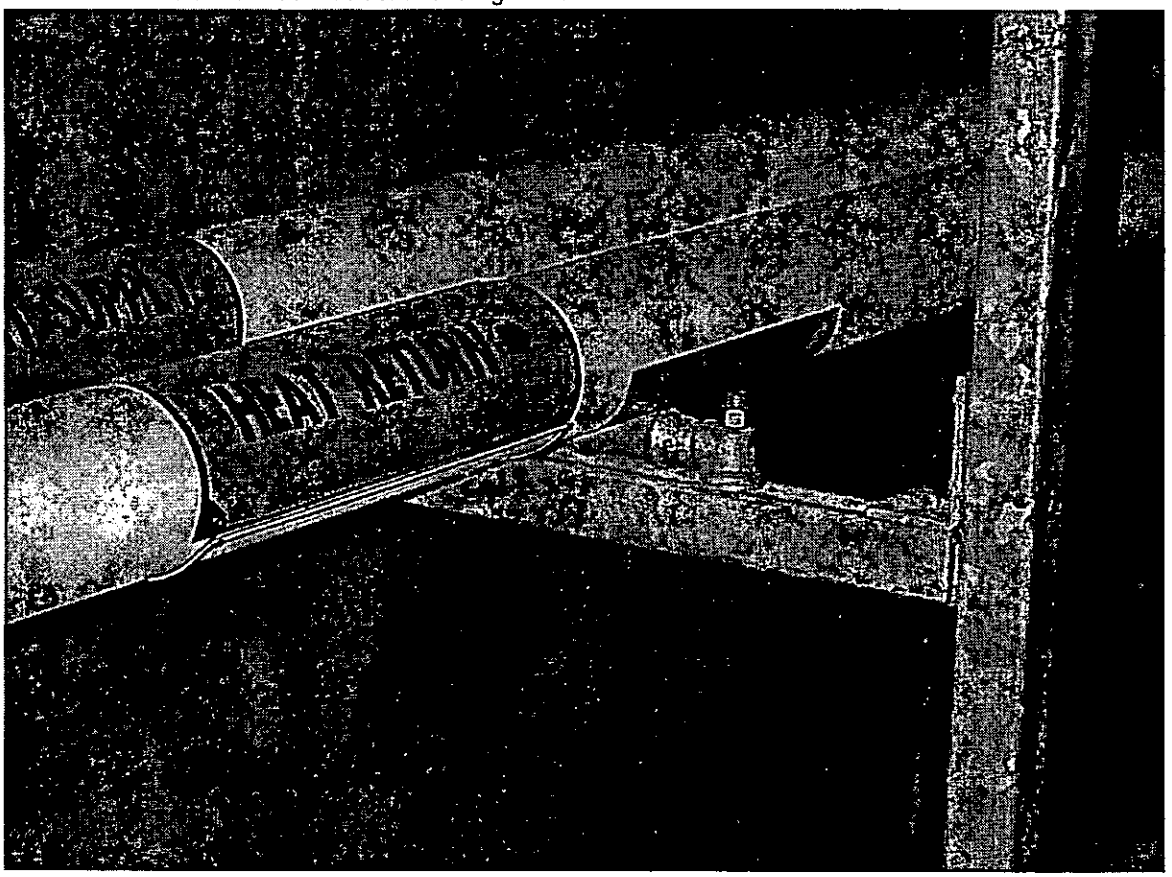

Photo 38: Tunnels. No pipe straps on heat piping.

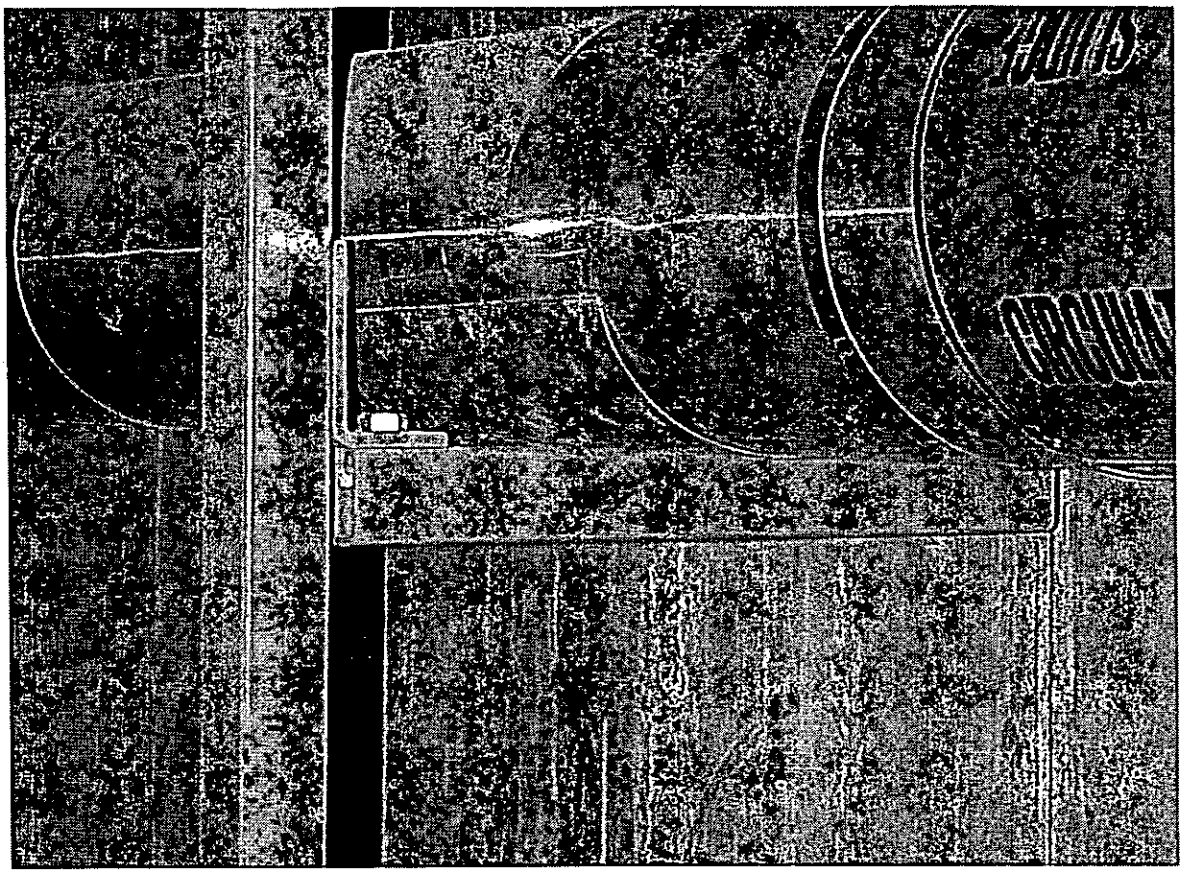

Photo 39: Tunnels. No pipe straps on circulated sludge piping.

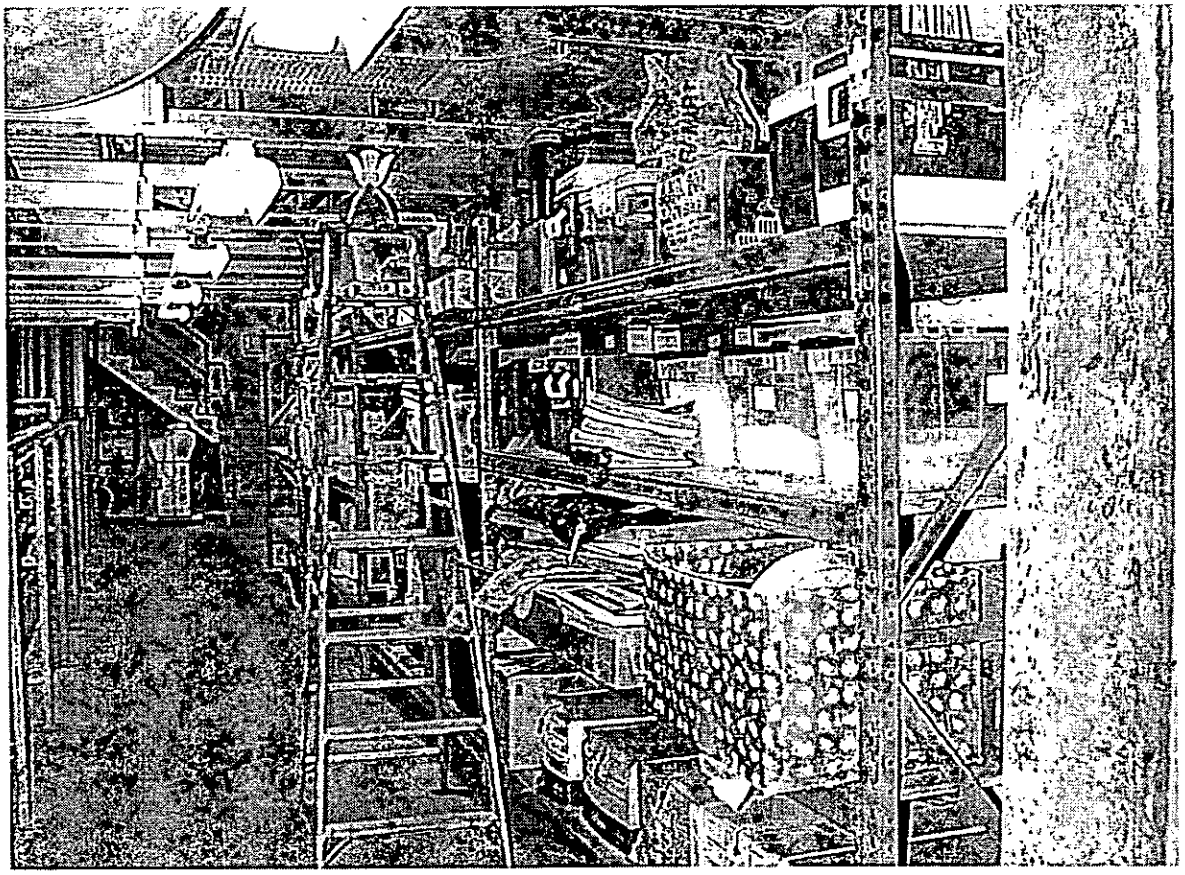

Photo 40: Tunnels. Storage shelves in tunnel.

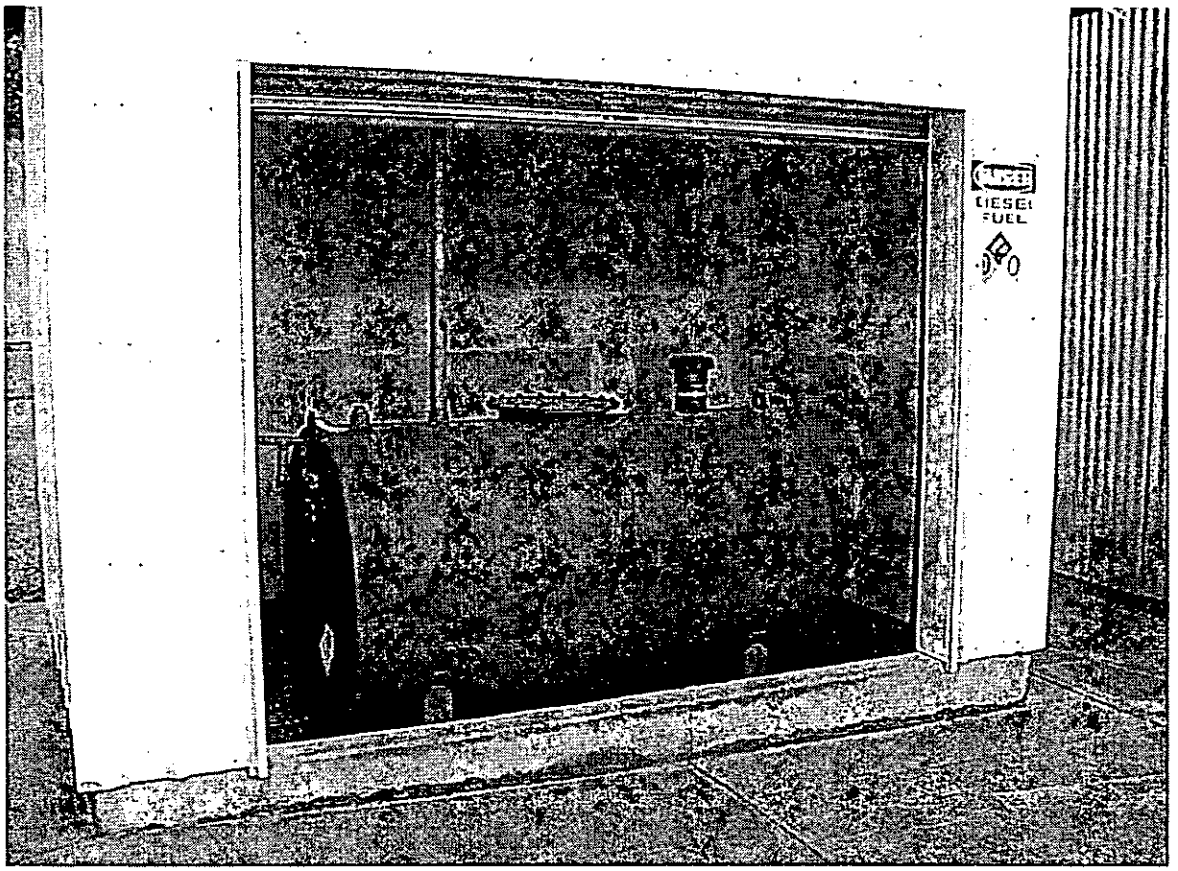

Photo 41: Diesel Oil Tank Building.

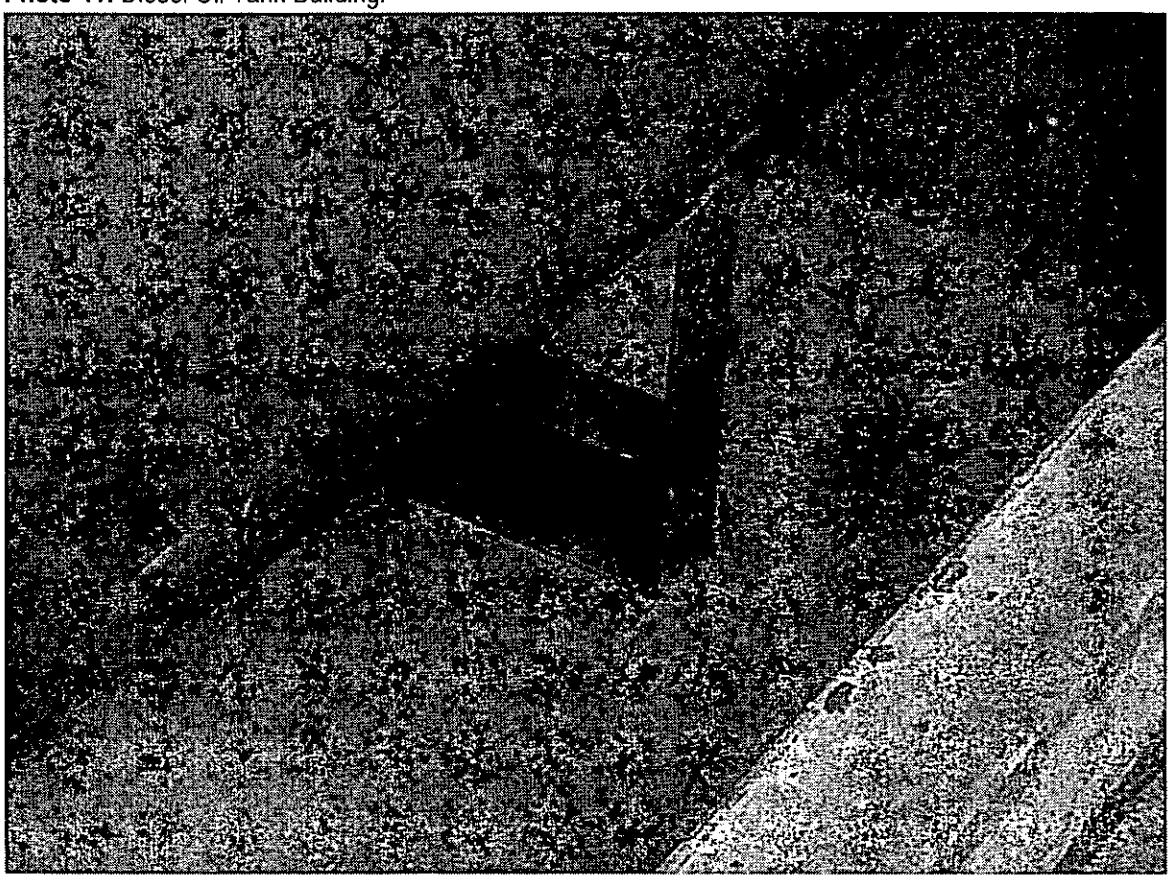

Photo 42: Diesel Oil Tank. No anchorage of tank to concrete slab.

