Oregon
Theodore R. Kulongoski, Governor

Department of Land Conservation and Development
635 Capitol Street, Suite 150
Salem, OR 97301-2540
(503) 373-0050
Fax (503) 378-5518
www.lcd.state.or.us

NOTICE OF ADOPTED AMENDMENT

August 15, 2008

TO: Subscribers to Notice of Adopted Plan or Land Use Regulation Amendments

FROM: Mara Ulloa, Plan Amendment Program Specialist

SUBJECT: City of Grants Pass Plan Amendment
DLCD File Number 003-08

The Department of Land Conservation and Development (DLCD) received the attached notice of adoption. Due to the size of amended material submitted, a complete copy has not been attached. A copy of the adopted plan amendment is available for review at the DLCD office in Salem and the local government office.

Appeal Procedures*

DLCD ACKNOWLEDGMENT or DEADLINE TO APPEAL: September 2, 2008

This amendment was submitted to DLCD for review 45 days prior to adoption. Pursuant to ORS 197.830 (2)(b) only persons who participated in the local government proceedings leading to adoption of the amendment are eligible to appeal this decision to the Land Use Board of Appeals (LUBA).

If you wish to appeal, you must file a notice of intent to appeal with the Land Use Board of Appeals (LUBA) no later than 21 days from the date the decision was mailed to you by the local government. If you have questions, check with the local government to determine the appeal deadline. Copies of the notice of intent to appeal must be served upon the local government and others who received written notice of the final decision from the local government. The notice of intent to appeal must be served and filed in the form and manner prescribed by LUBA, (OAR Chapter 661, Division 10). Please call LUBA at 503-373-1265, if you have questions about appeal procedures.

***NOTE:** **THE APPEAL DEADLINE IS BASED UPON THE DATE THE DECISION WAS MAILED BY LOCAL GOVERNMENT. A DECISION MAY HAVE BEEN MAILED TO YOU ON A DIFFERENT DATE THAN IT WAS MAILED TO DLCD. AS A RESULT YOUR APPEAL DEADLINE MAY BE EARLIER THAN THE ABOVE DATE SPECIFIED.**

Cc: Gloria Gardiner, DLCD Urban Planning Specialist
John Renz, DLCD Regional Representative
Jared Voice, City of Grants Pass

<paa> ya/

FORM 2

DLCD
Notice of Adoption

THIS FORM **MUST BE MAILED** TO DLCD
WITHIN 5 WORKING DAYS AFTER THE FINAL DECISION
PER ORS 197.610, OAR CHAPTER 660 - DIVISION 18

☐ In person ☐ electronic ☐ mailed
DATE STAMP
DEPT OF
AUG 13 2008
LAND CONSERVATION AND DEVELOPMENT
For DLCD Use Only

Jurisdiction: City of Grants Pass
Local file number: 08-40500002
Date of Adoption: 7/16/08 (oral) / 8/6/08 (Written)
Date Mailed: 8/11/08
Was a Notice of Proposed Amendment (Form 1) mailed to DLCD? **Select one** Date: Yes, mailed 4/18/08

☒ Comprehensive Plan Text Amendment
☐ Comprehensive Plan Map Amendment
☐ Land Use Regulation Amendment
☐ Zoning Map Amendment
☐ New Land Use Regulation
☐ Other:

Summarize the adopted amendment. Do not use technical terms. Do not write "See Attached".
- Updated water and sewer sections of Comprehensive Plan Element 10 (Public Facilities + Services)
- Amended Water Service, Sewer Service + General Service Comprehensive Plan policies
- Adoption of water and sewer planning documents by reference

Does the Adoption differ from proposal? Please select one
Yes, two policies were retained that were originally proposed for deletion

Plan Map Changed from: to:
Zone Map Changed from: to:
Location: Acres Involved:
Specify Density: Previous: New:
Applicable statewide planning goals:

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
☒	☒	☐	☐	☐	☒	☐	☐	☒	☒	☒	☐	☐	☒	☐	☐	☐	☐	☐

Was an Exception Adopted? ☐ YES ☒ NO
Did DLCD receive a Notice of Proposed Amendment...
45-days prior to first evidentiary hearing? ☒ Yes ☐ No
If no, do the statewide planning goals apply? ☐ Yes ☐ No
If no, did Emergency Circumstances require immediate adoption? ☐ Yes ☐ No

DLCD file No. 003-08 (16856)

Please list all affected State or Federal Agencies, Local Governments or Special Districts:

Josephine County
Grants Pass Irrigation District
State DEQ, Watermaster, Health Division

Local Contact: Jared Voice Phone: (541) 474-6355 Extension: 6317

Address: 101 NW 'A' Street Fax Number: 541-476-9218

City: Grants Pass Zip: 97526 E-mail Address: jvoice@grantspassoregon.gov

ADOPTION SUBMITTAL REQUIREMENTS

This form **must be mailed** to DLCD **within 5 working days after the final decision**
per ORS 197.610, OAR Chapter 660 - Division 18.

1. Send this Form and **TWO Complete Copies** (documents and maps) of the Adopted Amendment to:

ATTENTION: PLAN AMENDMENT SPECIALIST
DEPARTMENT OF LAND CONSERVATION AND DEVELOPMENT
635 CAPITOL STREET NE, SUITE 150
SALEM, OREGON 97301-2540

2. Electronic Submittals: At least **one** hard copy must be sent by mail or in person, but you may also submit an electronic copy, by either email or FTP. You may connect to this address to FTP proposals and adoptions: **webserver.lcd.state.or.us**. To obtain our Username and password for FTP, call Mara Ulloa at 503-373-0050 extension 238, or by emailing **mara.ulloa@state.or.us**.

3. Please Note: Adopted materials must be sent to DLCD not later than **FIVE (5) working days** following the date of the final decision on the amendment.

4. Submittal of this Notice of Adoption must include the text of the amendment plus adopted findings and supplementary information.

5. The deadline to appeal will not be extended if you submit this notice of adoption within five working days of the final decision. Appeals to LUBA may be filed within **TWENTY-ONE (21) days** of the date, the Notice of Adoption is sent to DLCD.

6. In addition to sending the Notice of Adoption to DLCD, you must notify persons who participated in the local hearing and requested notice of the final decision.

7. **Need More Copies?** You can now access these forms online at **http://www.lcd.state.or.us/**. Please print on **8-1/2x11 green paper only**. You may also call the DLCD Office at (503) 373-0050; or Fax your request to: (503) 378-5518; or Email your request to **mara.ulloa@state.or.us** - ATTENTION: PLAN AMENDMENT SPECIALIST.

ORDINANCE NO. 5460

ORDINANCE AMENDING SECTIONS 10.20 (WATER SERVICES) AND 10.30 (SEWER SERVICES) OF COMPREHENSIVE PLAN ELEMENT 10 (PUBLIC FACILITIES AND SERVICES), INCLUDING AMENDMENTS TO THE GENERAL