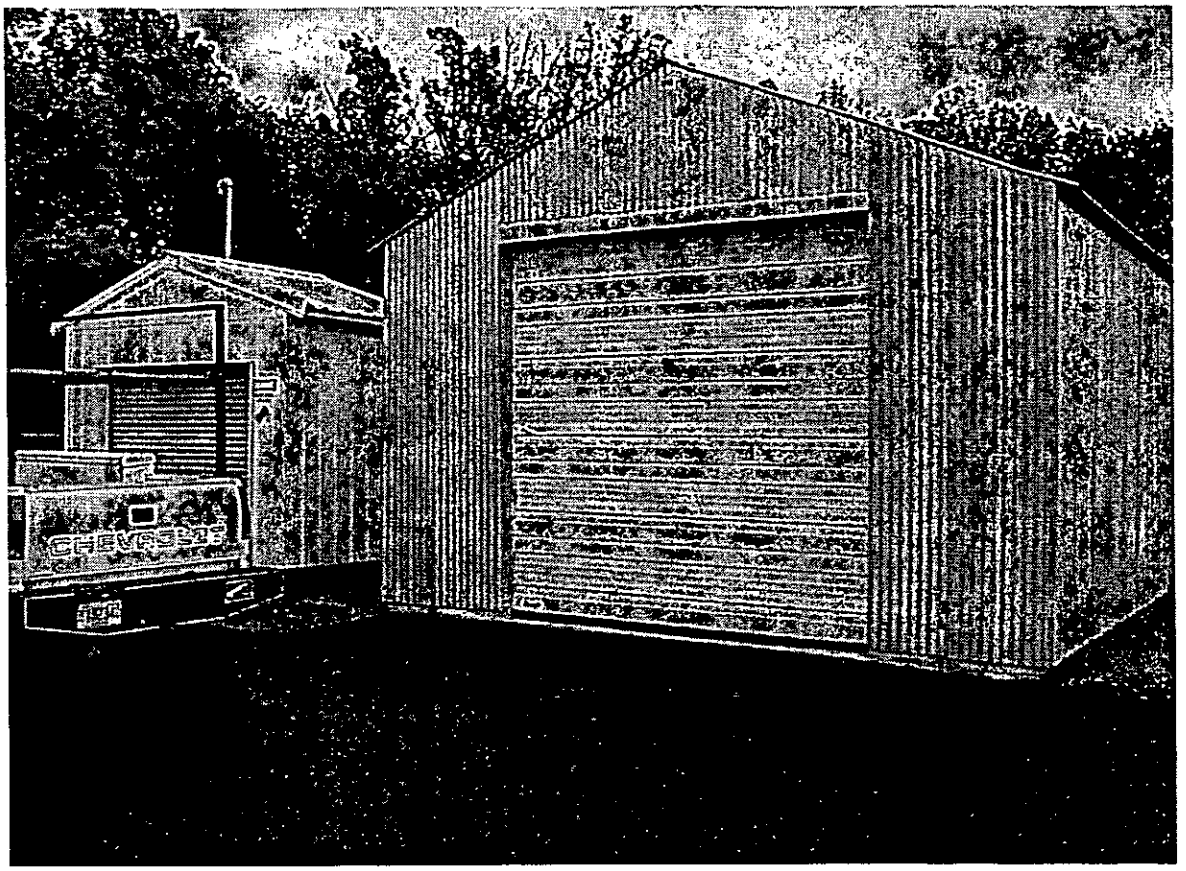

Photo 43: Empty Metal Building.

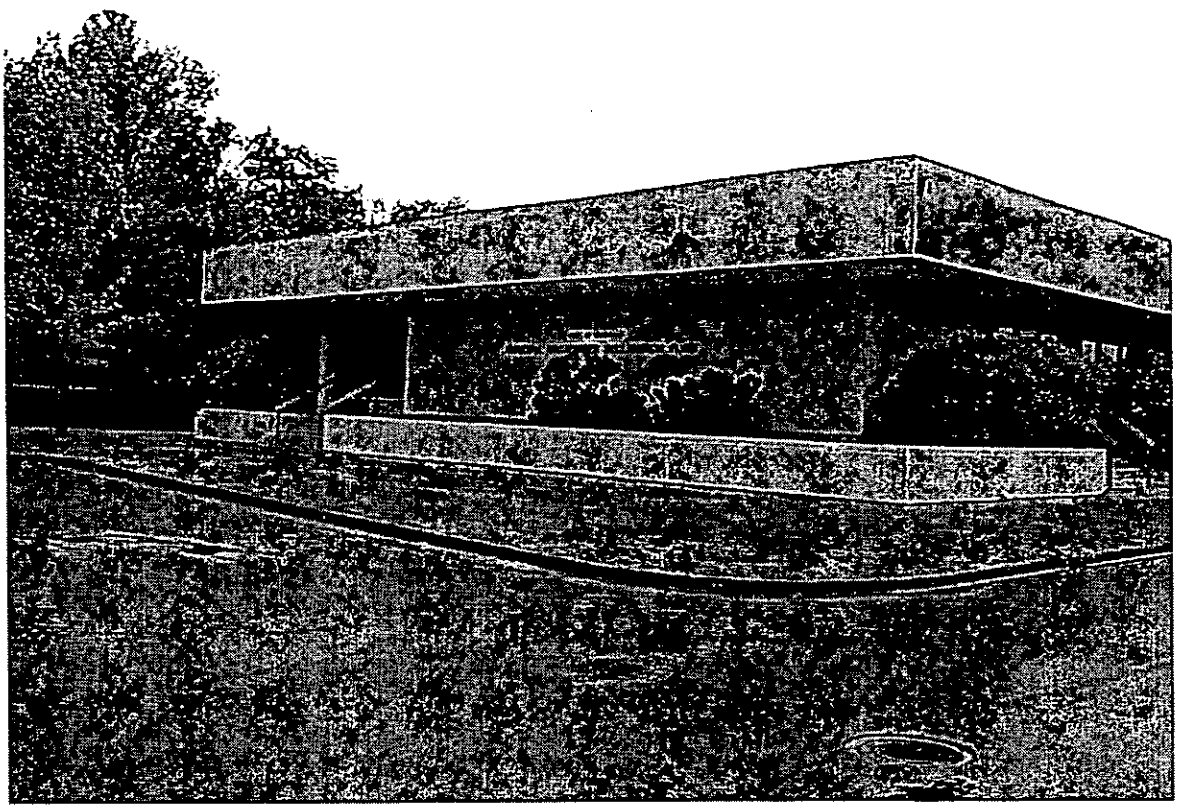

Photo 44: Operations and Maintenance Building.

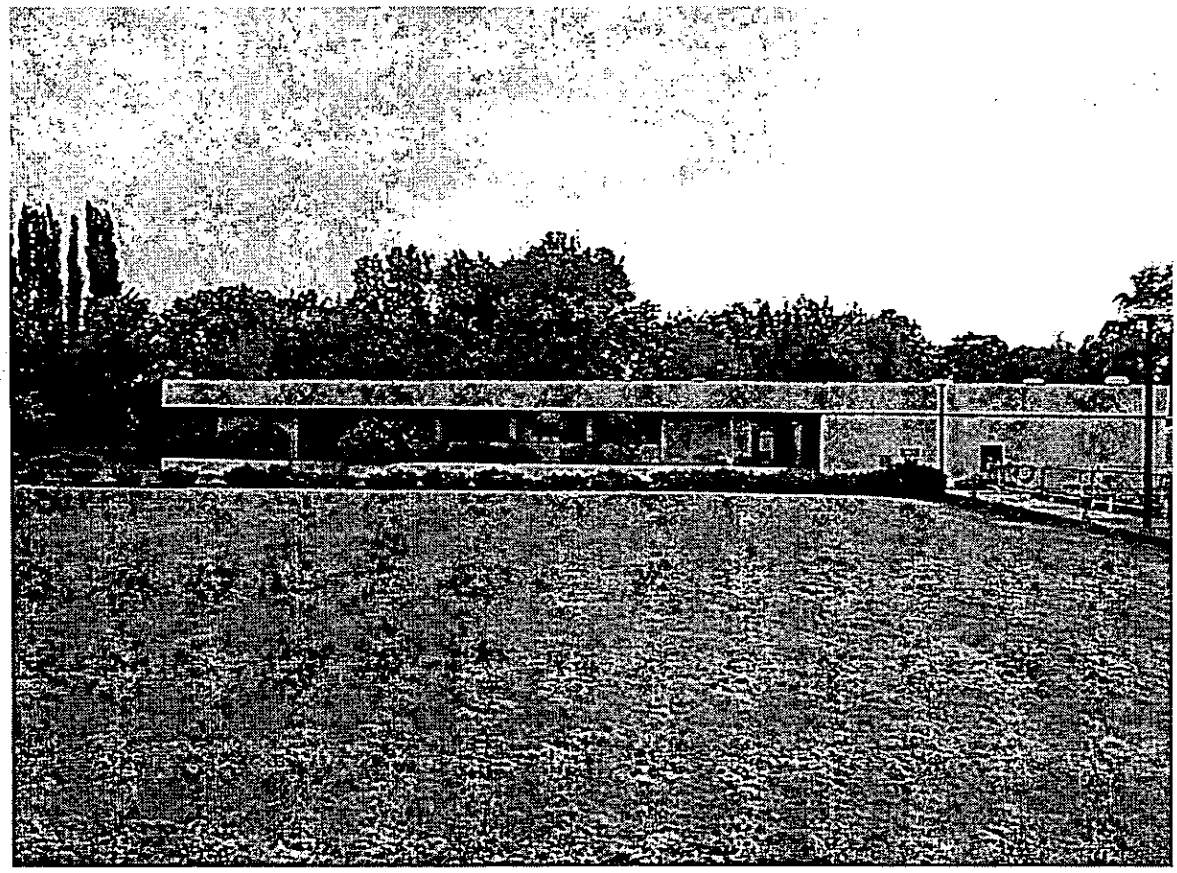

Photo 45: Operations and Maintenance Building.

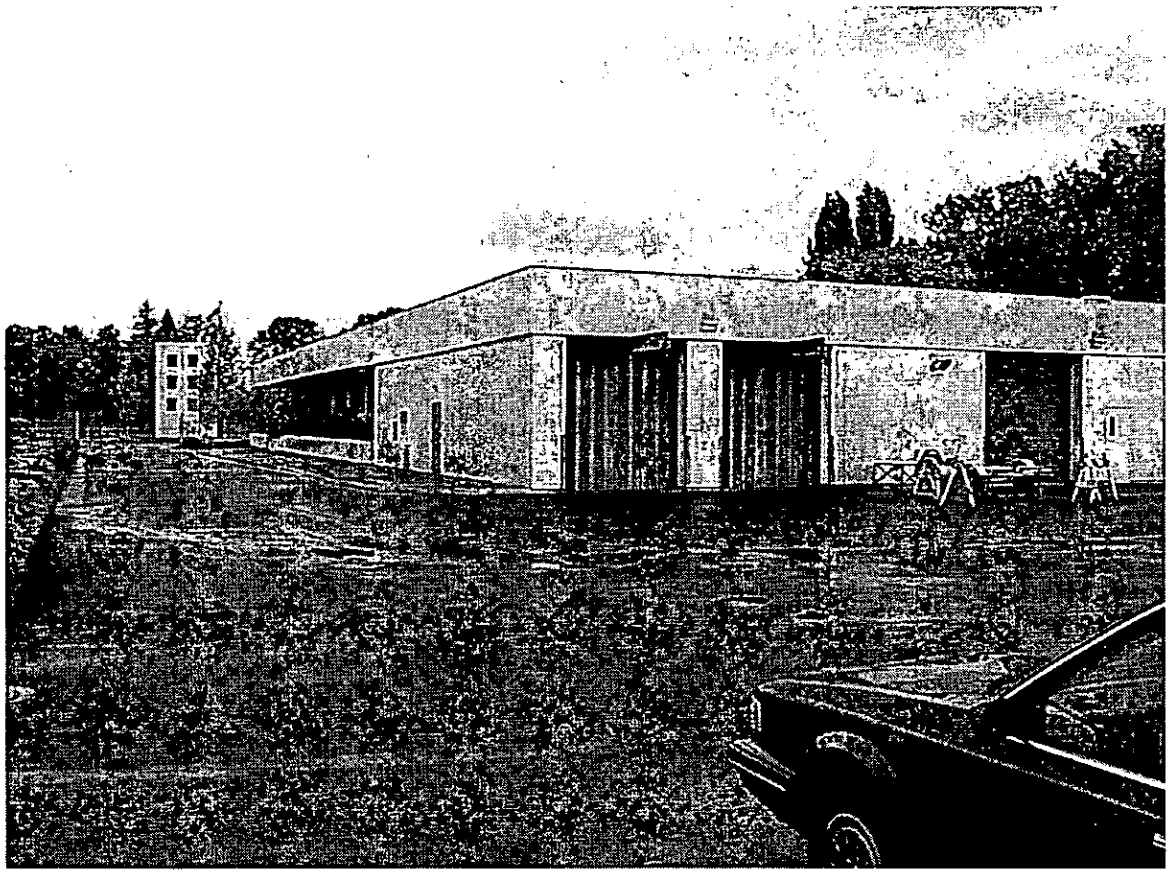

Photo 46: Operations and Maintenance Building.

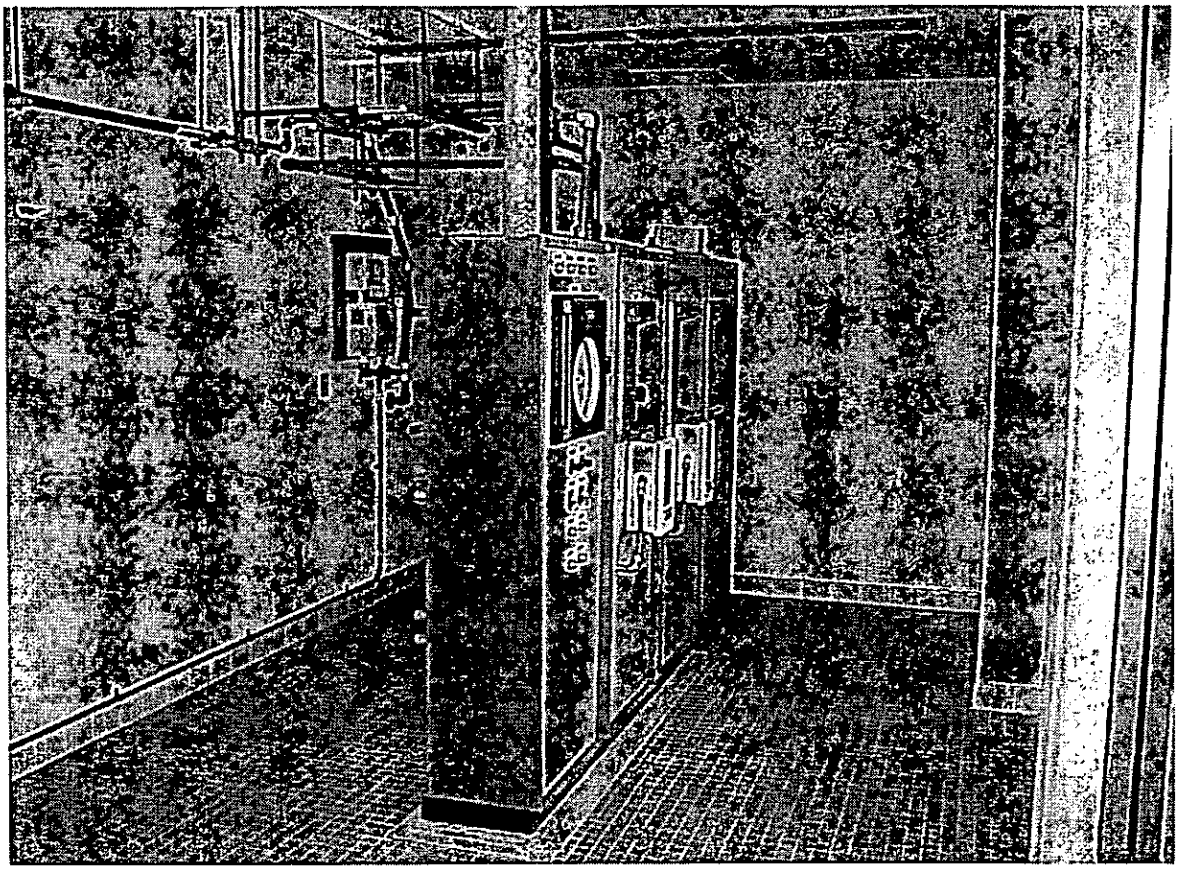

Photo 47: Operations and Maintenance Building. Chlorine Room.

Photo 48: Operations and Maintenance Building. Chlorine cylinder storage.

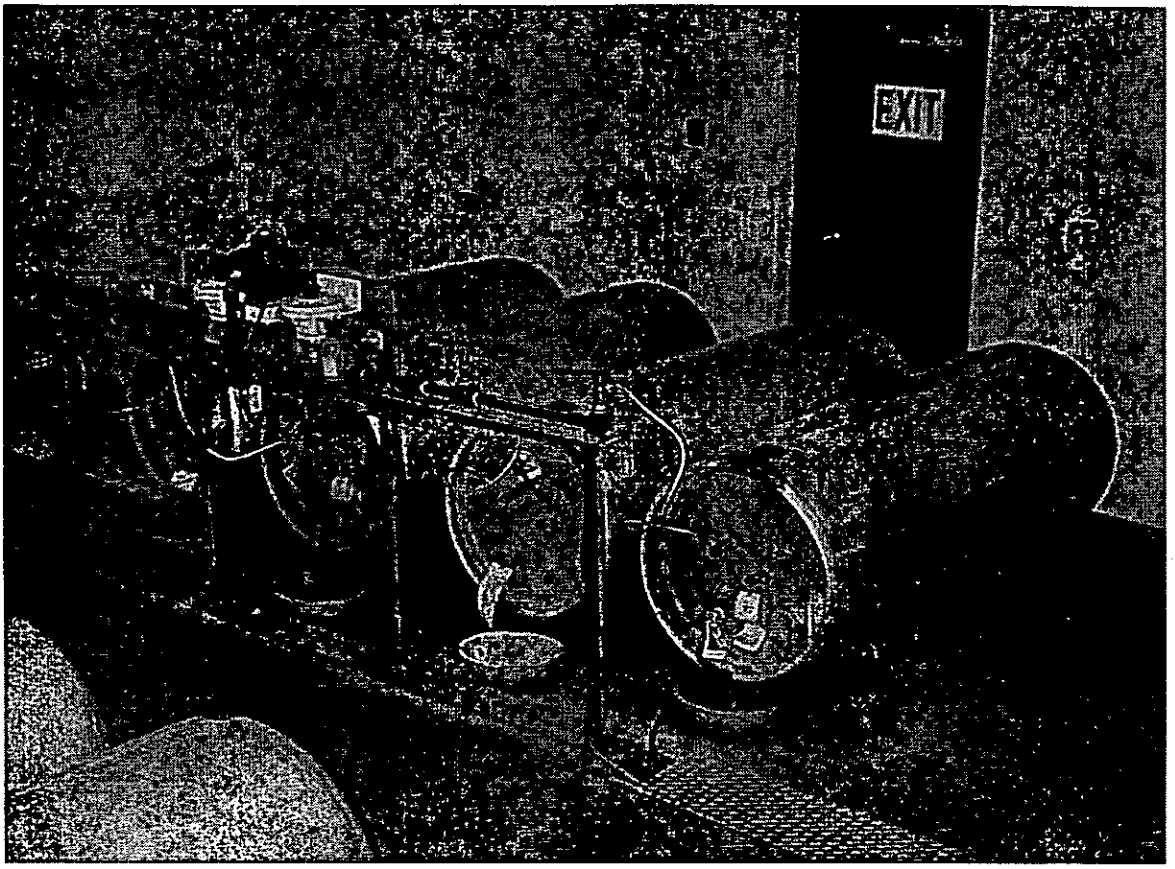

Photo 49: Operations and Maintenance Building. Chlorine cylinders in use.

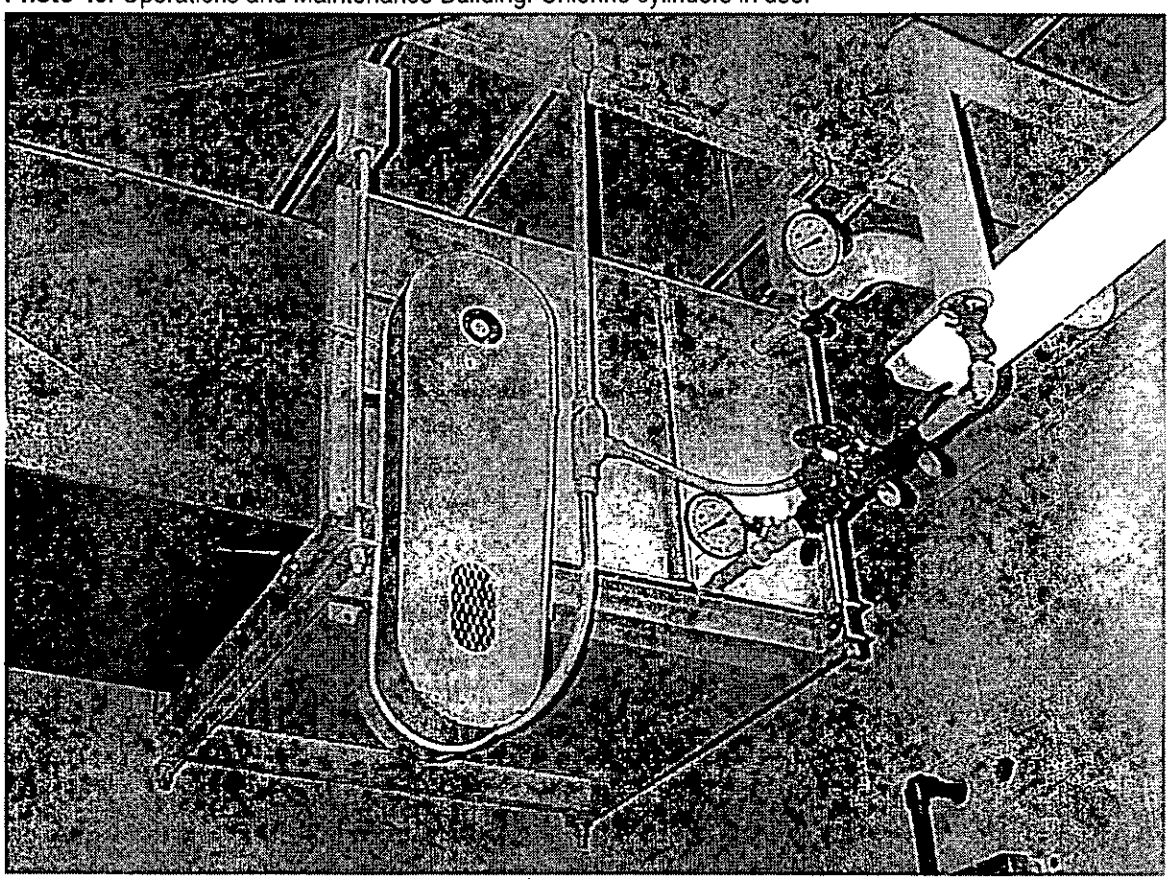

Photo 50: Operations and Maintenance Building. Chlorine storage room. Unbraced HVAC unit.

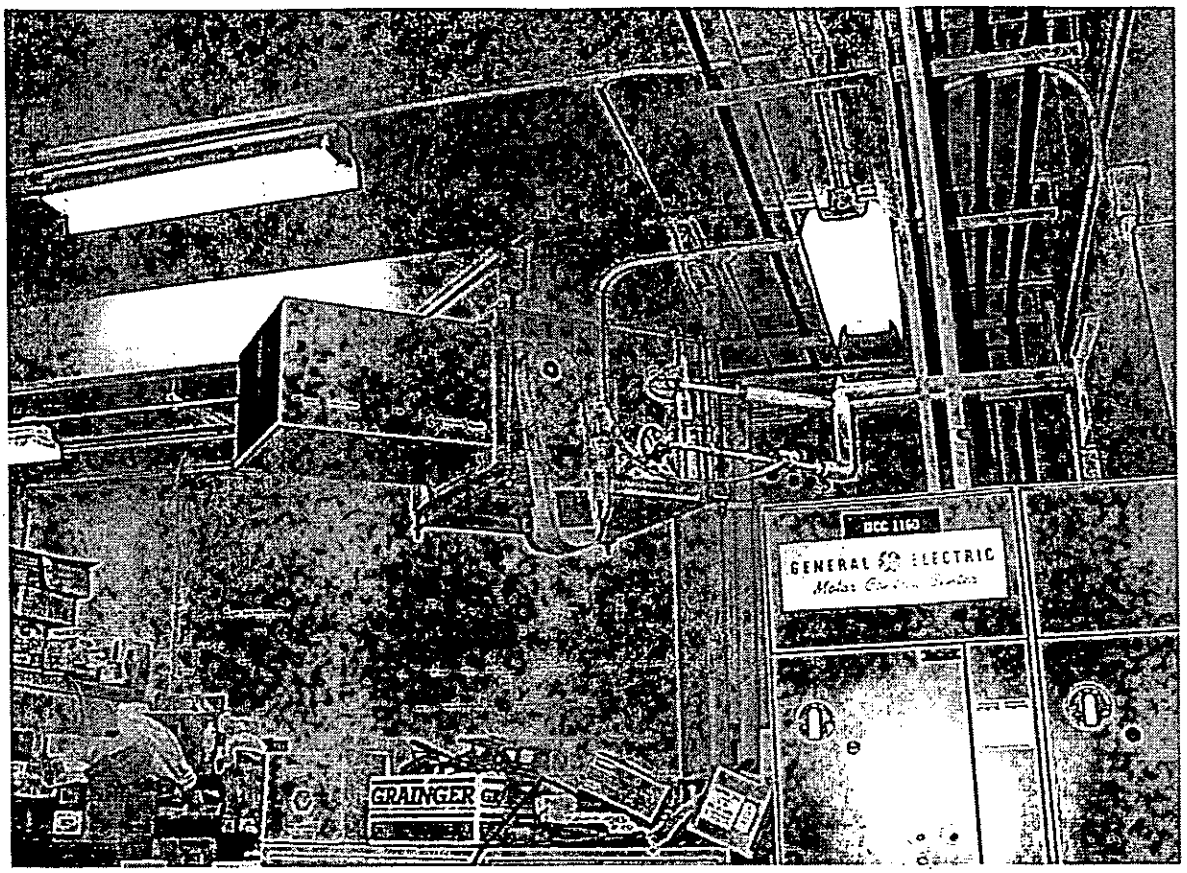

Photo 51: Operations and Maintenance Building. Maintenance Bay. Unbraced HVAC unit.

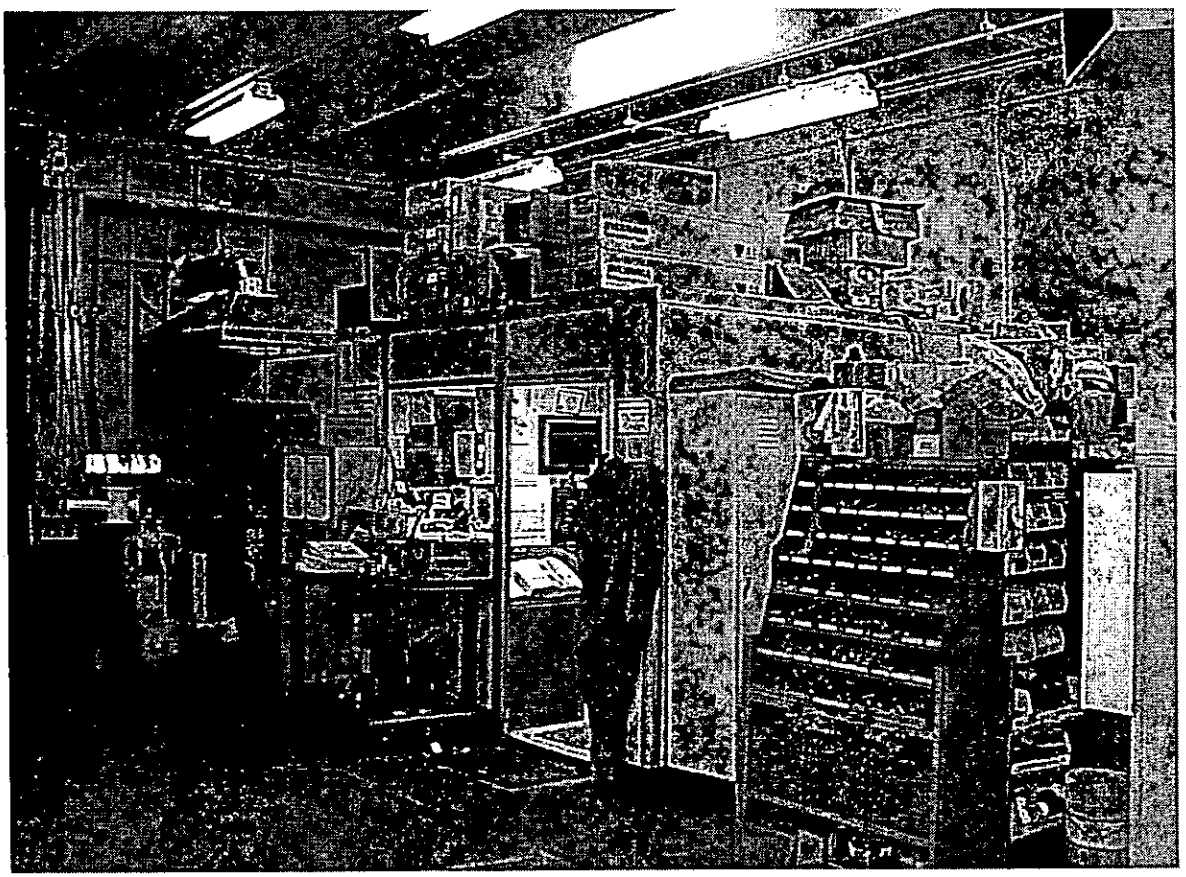

Photo 52: Operations and Maintenance Building. Maintenance Bay. Interior office with material stored above office space.

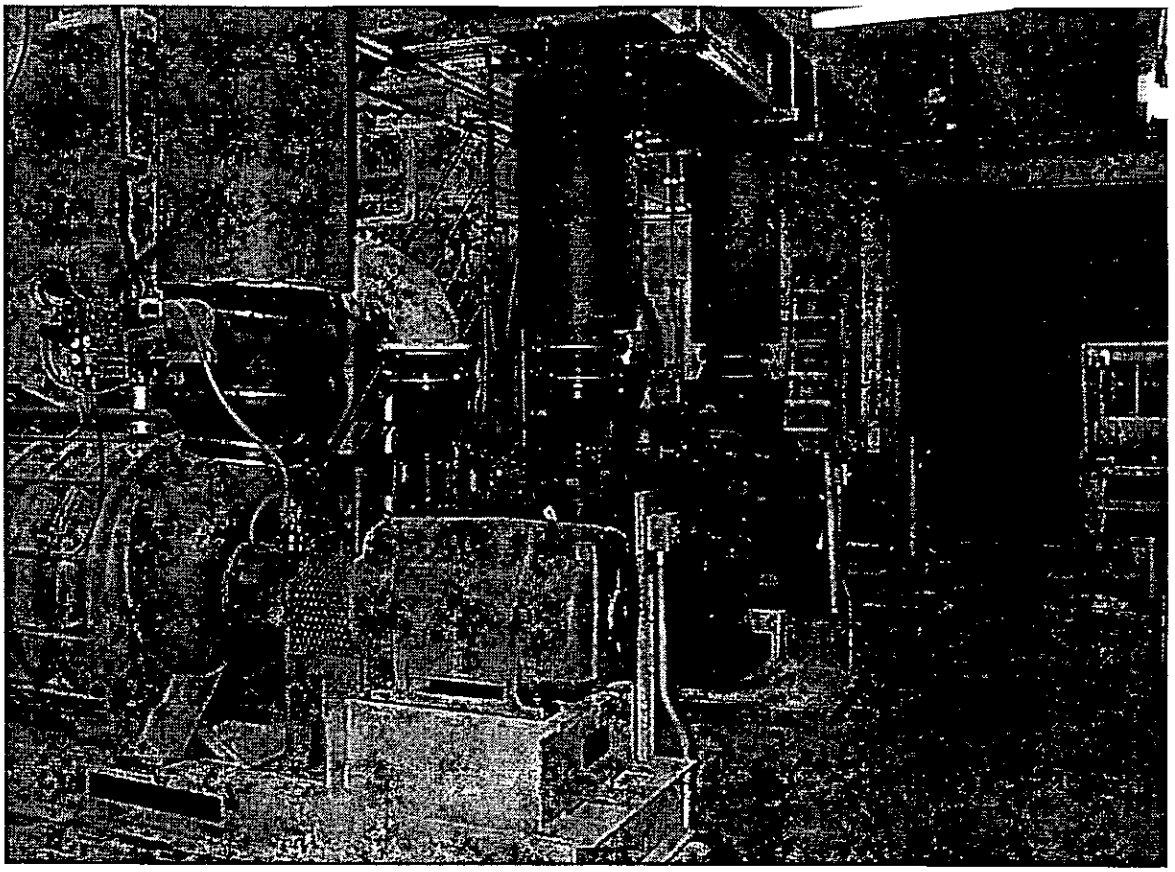

Photo 53: Operations and Maintenance Building. Blower Room.

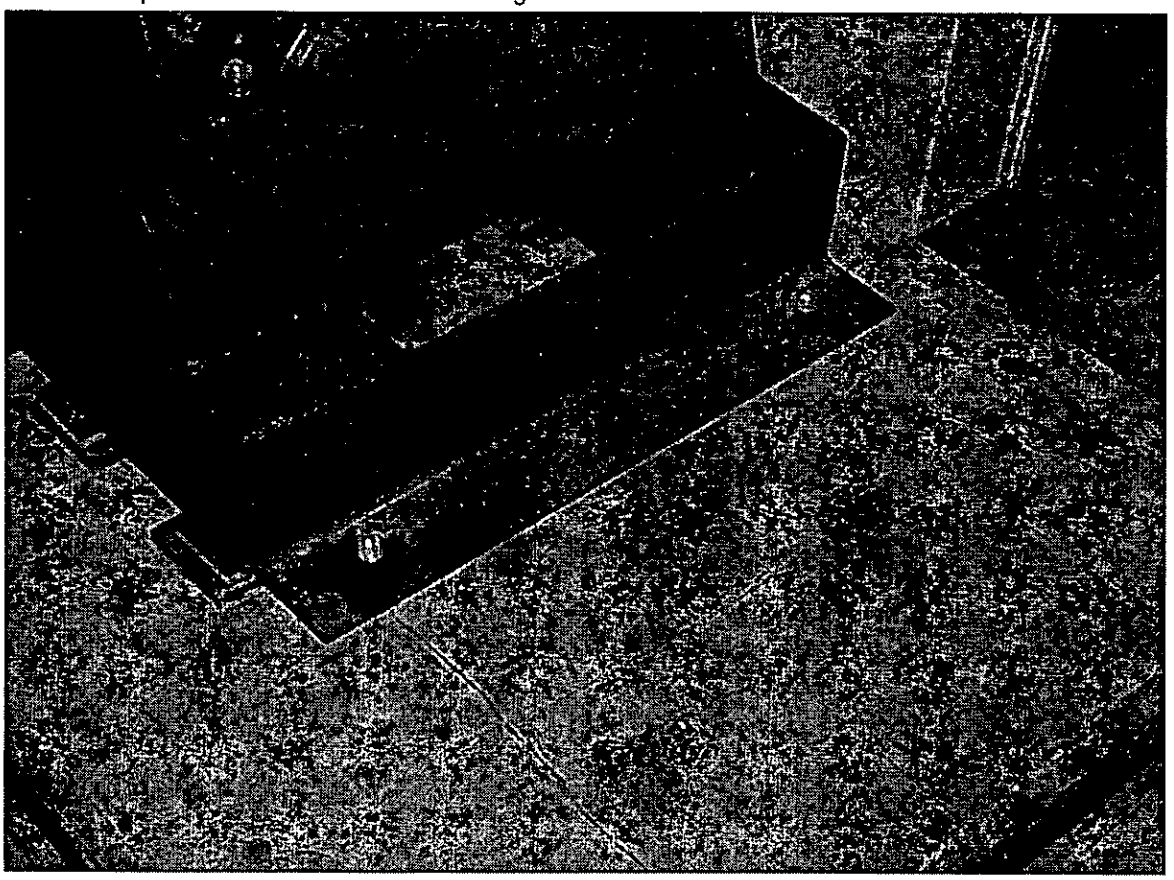

Photo 54: Operations and Maintenance Building. Blower Room. No nuts on blower anchor bolts.

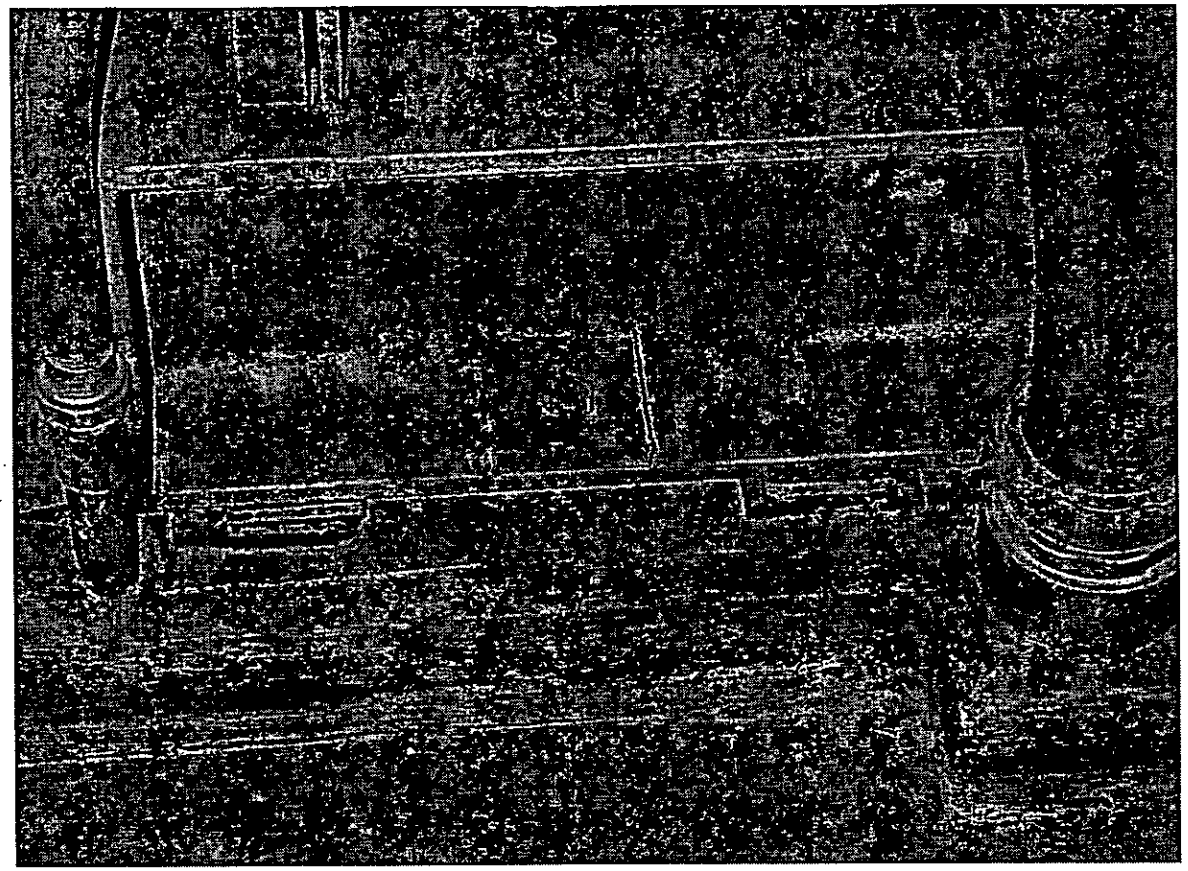

Photo 55: Operations and Maintenance Building. Blower Room. No nut on blower anchor bolt.

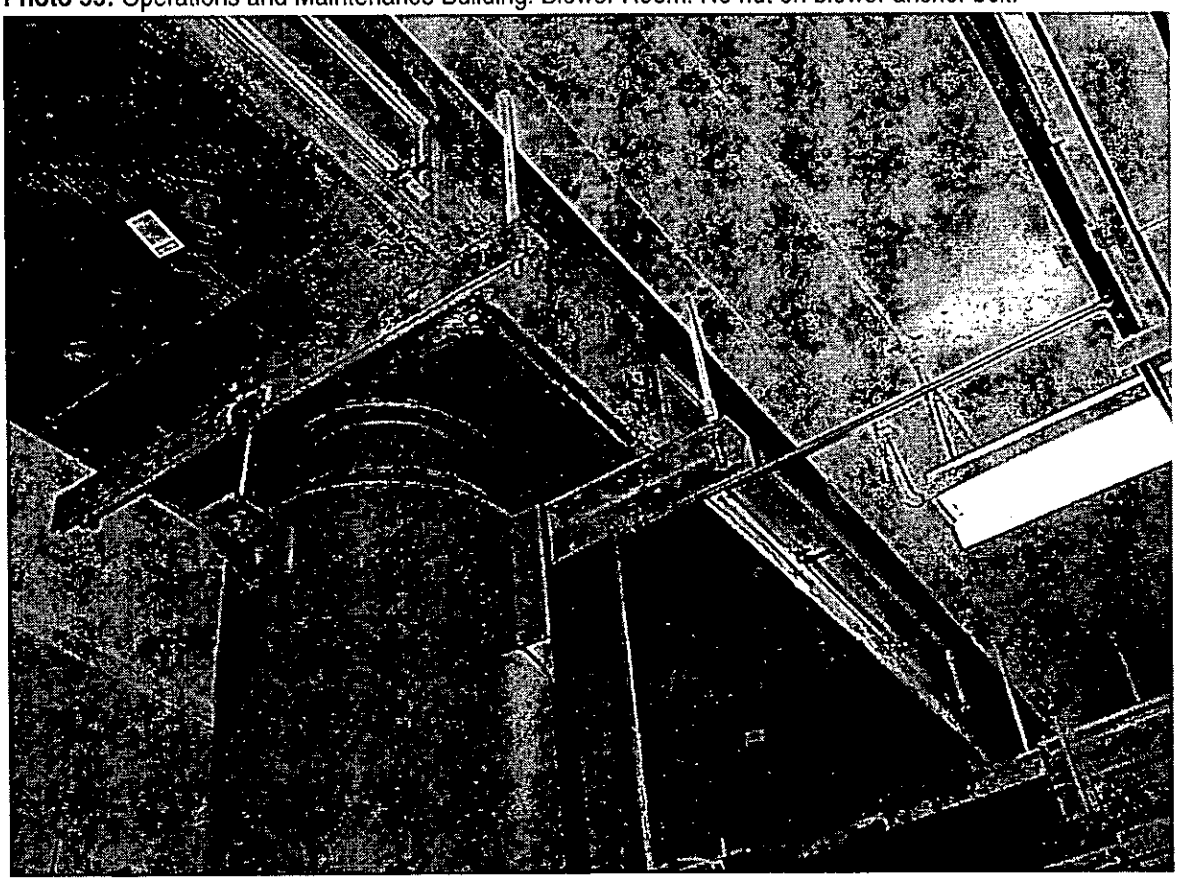

Photo 56: Operations and Maintenance Building. Blower Room. Vertical blower piping unbraced.

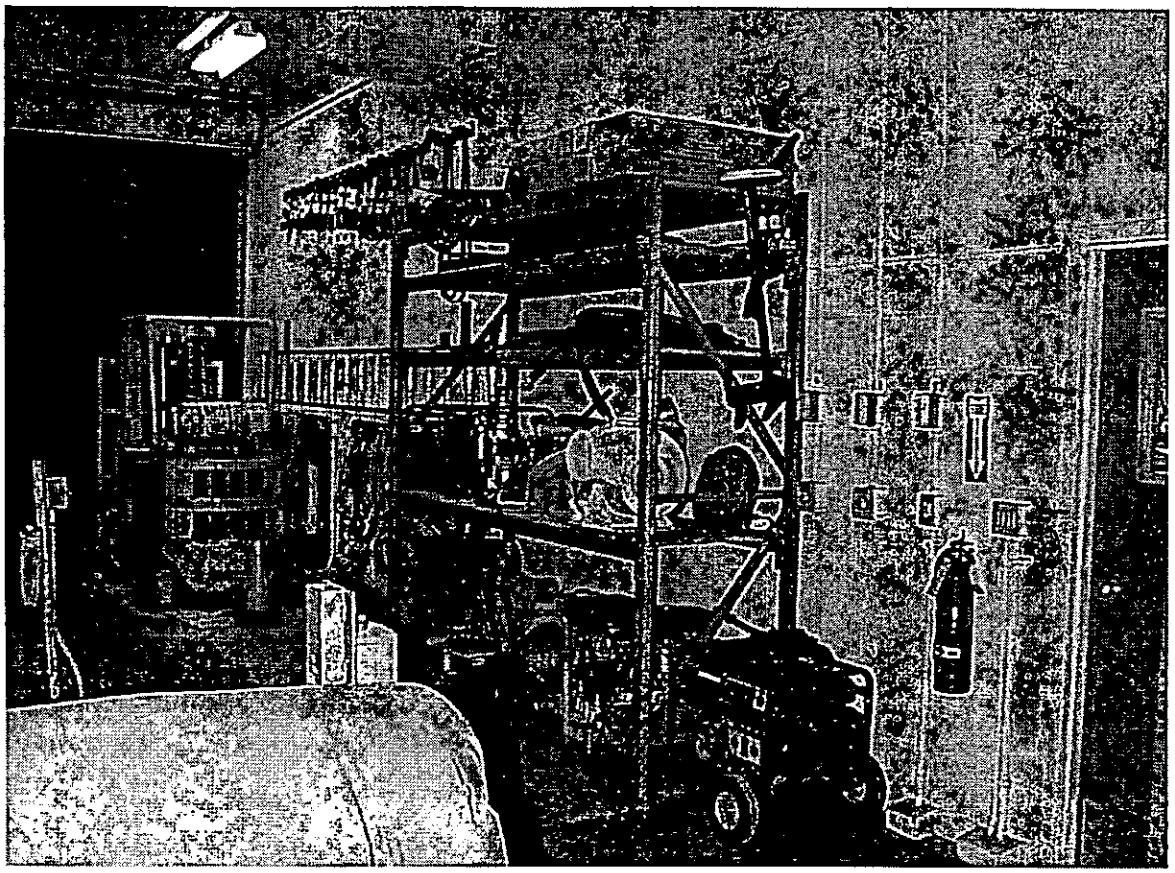

Photo 57: Operations and Maintenance Building. Blower Room. Storage shelves.

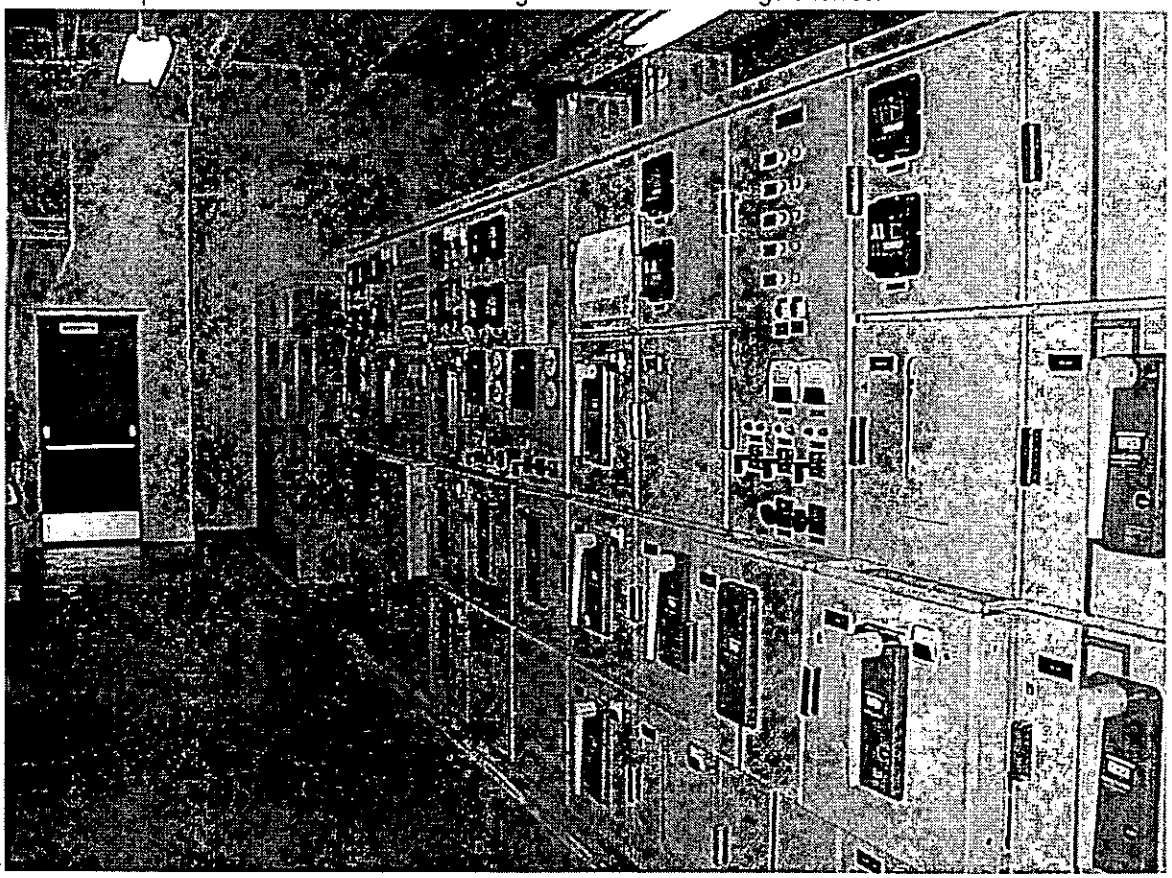

Photo 58: Operations and Maintenance Building. Electrical Room.

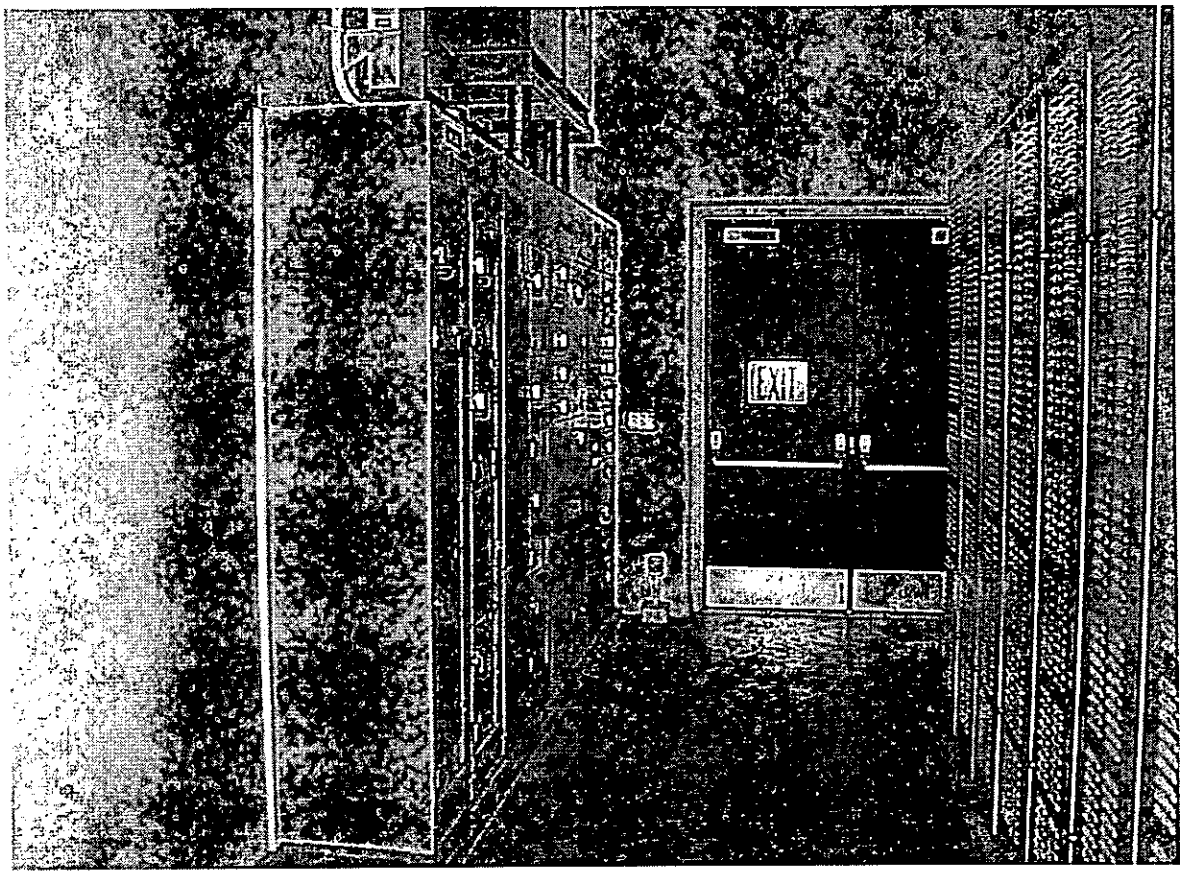

Photo 59: Operations and Maintenance Building. Electrical Room.

Photo 60: Operations and Maintenance Building. Electrical Room. Blower control panel not anchored to concrete.

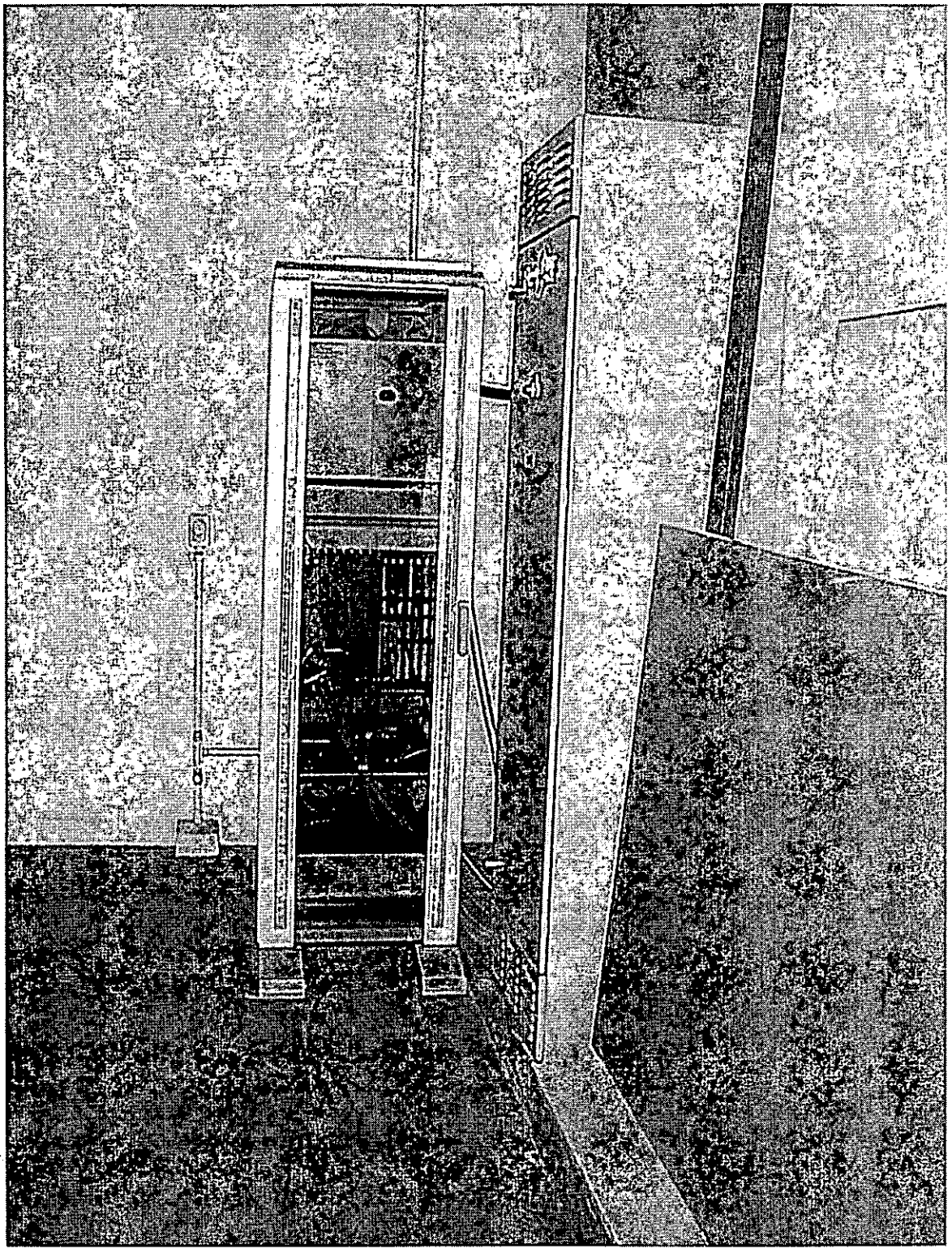

Photo 61: Operations and Maintenance Building. Electrical Room. Fiber optic control panel not anchored to concrete slab.

APPENDIX G

Public Outreach Materials

*** MEMORANDUM ***
May 11, 1998

To: Urban Services Committee

From: Neil Mann, Public Works Director

Subject: Status Report on Wastewater Utilities Master Plan Public Outreach Meeting #1

I. Issue Statement

This informational report is provided to summarize the outcome of the Wastewater Utilities Master Plan Community Outreach Meeting #1.

II. Background

The Wastewater Utilities Master Plan guides the management and implementation of facility improvements to the wastewater collection and treatment systems. The Plan is developed to support the City's Comprehensive Plan requirements for the logical extension of services within the Urban Growth Boundary (UGB).

The wastewater treatment plant master plan was last updated in 1973, and the latest collection system plan was completed in 1979. These plans are generally updated every 10 years to reflect changes in community planning, water quality regulations, and technological improvements in wastewater treatment. The plans are past due to be updated.

In December, 1996, CH2M Hill was hired to develop the Wastewater Utilities Master Plan update. Since that time, staff and the consultant have been developing the technical basis for the plan update. The completed plan will incorporate the following elements:

1. Anticipation of future water quality regulatory changes and identification of strategies to enable the City to maintain compliance with its DEQ/EPA wastewater discharge permit.

2. A hydraulic modeling tool to aid in the management and development of cost-effective improvements to the wastewater collection system.

3. A long-term "road map" of utility improvements that will respond to existing system deficiencies as well as accommodation of growth and development within the Urban Growth Boundary.

4. Responsiveness to community values and needs related to environmental quality and fiscal constraint.

At the March 16, 1998, City Council meeting, Public Works staff and its technical consultant, CH2M Hill, made a presentation to the Council on the process being used to update the City's Wastewater Utilities Master Plan. Various aspects of the plan update process were discussed, as well as the expected outcomes.

Staff informed the City Council that the underlying technical work had proceeded to the point where community input and advice on system improvement criteria, priorities, and

solutions to existing system deficiencies and future improvements were now desired. A public outreach process and schedule to accomplish this objective was shared with the Council. Attachment #1 is the public outreach schedule.

III. Discussion

Based on a positive affirmation from the City Council regarding the public outreach process and schedule, staff scheduled the first community outreach meeting for Wednesday, April 22, 1998.
This date was chosen to provide the least conflict with other City-sponsored meetings in late April.

Community Outreach Meeting #1 Summary

An extensive public outreach effort was undertaken to attract citizens to the meeting. Display advertisements were published on three dates in the *Gazette-Times* as well as a notice in the FYI section. An announcement was run on the TCI public access channel through April 22. Individual letters were sent to 95 different addresses, including homeowners associations, local consulting engineering firms that design improvements for new development in Corvallis, homebuilders and realtors associations, Corvallis Environmental Center, Benton County Health Department, City Council and Planning Commission members, major utility customers, and a number of citizens who have shown an interest in wastewater utility issues over the past several years. Attachment #2 is a compilation of the public outreach notification effort.

The community outreach meeting was held at the LaSells Stewart Center on April 22. Five citizens attended the meeting. Staff and CH2M Hill made a presentation and then concluded with asking citizens to review the various displays that were placed around the room regarding the wastewater master planning process, and to then write on flipcharts their issues, concerns, and questions. A copy of the meeting agenda, handouts, and wall displays is included in this memorandum as Attachment #3.

Two meeting attendees chose to provide staff with comments. One expressed desire for the City to make its plans comprehensive and do its part to minimize pollutants discharged to the Willamette River. The other citizen (the owner of Schaefer's Recreation located on NW 9th Street) expressed a desire for the City to eliminate the flooding and sewer backup problems their business experienced during the record rainfall and generalized flooding in late 1996 and early 1997.

Staff will incorporate these citizens' comments into the development of alternatives for the sewer piping upgrade/rehabilitation projects as well as upgrades to the wastewater treatment facilities. Staff had anticipated the NW 9th Street sewer concerns and has been developing alternatives and cost estimates for addressing the issues. Staff will contact each of the citizens who commented directly by letter to inform them of how their comments are being taken into consideration in the wastewater utilities master planning process.

Community Outreach Meeting #2 Schedule

In accordance with the Public Outreach Schedule approved by the City Council, staff proposes to hold the second community outreach meeting on Tuesday, June 9. The purpose of this meeting will be to bring to the community information about a draft capital improvements program (CIP) to address existing system deficiencies and accommodate growth and development. Included with this CIP will be planning level cost estimates for each project. Citizens will be asked to provide comments on their priorities on project timing/scheduling as well as any additional projects they may wish to have considered. A similar effort for the meeting notice and invitations as was used previously will be used for this public meeting.

After citizens' comments received at the June 9 meeting are incorporated into the draft master plan, staff will then proceed with the next steps on the master plan schedule. This includes another meeting with the Urban Service Committee (USC) in June, Planning Commission review of a draft Master Plan in July, with City Council review and possible adoption of a final Master Plan in July or August.

IV. Summary

The public comments that have been received will be incorporated into the development of policies, alternatives for correcting existing wastewater system deficiencies, and development of plans to meet future state regulatory authority water quality requirements. Staff will then proceed with development of a draft list of capital projects and associated cost estimates for inclusion in the draft Wastewater Utilities Master Plan. The draft list of capital projects will be presented to USC in June for discussion and action. The draft Master Plan will be reviewed by the Planning Commission in July, with City Council review and action in July or August, in accordance with the existing project schedule.

V. Recommendations

This report is for informational purposes. Staff requests the USC provide feedback on the proposed date for the second Community Outreach meeting, and an affirmation of the process and schedule for project completion.

TP/am
Attachments

Review and concur:

Jon S. Nelson

Jon S. Nelson
City Manager

Wednesday, April 15, 1998, Corvallis Gazette-Times, Corvallis, Ore.

You are invited to attend a community meeting about the City of Corvallis Wastewater Utility Master Plan

Wednesday, April 22 at 7:00 p.m.
LaSells Stewart Center, Agriculture Production Room
SW 26th Avenue & Western Boulevard

The Wastewater Master Plan guides the management of the wastewater collection and treatment systems for the City of Corvallis.

You will have an opportunity to learn about Corvallis' wastewater treatment infrastructure, as well as provide input and advice on system improvement criteria, priorities, and solutions to existing system deficiencies and future improvements.

Refreshments will be served.

For more information,
call Tom Penpraze at 757-6916

CORVALLIS
ENHANCING COMMUNITY LIVABILITY THROUGH IMPROVED WATER QUALITY

Advertisement run in G-T on

April 15, 19, 22.

F.Y.I.

Special events

TODAY

• **"Why People Don't Heal and How They Can,"** 12:15 to 1 p.m., Corvallis-Benton County Public Library, 645 N.W. Monroe Ave. Brown bag lecture "Love and Awakening Series" featuring Saren Nelson with discussion of book by Caroline Myss. Free. Sponsored by the Benton Center, library and the professional mental health community.

• **"Tips for Publishing,"** 7 p.m., Corvallis-Benton County Public Library, 645 N.W. Monroe Ave. Panel discussion by local writers. Information: 757-6926.

For complete information on these and other events see our Entertainer published every Friday.

Indoor park

TODAY

• **Tunison indoor park visitors day,** 10 to 11:30 a.m., Tunison Fire Station Community room, 364 S.W. Tunison Ave. Morning of play for children up to age 3, and siblings to age 5. Non-member visitors day is held the third Tuesday of each month. First visit is free, subsequent visits are $3.

Coast government

WEDNESDAY, APRIL 22

• **Lincoln County Board of Commissioners meeting,** 9:30 a.m., BOC meeting room, Lincoln County Courthouse, 225 W. Olive, Newport.

Open house

WEDNESDAY, APRIL 22

• **The Corvallis Aquatic Team** will hold an open house and level placement from 4 to 5 p.m. at Osborne Aquatic Center. Registration and information about the Mid-Valley BC Summer Swim League will be available. Swimmers ages 6 and over, with skills equivalent to SwimAmerica levels 7-10 are eligible to join the team. CAT provides swimming and fitness activities in a fun and healthy environment, supervised by an experienced professional coaching staff. For information, call 757-3902.

tional, 7 a.m., Burton's Sunnybrook Restaurant, 119 S.W. Third St. Information: 758-9542.

• **Le Tip,** 7:01 to 8:30 a.m., O'Callahans Restaurant, 1550 N.W. Ninth St. Weekly meeting of professional organization of men and women for the exchange of business tips. Information: 754-3417.

• **Altrusa International,** noon, Anderson, Searcy, Magedanz and Co., 310 N.W. Seventh St. Monthly board meeting. Information: 752-5248.

• **Philomath Lions Club,** noon, First Baptist Church, 335 S. 26th St., Philomath. Weekly luncheon meeting. Information: 453-5181, 929-6448.

• **Kiwanis Club,** noon, O'Callahan's Restaurant, 1550 N.W. Ninth St. Weekly member meeting. Lunch: $5.

• **Mid-Valley Singles,** 6 p.m., meet at Monteith Park. Meet together for a walk in the park. Call 928-1138 for directions.

• **Art discussion group,** 7 p.m., 825 N.W. 29th St. Open, weekly forum for discussing art and artists sponsored by American Association of University Women (AAUW). Information: 753-6241.

• **Corvallis Duplicate Bridge Club,** 7:30 p.m., Chintimini Senior Center, 2601 N.W. Taylor Ave. If you need a partner, call 745-3925. Cost: $3.

Government

WEDNESDAY, APRIL 22

• **Benton County Board of Commissioners,** 8 to 9:30 a.m., Benton Plaza (Large meeting room), 408 S.W. Madison Ave. interview with advisory board applicants.

• **Linn County Board of Commissioners,** 9:30 a.m., Linn County Courthouse, 300 Fourth Ave., Albany.

• **Corvallis Human Services Committee,** noon, Corvallis City Hall (Meeting room D), 501 S.W. Madison Ave.

• **Accountable Behavioral Health Alliance,** 4 to 5:30 p.m., ABHA Administrative OFfice, 209 N.W. Fifth St.

• **Corvallis Administrative Services Committee,** 5:30 p.m. Corvallis City Hall (Meeting room D), 501 S.W. Madison Ave.

• **Corvallis Planning Commission,** 7 p.m., Corvallis-Benton County Public Library (Meeting room), 645 N.W. Monroe Ave.

• **Philomath Transportation and Traffic Safety Committee,** 7 p.m., Philomath City Hall (Council chambers), 980 Applegate St., Philomath.

• **Corvallis Wastewater Facility Master Plan public meeting,** 7 p.m., LaSells Stewart Center (Agriculture Production Room), 875 S.W. 26th St. Community input is sought on such topics as wastewater system improvement criteria, priorities, solutions to existing system deficiencies and future improvements.

• **Albany City Council,** 7:15 p.m., Albany City Hall (Council chambers), 333 Broadalbin S.W., Albany.

Health

WEDNESDAY, APRIL 22

• **Stone Soup lunch,** 11:30 a.m. to 12:30 p.m., St. Mary's Catholic Church Gymnasium, 501 N.W. 25th St. Free meal for those in need.

• **Free medical clinic,** 7 to 8 p.m., Community Outreach, 128 S.W. Ninth St. Acute, outpatient care for low-income, uninsured patients with no other access to medical care. Information: 758-3000.

• **Lincoln health center,** 1 to 6 p.m., Lincoln Elementary School (southwest corner), 110 S.E. Alexander Ave. Appointments or walk-in care. Fees based on sliding scale basis; no one turned away if unable to pay. Information: 766-3546.

• **Birth control pill pickup for Family Planning patients,** 1 to 5 p.m., Benton County Health Department (Family Planning Clinic), 530 N.W. 27th St. Call for fee information: 757-6839.

• **HIV testing and counseling,** 1 to 4 p.m., Benton County Health Department (Family Planning Clinic), 530 N.W. 27th St. By appointment only. Sliding fee scale. Information: 757-6839.

• **Immunization clinic for children and travelers,** 8 a.m. to 4:30 p.m., Benton County Health Department, 530 N.W. 27th St. By appointment only. Information/appointments/fees: 757-6835.

• **Pregnancy testing and counseling,** 1 to 4 p.m., Benton County Health Department (Family Planning Clinic), 530 N.W. 27th St. By appointment only. Call for fee information; no one turned away if unable to pay. Information: 757-6839.

• **Pregnancy testing and counseling,** 10 a.m. to 4 p.m., Pregnancy Care Center, 2306 N.W. Kings Blvd. All services free. Information: 757-9645.

• **Sexually Transmitted Disease Clinic,** 9:30 a.m. to noon and 1 to 4 p.m., Benton County Health Department (Family Planning Clinic), 530 N.W. 27th St. By appointment only. Sliding fee scale. Information: 757-6839.

• **Women, Infants, Children (WIC) Program,** 8 a.m. to 4:30 p.m.,

ACTIVITY CALENDAR

April 20 - May 2, 1998

MONDAY, APRIL 20

- City Council - 12 noon and 7:00 pm - Central Park Municipal Building, 760 SW Madison

TUESDAY, APRIL 21

- Stormwater Planning Committee - 4:30 pm - Central Park Municipal Building, 760 SW Madison
- City Council (Planning Commission applicant interviews) - 5:30 pm - City Hall (Meeting Room "D"), 501 SW Madison
- Housing Work Group (Comprehensive Plan) - 7:00 pm - Chamber of Commerce Conference Room, 420 NW Second Street
- Land Use Work Group (Comprehensive Plan) - 7:00 pm - Chintimini Senior Center (Cline Room), 2601 NW Tyler
- Natural Resources Work Group (Comprehensive Plan) - 7:00 pm - Chintimini Senior Center, 2601 NW Tyler
- Policy Work Group (Comprehensive Plan) - 7:00 pm - Public Library (Conference Room "A"), 645 NW Monroe

WEDNESDAY, APRIL 22

- Human Services Committee - 12 noon - City Hall (Meeting Room "D"), 501 SW Madison
- Administrative Services Committee - 5:30 pm - City Hall (Meeting Room "D"), 501 SW Madison
- Planning Commission - 7:00 pm - Public Library Meeting Room, 501 SW Madison
- * Public Outreach meeting on the Wastewater Facility Master Plan - 7:00 pm - LaSells Stewart, 26th and Western

THURSDAY, APRIL 23

- Civic Beautification Commission - 8:00 am - Parks and Recreation Conference Room, 1310 SW Avery Park Drive
- City Council (Planning Commission applicant interviews) - 7:00 pm - City Hall (Meeting Room "D"), 501 SW Madison

From: Barbara HUGHES
To: LOUIE
Date: 4/13/98 4:12pm
Subject: CATV message

Kathy, please have the following announcement run on TCI's public access channel through the 22nd of April. Thanks!

Wastewater Master Plan Meeting
7 p.m. April 22, 1998
LaSells Stewart Center

Comments on system improvements, setting priorities, solutions to existing deficiencies, and future improvements are requested.

More Info: Public Works 757-6916

Economic Development Partnership
Attn: Doug Sweetland
420 NW 2nd Street
Corvallis, OR 97330

Corvallis Board of Realtors
777 NE 2nd Street
Corvallis, OR 97330

Corvallis Environmental Center
214 SW Monroe Ave
Corvallis, OR 97330

Kathleen Mulligan
100 Adams Hall
Oregon State University
Corvallis, OR 97331-2001

Home Builders Inc
2527 NW Walnut Blvd
Corvallis, OR 97330

Jane Thomas
Hewlett Packard
1000 NE Circle Blvd
Corvallis, OR 97330

Debbie Weaver
632 NW Third Street
Corvallis, OR 97330

Jim Moorefield
545 SW Second Street
Corvallis, OR 97333

Joe Malcolm
PO Box 1536
Corvallis, OR 97339

Pete Nelson
2605 SW Morris Avenue
Corvallis, OR 97333

Joel Palmer
612 NW 13th Street
Corvallis, OR 97330

Art Vosberg
6797 SW Winding Way
Corvallis, OR 97333

Paul E Hyde
221 NW 10th Street
Corvallis, OR 97330

ServiceMaster of Corvallis
Attn: Doug Crandall
2323 NW 9th Street
Corvallis, OR 97330

Schaefers' Recreation Equipment Co.
2319 NW 9th Street
Corvallis, OR 97330

David Evans & Associates
Attn: Rob Norton
1980 NW Ninth St., Suite 201
Corvallis, OR 97330

MSS Inc.
Attn: John Seaders
531 SW Fourth Street
Corvallis, OR 97333

Devco Engineering
Attn: Lyle Hutchins
245 NE Conifer Blvd
Corvallis, OR 97330

K&D Engineering
Attn: Howard Kraus
202 N.W. Hickory St.
Albany, OR 97321

Access Engineering
Attn: Thomas Bauer
310 NW 5th St., Suite 206
Corvallis, OR 97330

PacWest Engineering
Attn: Leonard Smith
123 Second Ave. SW
Albany, OR 97321

Mater Engineering
Scott Mater
P.O. Box 0
Corvallis, OR 97339

Century West Engineering
Ned Dempsey
825 NE Multnomah, Suite 425
Portland, OR 97232

CE/Western Engineering, Inc.
Robert T Reitmajer
1025 Bain SE
Albany, OR 97321

Balzhiser & Hubbard Engineers
James P. Krumsick
P.O. Box 10347
Eugene, OR 97440

West Yost and Associates
Walter J. Meyer
1800 Valley River Drive, Suite 250
Eugene, OR 97401

Ed Patton
39921 McDowell Creek Drive
Lebanon, OR 97355

CH2M Hill
Attn: Marty Warner
PO Box 428
Corvallis, OR 97339

Endex Engineering
223 NW Second Street
Corvallis, OR 97330

Environmental & Engineering Services Inc
687 NW 5th Street
Corvallis, OR 97330

Foundation Engineering Inc.
030 SW Philomath Blvd
Corvallis, OR 97333

Jay R. Lorenz
990 NW Highland Terrace
Corvallis, OR 97330

Southwood Engineering Corp
3301 Southwood Drive
Philomath, OR 97370

Marion Rose
149 NW Arrowood Circle
Corvallis, OR 97330

Hugh Crowe
c/o Web Press
439 SW 2nd Street
Corvallis, OR 97333

Lanny Zoeller
6225 NW Vineyard Drive
Corvallis, OR 97330

Tom Picht
PO Box 456
Corvallis, OR 97339-0456

Doug Squires
2800 NW 29th Street #10
Corvallis, OR 97330

Charles Honey
730 NW Lewisburg Avenue
Corvallis, OR 97330

Downtown Corvallis Association
460 SW Madison Avenue
Corvallis, OR 97333

Helen Berg
3645 NW Hayes
Corvallis, OR 97330

Patrick Peters
715 SW 13th Street, Apt A
Corvallis, OR 97333

Todd Lewis
06 NW 8th Street
Corvallis, OR 97330

Tony Howell
2030 SE DeBord Street
Corvallis, OR 97333

Bruce Sorte
526 NW 35th Street
Corvallis, OR 97330

Mary Christian
13 NW Kings Blvd
Corvallis, OR 97330

Guy Hendrix
1515 NW Hillcrest Drive
Corvallis, OR 97330

Tom Wogaman
2159 NW Beechwood Place
Corvallis, OR 97330

Betty Griffiths
3248 NW Taft Avenue
Corvallis, OR 97330

Ed Barlow-Pieterick
3542 NW Satinwood Street
Corvallis, OR 97330

Chamber of Commerce
420 NW 2nd
Corvallis, OR 97330

Benton County Health Dept
530 NW 27th
Corvallis, OR 97333

Avery Addition Neighborhood Association
Mark Schurman
835 SW 11th
Corvallis, OR 97333

Avery Addition Neighborhood Assn.
Judy Gibson
1002 SW 10th St
Corvallis, OR 97333

Avery Homestead Neighborhood Assn
Kirk Bailey
742 SW 4th Street
Corvallis, OR 97333

Avery Homestead Neighborhood Assn
Carolyn Ver Linden
644 SW 5th Street
Corvallis, OR 97333

Brooklane Area Neighborhood Assn.
Dick & Verna Gallagher
1635 SW Brooklane Drive
Corvallis, OR 97333

Brooklane Area Neighborhood Assn.
Lois & John Lacy
919 SW Brooklane Drive
Corvallis, OR 97333

Cedarhurst Neighborhood Assn.
Laura Evenson
3855 NW Van Buren
Corvallis, OR 97330

Hidden Creek Homeowners Assn.
Larry Christiansen
P. O. Box 213
Corvallis, OR 97339

Lancaster Bridge Neightorhood Assn
Jon Quinn
3434 "C" Bldg 2, Oxford Circle NW
Corvallis, OR 97330

Nash Avenue Neighborhood Assn.
Steve Lawton
4902 SW Nash Avenue
Corvallis, OR 97333

North College Hill Neighborhood Assn
Patricia Daniels
242 NW Kings Boulevard
Corvallis, OR 97330

NE Corvallis Neighborhood Assn.
Jennifer Ayotte
2455 NE Seavy Circle
Corvallis, OR 97330

NW Circle Blvd. Neighborhood Assn.
Elinor Griffiths
3535 NW Circle
Corvallis, OR 97330

NW Downtown Neighborhood Assn.
Todd Lewis
206 NW 8th Street
Corvallis, OR 97330

NW 10th St Traditional Neighborh'd Assn
Dian Cummings
1142 NW 10th Street
Corvallis, OR 97330

NW 10th St Traditional Neighborh'd Assn
Robyn Vega
1340 NW 10th Street
Corvallis, OR 97330

Orchard Downs Neighborhood Assn.
Davienne Jacobson
4115 NW Arrowood
Corvallis, OR 97330

Queens View Neighborhood Assn.
Norm Yates
4365 NW Queens Avenue
Corvallis, OR 97330

Queens View Neighborhood Assn.
Ron Shipman
3005 NW Princess
Corvallis, OR 97330

Satinwood Neighborhood Assn.
Carolyn Madsen
824 NW Ermine Place
Corvallis, OR 97330

Satinwood Neighborhood Assn.
Louis Ross
3518 NW Satinwood
Corvallis, OR 97330

Skyline Valley Neighorhood Assn.
Ralph Waldron
2610 NW Glenwood Drive
Corvallis, OR 97330

So Third Street Assessment Assn.
Herb Weisgerber
5100 NW Highway 99
Corvallis, OR 97330

Sunset Meadows Neighborhood Assn.
Kelley Wirth
4973 SW Aster St
Corvallis, OR 97333

Sunset Meadows Neighborhood Assn.
Lindsay Phoenix
4958 SW Roseberry Street
Corallis, OR 97333

Timberhill No. 1
Fred Dawson
2335 NW Monterey Drive
Corvallis, OR 97330

Timberhill SE 1st Addition
Gary Pond
1546 NW Terrace Green Place
Corvallis, OR 97330

Timber Ridge Neighborhood Assn.
Don Armstrong
2710 NW Garryanna
Corvallis, OR 97330

Tunison Park Neighborhood Assn.
Lynda Wolfenbarger
290 SW Tunison
Corvallis, OR 97333

West Corvallis Neighborhood Assn.
Don Bucella
3699 NW Tyler Place
Corvallis, OR 97330

West Corvallis Neighborhod Assn.
Steve Eckhart
3732 NW Tyler Place
Corvallis, OR 97330

West Hills Neighborhood Assn.
Heather Folts
4530 SW West Hills Road
Corvallis, OR 97333

West Hills Neighborhood Assn.
Vida Krantz
4020 SW West Hills Road
Corvallis, OR 97333

Karyle Butcher
2955 NW Royal Oaks Dr.
Corvallis, OR 97330

J. Patrick Lampton
The Inkwell
234 SW 3rd Street
Corvallis, OR 97333

Denis White
3775 NW Wisteria
Corvallis, OR 97330

Kirk Bailey
742 SW 4th Street
Corvallis, OR 97333

Mary Buckman
3248 SW Long Avenue
Corvallis, OR 97333

Kelley Panknin Wirth
973 SW Aster Street
Corvallis, OR 97333

Charles R. Gerke
3323 SW Chintimini Ave
Corvallis, OR 97333

Patricia Daniels
242 NW Kings Blvd
Corvallis, OR 97330

Bruce Osen
567 NW Terracegreen Pl
Corvallis, OR 97330

Michael Schweizer
976 NW Meadow Ridge Pl
Corvallis, OR 97330

F:\wwmasterplan mail list.wpd

Preliminary Public Outreach Overview

Wastewater Utility Master Plan

City of Corvallis

March, 1998

April, 1998	Community Meeting No. 1:	Utility Background Master Plan Objectives Problem(s) Understanding Input on Solutions/Criteria
May, 1998	Urban Services Briefing:	Community Feedback
June, 1998	Community Meeting No. 2:	Draft Improvements Projects Implementation Schedule/Costs Input on Priorities/Additional Projects
June, 1998	Urban Services Briefing:	Community Feedback Draft Capital Improvements Plan
July, 1998	Planning Commission:	Draft Master Plan review
July, 1998	City Council:	Draft Master Plan Review Final Master Plan Adoption

Community Comment and Suggestion Form

Corvallis Wastewater Utilities Master Plan
April 22, 1998

1. Master Plan Objective/Wastewater Utility Background:

2. Planning Issues

   - Land use and population:

   - Regulatory:

   - Environmental Adjacency:

3. Existing Collection System Assessment:

4. Wastewater Reclamation Facility Assessment:

Please leave at exit table or mail to: Public Works Department
P.O. Box 1083, Corvallis, OR 97339-1083

*** MEMORANDUM ***
June 16, 1998

TO: Urban Services Committee

FROM: Neil Mann, Public Works Director Neil

SUBJECT: Status Report on Wastewater Utilities Master Plan Community Outreach Meeting #2

I. Issue Statement

This informational report is provided to summarize the outcome of the Wastewater Utilities Master Plan Community Outreach Meeting #2.

II. Background

The Wastewater Utilities Master Plan (Master Plan) guides the management and implementation of facility improvements to the wastewater collection and treatment systems. The Plan is being developed to support the City's Comprehensive Plan for the logical extension of services within the Urban Growth Boundary (UGB).

The wastewater treatment master plan was last updated in 1973, and the latest collection system plan was completed in 1979. These plans are generally updated every 10 years to reflect changes in community planning, water quality regulations, and technological improvements in wastewater treatment. The current plans are outdated and need to be updated.

In December 1996, CH2M Hill was hired to develop the Wastewater Utilities Master Plan update. Since that time, staff and the consultant have been developing the technical basis for the plan update. The completed plan will incorporate the following elements:

1. Anticipation of future water quality regulation changes and identification of strategies to enable the City to maintain continued full compliance with the DEQ/EPA wastewater discharge permit.
2. A hydraulic computer modeling tool to aid in the management and development of cost-effective improvements to the wastewater collection system.
3. A long-term "road map" of utility improvements that will respond to existing system deficiencies as well as accommodation of future growth and development within the UGB.
4. Responsiveness to community values and needs related to environmental quality and fiscal constraint.

Staff briefed the City Council at their March 16, 1998, meeting on the process being used to update the Plan. Included in the briefing was a community outreach process and schedule that was endorsed by the Council. Attachment #1 is the community outreach schedule.

III. Discussion

Staff and the consultant held the first community outreach meeting on Wednesday, April 22, 1998. The Urban Services Committee was given a briefing on the community meeting at their May 26 meeting.

Staff then proposed, and the Committee endorsed, a June 9th date for the second community outreach meeting. The Urban Services Committee requested staff extend its public outreach efforts for the June 9th meeting to attract more citizens. The Committee asked staff to expand the mailing list and to add language to the mailings and advertisements regarding alternative methods of wastewater treatment and disinfection in an attempt to attract citizens interested in these aspects of the Master Plan.

Community Outreach Meeting #2 Summary

Considerable efforts were expended to attract citizens to the meeting. Display advertisements were published in the *Gazette-Times* as well as a notice in the FYI section. An announcement was run on the public access channel through June 9th. Individual letters were sent to 417 different addresses, including homeowner associations, local consulting engineering firms that design improvements for new development in Corvallis, homebuilders and realtors associations, Corvallis Environmental Center, Benton County Health Department, City Council and Planning Commission members, major utility customers, and a number of citizens who have shown interest in wastewater utility issues over the past several years. Attachment #2 is a compilation of the public outreach notification efforts.

The community outreach meeting was held at LaSells Stewart Center on June 9th. Five citizens attended the meeting. Staff and CH2M Hill made a presentation to the audience, and referred to the handout material and numerous displays that were placed around the room. The major topics discussed at the meeting included:

1. Collection system improvements
2. Treatment plant improvements, including a discussion of disinfection alternatives (such as using non-chlorine based methods) and treatment alternatives (such as wetlands)
3. The proposed implementation plan (which included a capital improvement schedule and estimated costs
4. Utility policy recommendations to be implemented to maintain and/or enhance the efficiency and effectiveness of the wastewater utility.

A copy of the meeting agenda, handouts, and wall displays is included in this memorandum as Attachment #3.

The citizens asked numerous questions throughout the presentation. Discussion centered around:

1. The adequacy of the Plan to serve west Corvallis
2. The evaluation of using wetlands as a means of wastewater treatment
3. The quality of biosolids (sludge) regarding metals content and the facts regarding the biosolids land application program
4. Collection system pipeline design criteria and its adequacy to prevent sewer system overflows except during unusual storm events
5. The definition of inflow/infiltration (I/I) and its impact on the collection system and treatment plant
6. Time lines, cost, and process for bringing a wastewater system capital improvement from the planning stage to project completion
7. Capital project financing

Copies of the draft capital improvements schedule and draft recommended utility policies are included in Attachment # 3.

Staff responded to all questions at the meeting. Overall, positive support was expressed for the Plan's major elements that support protection of community public health and the environment, and the means of accomplishing these goals through effective utility policies and the capital improvement program.

IV. Next Steps

Staff will integrate citizen comments and suggestions from the community outreach meetings into the draft master plan and then proceed with producing a draft master plan for review by citizens, the Planning Commission, and the City Council.

Following the interaction with the Urban Services Committee on June 22nd, the draft master plan will be compiled and printed with copies forwarded to the City Council and the Planning Commission. Copies will also be placed in the Library, City Hall, and at the Public Works office for citizen review.

The Planning Commission is scheduled to review the Wastewater Utilities Master Plan at the July 15 Planning Commission meeting, with City Council review and possible adoption at the end of July or August, 1998.

V. Recommendations

This report is for informational purposes. Staff requests that the USC provide feedback on the process to date, the implementation (capital improvements) plan elements, and the recommended wastewater utility policies.

Review and concur:

Jon S. Nelson
City Manager

TP/bh

Preliminary Public Outreach Overview

Wastewater Utility Master Plan

City of Corvallis

March, 1998

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July, 1998	Planning Commission:	Draft Master Plan review
July, 1998	City Council:	Draft Master Plan Review Final Master Plan Adoption

Attachment # 2

CORVALLIS
ENHANCING COMMUNITY LIVABILITY

Public Works Department
1245 NE 3rd Street
P.O. Box 1083
Corvallis, OR 97339-1083
(541) 757-6916
FAX: (541) 757-6920
TTY: (541) 757-6477
E-MAIL: public.works@ci.corvallis.or.us

May 26, 1998

Re: City of Corvallis Wastewater Utility Master Plan Update Meeting #2

You are invited to attend the second public meeting about the City of Corvallis Wastewater Utility Master Plan.

The Wastewater Utility Master Plan guides the management of the City's wastewater collection and treatment systems. The plan is developed in support of the City's Comprehensive Plan requirements for the logical extension of services within the City's Urban Growth Boundary.

The Wastewater Treatment Master Plan was last updated in 1973, and the collection (piping) system plan was completed in 1979. These plans are typically updated every 10 years to reflect changes in community planning, water quality regulations, and technological improvements.

In December, 1996, the engineering consulting firm CH2M Hill was hired to develop the Master Plan update. Since that time, City staff and the consultant have been gathering information and developing the technical basis for the Master Plan update. The completed plan will incorporate the following major elements:

- Integration with the City's ongoing combined sewer overflow remediation project
- Anticipation of future water quality regulations
- Responsiveness to community values and needs related to environmental quality and fiscal constraint
- A long-term "road map" of utility improvements that will respond to existing system deficiencies as well as accommodate growth and development within the Urban Growth Boundary service area.

At this meeting, City staff will

- Present a draft schedule of capital improvement projects
- Present a draft implementation schedule and project cost estimates
- Seek input on project priorities and any additional projects the public feels may be needed

The second community outreach meeting has been scheduled for Tuesday, **June 9, starting at 7 p.m., at the LaSells Stewart Center, Agriculture Production Room**, located at 26th and Western Boulevard on the OSU campus. Refreshments will be served.

I hope you can attend the meeting so we may hear your thoughts and ideas on the future of the Corvallis wastewater system. If you have any questions, please call me at 757-6916.

Sincerely,

Tom Penpraze
Utilities Division Manager

TP/bh

Economic Development Partnership
Attn: Doug Sweetland
420 NW 2nd Street
Corvallis, OR 97330

Corvallis Board of Realtors
777 NE 2nd Street
Corvallis, OR 97330

Corvallis Environmental Center
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PacWest Engineering
Attn: Leonard Smith
1600 9th Street SE
Albany, OR 97321

Mater Engineering
Scott Mater
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825 NE Multnomah, Suite 425
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Robert T Reitmajer
1025 Bain SE
Albany, OR 97321

Balzhiser & Hubbard Engineers
James P. Krumsick
P.O. Box 10347
Eugene, OR 97440

West Yost and Associates
Walter J. Meyer
1800 Valley River Drive, Suite 250
Eugene, OR 97401

Ed Patton
39921 McDowell Creek Drive
Lebanon, OR 97355

CH2M Hill
Attn: Ray Topping
PO Box 428
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Endex Engineering
223 NW Second Street
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oundation Engineering Inc.
30 SW Philomath Blvd
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Jay R. Lorenz
990 NW Highland Terrace
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Southwood Engineering Corp
3301 Southwood Drive
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May 29, 1998

Re: City of Corvallis Wastewater Utility Master Plan Update Meeting #2

You are invited to attend the second public meeting about the City of Corvallis Wastewater Utility Master Plan.

The Wastewater Utility Master Plan guides the management of the City's wastewater collection and treatment systems. The plan is developed in support of the City's Comprehensive Plan requirements for the logical extension of services within the City's Urban Growth Boundary.

The Wastewater Treatment Master Plan was last updated in 1973, and the collection (piping) system plan was completed in 1979. These plans are typically updated every 10 years to reflect changes in community planning, water quality regulations, and technological improvements.

In December, 1996, the engineering consulting firm CH2M Hill was hired to develop the Master Plan update. Since that time, City staff and the consultant have been gathering information and developing the technical basis for the Master Plan update. The completed plan will incorporate the following major elements:

- Anticipation of future water quality regulations and how best to meet them
- Responsiveness to community values and needs related to environmental quality and fiscal constraint
- A long-term "road map" of utility improvements that will protect public health by responding to existing system deficiencies as well as accommodating growth and development within the Urban Growth Boundary service area

At this meeting, City staff will

- Present an evaluation of alternative methods of wastewater treatment, including the use of wetlands, wastewater reuse, and dispersed treatment
- Discuss the feasibility of alternative methods to using chlorine for wastewater disinfection
- Present a draft schedule of capital improvement projects
- Present a draft implementation schedule and project cost estimates
- Seek input on project priorities and any additional projects our customers feel should be considered to adequately protect the public health of the community

The meeting has been scheduled for Tuesday, **June 9, starting at 7 p.m., at the LaSells Stewart Center, Agriculture Production Room**, located at 26th and Western Boulevard on the OSU campus. Refreshments will be served.

I hope you can attend the meeting so we may hear your thoughts and ideas on the future of the Corvallis wastewater system. If you have any questions, please call me at 757-6916.

Sincerely,

Tom Penpraze
Utilities Division Manager

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Corvallis, OR 97330

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c/o Police Dept.
P O Box 1083
Corvallis, OR 97339

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City of Philomath
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Sue Johnston, Chair
PO Box 863
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Sue Ann Jenkins
819 NW 35th Street
Corvallis, OR 97330

Susan Fairchild
2235 NW 11th Street
Corvallis, OR 97330

Susan Christie
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Corvallis, OR 97330

Susan Soukop
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3328 NW Firwood Drive
Corvallis, OR 97330

Todd Pawloski
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Corvallis, OR 97330

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Wayne C. Huber
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Corvallis, OR 97330

West Hills Terrace Neighborhood Assc.
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Corvallis, OR 97333

William J. Hoffman
2120 NW Evergreen
Corvallis, OR 97330

Wilson School Neighborhood Assn.
455 NW Ponderosa
Corvallis, OR 97330

Yaney MacIver
5370 NW Lawrence
Corvallis, OR 97330

Zach Payne
3230 NW 60th Street
Corvallis, OR 97330

Corvallis Gazette-Times, Corvallis, Ore., Sunday, June 7, 1998

If you care about:

- How wastewater is treated in your community
- Ensuring Willamette River water quality protection
- Long-term protection of public health and the environment

Plan to attend an important Community Meeting about the City of Corvallis Wastewater Utility Master Plan

Tuesday, June 9, 1998, at 7:00 p.m.
LaSells Stewart Center, Agriculture Production Room
SW 26th Avenue & Western Boulevard

The Wastewater Master Plan guides the management of the wastewater collection and treatment systems for the City of Corvallis.

At this meeting, City staff will:

- Present an evaluation of alternative methods of wastewater treatment (including the use of wetlands, wastewater reuse, and dispersed treatment)
- Discuss alternatives to using chlorine for wastewater disinfection
- Present a draft schedule of capital improvement projects
- Present a draft implementation schedule & project cost estimates
- Seek input on project priorities and any additional projects the public feels should be considered to adequately protect the public health of the community

Refreshments will be served.

For more information,
call Tom Penpraze at 757-6916

CORVALLIS
ENHANCING COMMUNITY LIVABILITY
THROUGH IMPROVED WATER QUALITY

Advertisement run in the G-T on
June 3,7, and 9.

CORVALLIS
ENHANCING COMMUNITY LIVABILITY

Public Works Department
1245 NE 3rd Street
P.O. Box 1083
Corvallis, OR 97339-1083
(541) 757-6916
FAX: (541) 757-6920
TTY: (541) 757-6477
E-MAIL: public.works@ci.corvallis.or.us

May 20, 1998

Gazette Times
FYI News Coordinator
PO Box 368
Corvallis, OR 97339

Please publish the following information in the **F.Y.I.** column on Monday, June 8, 1998, in place of the information sent to you previously on the same meeting.

Wastewater Utility Master Plan Update Meeting

A public meeting will be held on the Wastewater Utility Master Plan Update on Tuesday, June 9, 1998, at 7:00 p.m., at the LaSells Stewart Center, Agriculture Production Room, 26th Avenue and Western Boulevard.

City staff will:

- Present an evaluation of alternative methods of wastewater treatment (including the use of wetlands, wastewater reuse, and dispersed treatment)
- Discuss alternatives to using chlorine for wastewater disinfection
- Present a draft schedule of capital improvement projects
- Present a draft implementation schedule & project cost estimates
- Seek input on project priorities and any additional projects the public feels should be considered to adequately protect the public health of the community.

Refreshments will be served.

For more information, contact Tom Penpraze, Utilities Division Manager, City of Corvallis Public Works Department, at 757-6916.

Thank you for your assistance.

Barbara Hughes
Public Works Secretary

From: Barbara HUGHES
To: LOUIE
Subject: CATV message

Kathy, please replace this announcement I sent you last week:

Wastewater Master Plan Meeting
7 p.m. June 9, 1998
LaSells Stewart Center

City staff will present proposed improvements, implementation schedule and project cost estimates. Public input is requested.

More info: Public Works 757-6916

with the following, and run through June 9... thanks!

Wastewater Master Plan Meeting
7 p.m. June 9, 1998
LaSells Stewart Center

Topics include wastewater treatment processes, Willamette River water quality, and proposed projects to protect the public health and environment. Public input is requested. More info: Public Works 757-6916

Community Comment and Suggestion Form

Corvallis Wastewater Utilities Master Plan
June 9, 1998

1. Master Plan Objectives:

2. Planning Criteria/System Deficiencies:

3. Collection System Improvements:

4. Wastewater Reclamation Plant (WWRP) Assessment:

5. Implementation Plan:

Please leave at exit table or mail to: Public Works Department
P.O. Box 1083, Corvallis, OR 97339-1083

PDX\AG0609_DOC

APPENDIX H

Abbreviations and Acronyms

ABF	average base flow
ACWA	Association of Clean Water Agencies
BOD	biochemical oxygen demand
$CBOD_5$	5-day carbonaceous biochemical oxygen demand
CFR	code of federal regulations
COD	chemical oxygen demand
Corps	U.S. Army Corps of Engineers
CSO	combined sewer overflow
DEQ	Oregon Department Of Environmental Quality
DWMM	dry weather maximum month
EPA	U.S Environmental Protection Agency
EQC	Environmental Quality Board
FSLs	facultative sludge lagoons
gpd/capita	gallons per day per capita
HGL	hydraulic grade line
I&C	instrument and control
I/I	infiltration and inflow
MCRT	mean cell residence time
MG	million gallons
mg/L	milligram(s) per liter
mgd	million gallons per day
NPDES	National Pollution Discharge Elimination System
NURP	National Urban Runoff Program
O&M	operation and maintenance; operations and maintenance
OAR	Oregon Administrative Rule

OWRD	Oregon Water Resources Division
pH	Hydrogen (ion) concentration
ppd/capita	pounds per day per capita
PWD	public works department
RDII	rainfall-derived infiltration and inflow
SCADA	supervisory control and data acquisition
SFO	stipulation and final order
SLR	solids loading rate
SSES	sanitary sewer evaluation study
TF/AS	trickling filter/activated sludge
TM	technical memoranda
TMDL	total maximum daily load
T_s	instream temperature
TSS	total suspended solids
USC	Council Urban Services Committee
UV	ultraviolet
VS	volatile solids
WAS	waste activated sludge
WIC	water infrastructure committee
WWMM	wet weather maximum month
WWRP	wastewater reclamation plant
WWTF	wet weather treatment facility
ZID	zone of initial dilution

owners for their repair is approximately $20/lineal foot of lateral replaced. For the purposes of estimating, an 80-foot lateral replacement was assumed.

Open Plumbing

Several open plumbing defects were identified on private property. These defects consist of open cleanouts or broken pipe. A number of defective cleanouts were discovered during the smoke testing program. Most of the cleanout defects are located on private property. These cleanouts should be repaired since they can be an entrance point for rodents into the sewer system. Rehabilitation of open cleanouts consists of replacing missing or broken covers. An odd cover configuration may complicate the rehabilitation effort. Excavation and replacement of a portion of the lateral may be required if the stem pipe of the cleanout is damaged. The City's costs for cleanout repair consists of notifying property owners of the needed repairs. This notification and follow-up repair verification is estimated at $100 per defect. The property owner's repair cost would be approximately $50.

Catch Basins

The rehabilitation procedure for area and yard drains located on private property involves disconnection from the sanitary system and redirection of the drainage. Disconnection includes excavating at the lateral, removing the drain connection, and repairing lateral piping. The drain piping must then be redirected either to a storm drain, the street gutter or, if possible, the landscape or nearby ditch. Several factors that affect area drain disconnection are access to the facility, proximity to other utilities and structures, and available storm drain facilities. Costs for rehabilitation of area drains consist of the City's cost to notify property owners of the required repair. This notification and follow-up repair verification process is estimated at $100 per defect.

The rehabilitation procedure for catch basins located in streets will require disconnection from the sanitary sewer and reconnection to a storm drainage system. Disconnection includes excavation at the point where the catch basin discharged to the sanitary sewer so that this connection may be capped permanently. Cost for this rehabilitation will vary widely because of the varying distances between the catch basin and the closest storm sewer. The cost to repair this defect was assumed to be approximately $5,000 each.

Downspouts

Downspout disconnection involves separating the downspout from the sanitary system, capping the pipe connection, and installing an elbow. This procedure may require installation of lateral storm drain piping to direct the stormwater from the building to a storm drain. Rerouting the rain gutters to the landscape or nearby ditch will minimize the impact of the storm drainage pipe sizing. The City's cost for downspout repairs consists of notifying property owners to disconnect or repair the downspout. This notification and follow-up repair verification is estimated at $100 per defect. The property owner's cost for this repair is approximately $500.

Rehabilitation Plan

The rehabilitation plan has the following components:

- Manhole raising to grade
- Manhole sealing
- Catch basin disconnects
- Mainline spot repairs
- Service lateral repairs
- Open plumbing repairs
- Downspout disconnects

The recommended rehabilitation program summarized in Table 2 would cost the City and property owners approximately $395,000 if implemented in its entirety. The City may elect to perform those repairs that will have the greatest impact on reducing I&I within the basin. It should be noted that over 50 percent of I&I identified by this program was contributed by defects located on private property. The cost for removal of this flow is about $236,000 or 60 percent of the total cost for the program. The manhole rehabilitation appears to be the most cost-effective repair that would eliminate substantial I&I. The cost of the manhole rehabilitation would be about $13,000 or 3 percent of the program cost.

Further physical inspection of manholes within the basin should be completed prior to starting the manhole rehabilitation program. After a complete inspection is performed, the defective manholes could be prioritized according to the severity of the I&I problem. The inspections should be performed during the wet weather season when groundwater levels are highest.

Closed circuit television (CCTV) inspection of sewer mains that have defects discovered during the smoke test program should be performed. The inspection should be performed prior to making plans for any spot repairs on that line. CCTV inspection may reveal that there are more defects than where located during the smoke testing, and that another type of repair may be more cost-effective.

Conclusion

The results presented in this memorandum identify 265 defects that were located and logged in the Garfield Basin. The estimated I&I contributed by these defects would be approximately 600,000 gallons during a 5 year–24 hour winter rain event. Flows measured during the 1993/94 flow monitoring season indicate that there is a much larger quantity of I&I entering the sewers in the Garfield Basin. Further investigations such as CCTV inspection and manhole physical surveys would undoubtedly reveal more defects in the sewer system. Any further pipeline and manhole inspections should be conducted during periods of high groundwater. Table 3 lists defects with priority levels of 1-5. Defects with a priority of 1 should be considered for rehabilitation or repair regardless of their cost-effectiveness. These defects include catchbasins connected to the sanitary sewer, mainline defects that show sink holes developing and creeks leaking into the sewer. Defects with second priority are

mainline defects only. The Number 3 priority defects are primarily manhole defects. The Number 4 and 5 priority defects are private property defects.

Table 1

City of Corvallis

Smoke Test Program

	DEFECT FORM #	TEST BASIN #	LOCATION	MANHOLE DEFECT	CATCH BASIN CONNECTED	MAINLINE DEFECT	LATERAL DEFECT	OPEN PLUMBING	DOWN-SPOUT	OTHER	COMMENTS
1	2	1	1900 14th St				x				
2	3	1	1910 14th St				x				
3	5	1	1925 14th St	x							Buried mh
4	7	1	1310 Cleveland St						x		All downspouts smoked
5	9	1	1417 Menlo St						x		All downspouts smoked
6	11	1	1775 Menlo St					x			Crawl space smoked
7	12	1	1525 Menlo St						x		
8	14	1	NE corner Cleveland & Garyanna		x						
9	15	1	NW corner Cleveland & Garyanna		x						
10	16	1	SE corner Cleveland & Garyanna		x						
11	18	1	1530 Menlo St				x	x			
12	19	1	1920 Menlo St				x				
13	20	1	1852 Garryanna St				x				
14	22	1	1715 17th St				x				
15	23	1	SE corner 17th & Arthur St	x							
16	24	1	1625 17th St			x	x				
17	25	1	1620 Garfield St		x						
18	26	1	Arthur St		x						Near SS mh 46-77-43
19	27	1	1710 Menlo St				x				2 places
20	28	1	1627 Garfield St						x		
21	32	1	1640 17th St				x				
22	34	1	1865 Arthur Circle				x				
23	35	1	1855 Authur Circle							C.O. acting as CB	
24	36	1	1900 Arthur Circle	x							
25	37	1	1650 Authur Circle				x				
26	38	1	1885 Authur Circle							C.O. leaking in St	
27	39	1	1885 Authur Circle				x				
28	40	1	1875 Authur Circle				x				
29	42	1	1795 Authur Circle	x							
30	43	1	1715 Aurthur Circle				x				
31	46	1	2021 Garfield St				x				C.O. leaking
32	49	1	SE corner Greely & 15th St		x						
33	50	1	1430 Greely St				x				
34	51	1	NE corner Greely & 15th St		x						SS mh 6 ft south smoked
35	53	1	1555 14th St						x		
36	54	1	1432 13th St	x							
37	56	1	1625 Highland St					x			
38	57	1	1696 Division St						x		
39	59	1	935 Hobart St				x				betw apt. 5 & 6
40	61	1	1840 Division St				x				
41	62	1	1910 Division St			x					sinkholes developing
42	63	1	1910 Division St				x				
43	64	1	1914 Division St						x		
44	65	1	978 Cleveland Pl	x							private mh
45	66	1	1820 Division St				x				
46	67	1	Corner Division & Division Pl		xxx						NW,NE & SE corners
47	69	1	1690 Highland St						x		
48	70	1	1920 Highland - School	x							
49	71	1	" " "						x		
50	72	1	1641 11th St				x				
51	73	1	1665 11th St						x		
52	74	1	1717 11th St.						x		
53	75	1	1727 11th St						x		
54	76	1	1737 11th St						x		
55	78	1	1920 Highland - School	x							
56	79	1	" " "		x						
57	80	1	" " "				x		x		

Table 1

City of Corvallis

Smoke Test Program

	DEFECT FORM #	TEST BASIN #	LOCATION	MANHOLE DEFECT	CATCH BASIN CONNECTED	MAINLINE DEFECT	LATERAL DEFECT	OPEN PLUMBING	DOWN-SPOUT	OTHER	COMMENTS
58	83	1	1616 Dixon St	x							
59	84	1	1736 Dixon St				x				
60	85	1	1610 Dixon St				x		x		
61	87	1	1420 Highland St	x							mh in gutter recieves inflow
62	88	1	1430 Division St	xx							2 mh leaking
63	89	1	900 Hayes St			x					
64	90	1	900 Hayes St	x							
65	91	1	900 Hayes St								SS mh smoked
66	94	1	4600 Kings Blvd	x							
67	96	2	925 Sycamore St				x				
68	97	2	923 Sycamore St				x				
69	98	2	927 Sycamore St	x							SS mh smoked
70	99	2	934 Sycamore St						x		
71	100	2	934 Sycamore St					x			smoke around C.O.
72	101	2	938 Sycamore St	x							private mh
73	103	2	973 Sycamore St				x				
74	104	2	973 Sycamore St			x					smoked at base of pole
75	105	2	2160 Janssen St				x				
76	107	2	972 Sequoia St				x				
77	108	2	978 Sequoia St				x				
78	109	2	982 Sequoia St				x				
79	110	2	988 Sequoia St				x				
80	111	2	981 Sequoia St						x		
81	112	2	971 Sequoia St						x		
82	113	2	975 Sequoia St				x				
83	114	2	966 Sequoia St					x			broken C.O. cap
84	115	2	960 Sequoia St				x				
85	116	2	955 Sequoia St				x		x		
86	117	2	969 Sequoia St				x		x		
87	118	2	971 Sequoia St				x				
88	120	2	939 Sequoia St				x				
89	121	2	944 Sequoia St				x				
90	123	2	2321 Highland St					x			
91	124	2	2327 Highland St	x			x				
92	125	2	2340 Wayland				x				
93	127	2	987 Cypress St				x				
94	130	2	580 Linden Ave				x	x			
95	131	2	612 Linden Ave		x						
96	132	2	655 Linden Ave				x				
97	133	2	670 Linden Ave				x				
98	134	2	660 Linden Ave				x				
99	137	2	952 Oak St	x							
100	138	2	856 Oak St				x				
101	139	2	828 Oak St				x				
102	140	2	805 Oak St				x				
103	143	2	810 Sundance Ct	xx							mhs act as catchbasins
104	147	2	2901 Satinwood	xx							buried mhs
105	149	2	924 Meadowridge				x				
106	151	2	992 Meadowridge Pl			x					creek leaking into sewer
107	152	2	North of 992 Meadowridge Pl	x		x					
108	155	2	857 Chipmunk Ave				x				
109	158	2	782 Deer Pl				x				
110	162	2	658 Survista Ave	x							
111	163	2	590 Survista Ave	x							
112	165	2	534 Maxine	x							
113	167	2	2810 Stewart Ave				x				
114	168	2	2765 Stewart Ave					x			

Table I

City of Corvallis

Smoke Test Program

	DEFECT FORM #	TEST BASIN #	LOCATION	MANHOLE DEFECT	CATCH BASIN CONNECTED	MAINLINE DEFECT	LATERAL DEFECT	OPEN PLUMBING	DOWN-SPOUT	OTHER	COMMENTS
115	169	2	2830 Stewart Ave				x				
116	170	2	2770 Stewart Ave				x				
117	171	2	560 Hemlock Ave	x							
118	172	2	2735 Stewart Ave				x				
119	173	2	2745 Stewart Ave				x				
120	174	2	2765 Stewart Ave		x						
121	176	2	520 Hemlock Ave.				x				
122	177	2	Intersection of Hemlock & Bryant		xxx						all 3 catch basins smoked
123	179	2	465 Ponderosa Ave	x							
124	180	2	435 Ponderosa Ave				x				
125	181	2	470 Conifer Ave					x			
126	183	2	2921 Conifer Pl				x	x			
127	184	2	2951 Conifer Pl		x		x				
128	185	2	2918 Conifer Pl				x				
129	187	2	3220 9th St				x				
130	188	2	3220 9th St	x							
131	189	2	3124 9th St				x				
132	190	2	3008 9th St	x							
133	191	2	2950 9th St				x				
134	193	2	Intersection of 9th & Conifer	x							
135	194	2	390 Conifer Ave	x		x					
136	195	2	2950 9th St	x							
137	197	2	410 Pondersosa Ave					x	x		
138	198	2	430 Ponderosa Ave						x		
139	199	2	440 Ponderosa Ave						x		
140	200	2	2641 9th St				x				
141	202	2	390 Maxine Ave				x				
142	203	2	2939 Conifer Pl					x			
143	204	2	3045 Autumn St	x							
144	206	2	2300 9th St - Wendys		x						
145	207	2	2350 9th St - video store		x		x				
146	208	2	NW corner of 9th & Linden St		x						
147	209	2	Intersection of 9th & Sequoia		x						Storm sewer mh smoked
148	210	2	923 Sequoia St				x				
149	211	2	NW corner of 9th & Oak St		x						
150	212	2	400 Walnut Blvd - Enterprize Rental				x				
151	213	2	2503 9th St - Arbys		x						
152	215	2	1895 9th St - Plaza 9				x				
153	216	2	NW corner of 9th & Circle		x						
154	217	2	2123 9th St - Highland Bowl		x						
155	218	2	9th & Sycamore St	x							Storm sewer mh smoked
156	220	2	1800 9th St					x			
157	221	2	1895 9th St - Plaza 9				x				
158	222	2	560 Walnut Blvd					x			
159	224	3	2525 27th St				x				
160	225	3	2550 27th St	x				x			
161	226	3	2589 Walnut Blvd	x							buried mh
162	227	3	2650 Roosevelt	x							
163	228	3	2695 Roosevelt				x				
164	231	3	2154 27th Pl						x		
165	233	3	2620 Elmwood St					x			
166	234	3	2717 Elmwood St						x		
167	235	3	2803 Elmwood St				x		x		
168	236	3	2813 Elmwood St				x		x		
169	238	3	2050 23rd St		x						private area drain
170	239	3	2125 25th St	x							
171	240	3	2140 25th Pl				x				

Table 1

City of Corvallis

Smoke Test Program

	DEFECT FORM #	TEST BASIN #	LOCATION	MANHOLE DEFECT	CATCH BASIN CONNECTED	MAINLINE DEFECT	LATERAL DEFECT	OPEN PLUMBING	DOWN-SPOUT	OTHER	COMMENTS
172	242	3	2230 29th St				x				
173	245	3	4300 23rd St	x							
174	246	3	9800 23rd St	x							
175	247	3	2276 23rd St	x							
176	250	3	2115 Elmwood St	x							
177	251	3	2135 Elmwood St				x				
178	252	3	2115 Elmwood St	x							buried mh
179	254	3	2683 Circle Blvd				x				
180	255	3	1925 27th St			x		x			
181	256	3	1820 27th St						x		
182	258	3	2615 Circle Blvd						x		
183	259	3	2525 Pendleton St				x				
184	260	3	1930 Kings Blvd	x							in street
185	261	3	1930 Kings Blvd	x							backyard
186	262	3	2130 Elmwood Pl	x							
187	264	3	2310 Elmwood Pl	x							
188	265	3	1956 23rd St			x					numerous leaks
189	266	3	1940 25th St				x				
190	267	3	1922 25th St				x				
	TOTAL			48	26	9	83	17	29	2	214

Table 2

CITY OF CORVALLIS

SMOKE TEST PROGRAM SUMMARY

DEFECT DESCRIPTION	NUMBER OF DEFECTS (actual)	PEAK I&I CONTRIBUTION (gal/day) [9]	TOTAL CONTRIBUTION (gal/day)	REPAIR METHOD	REPAIR COST - CITY PORTION ONLY [10]	REPAIR COST - PRIVATE PROPERTY ONLY	TOTAL COST ($)	CONSTRUCTION COST/GAL. REMOVED ($)	TREATMENT COST/YR ($) [11] ASSUME 45 IN/YR RAIN
MANHOLE - casting leak [1]	48	3,672	176,256	non-shrink grout	200	0	9,600	0.06	10,171
MANHOLE - invert or joint leak [2]	10	2,880	28,800	non-shrink or chemical grout	300	0	3,000	0.11	1,662
CATCH BASIN - direct connection [3]	7	8,583	60,081	connect to storm sewer	5,000	0	35,000	1.72	3,467
CATCH BASIN - indirect connection [4]	21	1,440	30,240	spot repair	2,500	0	52,500	1.74	1,745
MAINLINE SEWER [5]	9	1,440	12,960	spot repair	2,500	0	22,500	1.74	748
PRIVATE SEWER LATERAL [6]	124	1,440	178,560	lateral replacement	100	1,200	161,200	0.90	10,304
OPEN PLUMBING [7]	17	1	17	repair defect	100	50	2,550	150.00	3
DOWNSPOUT [8]	29	3,177	92,133	route to street or storm system	100	500	17,400	0.19	5,317
SUBTOTAL							303,750		
CONSTRUCTION CONTINGENCY 30%							91,125		
TOTAL	265		579,000				394,875		33,417

ASSUMPTIONS:

[1] 1 gpm (per inch of rainfall) inflow at concrete/cast iron ring interface
[2] Assume 2 gpm leak at invert or joint - varies with groundwater level
[3] 90% of rainfall within collection area (assume 6,000 sq. ft.) enters the sewer
[4] 1 gpm infiltration per catchbasin
[5] 1 gpm infiltration per defect
[6] 1 gpm infiltration per defect (cost assume 60' of 4" @ $20/LF)
[7] 1 gpd inflow thru open or damaged cleanout
[8] 100% of rainfall from 2000 sq. ft. roof enters the sewer
[9] I&I rate is based on a 2.55 inch in 24-hour rainfall event
[10] City costs are for construction and notification of private owners
[11] Assume $0.000327/gal. treatment cost

Repair cost assume the work would be completed by contractors

Table 3

City of Corvallis

Smoke Test Program - Rehabilitation Priorities

	REHAB PRIORITY	DEFECT FORM #	TEST BASIN #	LOCATION	MANHOLE DEFECT	CATCH BASIN CONNECTED	MAINLINE DEFECT	LATERAL DEFECT	OPEN PLUMBING	DOWN-SPOUT	OTHER	COMMENTS
1	1	14	1	NE corner Cleveland & Garyanna		x						
2	1	15	1	NW corner Cleveland & Garyanna		x						
3	1	16	1	SE corner Cleveland & Garyanna		x						
4	1	25	1	1620 Garfield St		x						
5	1	26	1	Arthur St		x						
6	1	35	1	1855 Authur Circle								Near SS mh 46-77-43
7	1	49	1	SE corner Greely & 15th St		x					C.O. acting as CB	
8	1	51	1	NE corner Greely & 15th St		x						
9	1	62	1	1910 Division St			x					SS mh 6 ft south smoked
10	1	67	1	Corner Division & Division Pl		xxx						sinkholes developing
11	1	79	1	Highland Middle School		x						NW,NE & SE corners
12	1	91	1	900 Hayes St	x							school parking lot
13	1	104	2	973 Sycamore St			x					SS mh smoked
14	1	131	2	612 Linden Ave		x	x					smoked at base of pole
15	1	151	2	992 Meadowridge Pl			x					possible main leak
16	1	152	2	North of 992 Meadowridge Pl	x		x					creek leaking into sewer
17	1	174	2	2765 Stewart Ave		x						
18	1	177	2	Intersection of Hemlock & Bryant		xxx						
19	1	184	2	2951 Conifer Pl		x		x				all 3 catch basins smoked
20	1	206	2	2300 9th St - Wendys		x						ribbon drain
21	1	207	2	2350 9th St - video store		x		x				
22	1	208	2	NW corner of 9th & Linden St		x						
23	1	209	2	Intersection of 9th & Sequoia		x						
24	1	211	2	NW corner of 9th & Oak St		x						possible cross connection
25	1	213	2	2503 9th St - Arbys		x						
26	1	216	2	NW corner of 9th & Circle		x						parking lot drain
27	1	217	2	2123 9th St - Highland Bowl		x						
28	2	24	1	1625 17th St			x	x				sinkhole forming
29	2	89	1	900 Hayes St			x					
30	2	179	2	465 Ponderosa Ave			x					
31	2	255	3	1925 27th St			x		x			in easement
32	2	265	3	1956 23rd St			x					numerous leaks

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33	3	5	1	1925 14th St	x							Buried mh
34	3	23	1	SE corner 17th & Arthur St	x							
35	3	36	1	1900 Arthur Circle	x							
36	3	42	1	1795 Authur Circle	x							
37	3	54	1	1432 13th St	x							
38	3	65	1	978 Cleveland Pl	x							private mh
39	3	70	1	1920 Highland - School	x							see defect 78
40	3	78	1	1920 Highland - School	x							
41	3	83	1	1616 Dixon St	x							
42	3	87	1	1420 Highland St	x							mh in gutter recieves inflow
43	3	88	1	1430 Division St	x							2 mh leaking
44	3	90	1	900 Hayes St	x							major leak
45	3	94	1	4600 Kings Blvd	x							Dixon Creek Professional
46	3	98	2	927 Sycamore St	x							SS mh smoked
47	3	101	2	938 Sycamore St	x							private mh
48	3	124	2	2327 Highland St	x				x			
49	3	137	2	952 Oak St	x							
50	3	143	2	810 Sundance Ct	xx							mhs act as catchbasins
51	5	147	2	2901 Satinwood	xx							buried mhs
52	3	162	2	658 Survista Ave	x							
53	3	163	2	590 Survista Ave	x							
54	3	165	2	534 Maxine	x							
55	3	171	2	560 Hemlock Ave	x							
56	3	188	2	3220 9th St	x							
57	3	190	2	3008 9th St	x							
58	3	193	2	Intersection of 9th & Conifer	x							
59	3	194	2	390 Conifer Ave	x		x					buried manhole
60	3	195	2	2950 9th St	x							
61	3	204	2	3045 Autumn St	x							
62	3	218	2	9th & Sycamore St	x							Storm sewer mh smoked
63	3	225	3	2550 27th St	x				x			
64	3	226	3	2589 Walnut Blvd	x							buried mh
65	3	227	3	2650 Roosevelt	x							
66	3	239	3	2125 25th St	x							
67	3	245	3	4300 23rd St	x							
68	3	246	3	9800 23rd St	x							
69	3	247	3	2276 23rd St	x							
70	3	250	3	2115 Elmwood St	x							
71	3	252	3	2115 Elmwood St	x							buried mh
72	3	260	3	1930 Kings Blvd	x							in street
73	3	261	3	1930 Kings Blvd	x							backyard
74	3	262	3	2130 Elmwood Pl	x		x					
75	3	264	3	2310 Elmwood Pl	x		x					

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City of Corvallis

Smoke Test Program - Rehabilitation Priorities

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76	4	7	1	1310 Cleveland St						x		All downspouts smoked
77	4	9	1	1417 Menlo St						x		All downspouts smoked
78	4	12	1	1525 Menlo St						x		
79	4	28	1	1627 Garfield St						x		
80	4	53	1	1555 14th St						x		
81	4	57	1	1696 Division St						x		
82	4	64	1	1914 Division St						x		
83	4	69	1	1690 Highland St						x		
84	4	71	1	1920 Highland St. - School						x		
85	4	73	1	1665 11th St						x		
86	4	74	1	1717 11th St.						x		
87	4	75	1	1727 11th St						x		
88	4	76	1	1737 11th St						x		
89	4	80	1	Highland Middle School				x		x		
90	4	85	1	1610 Dixon St				x		x		
91	4	99	2	934 Sycamore St						x		
92	4	112	2	971 Sequoia St						x		
93	4	113	2	975 Sequoia St						x		
94	4	117	2	969 Sequoia St				x		x		
95	4	118	2	971 Sequoia St				x		x		
96	4	197	2	410 Pondersosa Ave					x	x		
97	4	198	2	430 Ponderosa Ave						x		
98	4	199	2	440 Ponderosa Ave						x		
99	4	231	3	2154 27th Pl						x		
100	4	234	3	2717 Elmwood St						x		
101	4	235	3	2803 Elmwood St				x		x		
102	4	236	3	2813 Elmwood St				x		x		
103	4	238	3	2050 23rd St		x						private area drain
104	4	256	3	1820 27th St						x		
105	4	258	3	2615 Circle Blvd						x		

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Smoke Test Program - Rehabilitation Priorities

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106	5	2	1	1900 14th St				x				
107	5	3	1	1910 14th St				x				
108	5	11	1	1775 Menlo St					x			Crawl space smoked
109	5	18	1	1530 Menlo St				x	x			
110	5	19	1	1920 Menlo St				x				
111	5	20	1	1852 Garryanna St				x				
112	5	22	1	1715 17th St				x				
113	5	27	1	1710 Menlo St				x				2 places
114	5	32	1	1640 17th St				x				
115	5	34	1	1865 Arthur Circle				x				
116	5	37	1	1650 Authur Circle				x				
117	5	38	1	1885 Authur Circle							C.O. leaking in St	
118	5	39	1	1885 Authur Circle				x				
119	5	40	1	1875 Authur Circle				x				
120	5	43	1	1715 Aurthur Circle				x				
121	5	46	1	2021 Garfield St				x				C.O. leaking
122	5	50	1	1430 Greely St				x				
123	5	56	1	1625 Highland St					x			
124	5	59	1	935 Hobart St				x				betw apt. 5 & 6
125	5	61	1	1840 Division St				x				
126	5	63	1	1910 Division St				x				
127	5	66	1	1820 Division St				x				
128	5	72	1	1641 11th St				x				
129	5	84	1	1736 Dixon St				x				
130	5	96	2	925 Sycamore St				x				
131	5	97	2	923 Sycamore St				x				
132	5	100	2	934 Sycamore St					x			smoke around C.O.
133	5	103	2	973 Sycamore St				x				
134	5	105	2	2160 Janssen St				x				
135	5	107	2	972 Sequoia St				x				
136	5	108	2	978 Sequoia St				x				
137	5	109	2	982 Sequoia St				x				
138	5	110	2	988 Sequoia St				x				
139	5	111	2	981 Sequoia St				x				
140	5	114	2	966 Sequoia St				x				
141	5	115	2	960 Sequoia St					x			broken C.O. cap
142	5	116	2	955 Sequoia St				x				
143	5	120	2	939 Sequoia St				x				
144	5	121	2	944 Sequoia St				x				
145	5	123	2	2321 Highland St				x				
146	5	125	2	2340 Wayland				x				
147	5	127	2	987 Cypress St				x				
148	5	130	2	580 Linden Ave				x	x			
149	5	132	2	655 Linden Ave				x				
150	5	133	2	670 Linden Ave				x				

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151	5	134	2	660 Linden Ave				x				
152	5	138	2	856 Oak St				x				
153	5	139	2	828 Oak St				x				
154	5	140	2	805 Oak St				x				
155	5	149	2	924 Meadowridge				x				
156	5	155	2	857 Chipmunk Ave				x				
157	5	158	2	782 Deer Pl				x				
158	5	167	2	2810 Stewart Ave				x				
159	5	168	2	2765 Stewart Ave					x			
160	5	169	2	2830 Stewart Ave				x				
161	5	170	2	2770 Stewart Ave				x				
162	5	172	2	2735 Stewart Ave				x				
163	5	173	2	2745 Stewart Ave				x				
164	5	176	2	520 Hemlock Ave.				x				
165	5	180	2	435 Ponderosa Ave				x				
166	5	181	2	470 Conifer Ave					x			
167	5	183	2	2921 Conifer Pl				x	x			
168	5	185	2	2918 Conifer Pl				x				
169	5	187	2	3220 9th St				x				
170	5	189	2	3124 9th St				x				
171	5	191	2	2950 9th St				x				
172	5	200	2	2641 9th St				x				
173	5	202	2	390 Maxine Ave				x				
174	5	203	2	2939 Conifer Pl					x			
175	5	210	2	923 Sequoia St				x				
176	5	212	2	400 Walnut Blvd - Enterprize Rental				x				
177	5	215	2	1895 9th St - Plaza 9				x				
178	5	220	2	1800 9th St					x			
179	5	221	2	1895 9th St - Plaza 9				x				
180	5	222	2	560 Walnut Blvd					x			
181	5	224	3	2525 27th St				x				
182	5	228	3	2695 Roosevelt				x				
183	5	233	3	2620 Elmwood St					x			
184	5	240	3	2140 25th Pl				x				
185	5	242	3	2230 29th St				x				
186	5	251	3	2135 Elmwood St				x				
187	5	254	3	2683 Circle Blvd				x				
188	5	259	3	2525 Pendleton St				x				
189	5	266	3	1940 25th St				x				
190	5	267	3	1922 25th St				x				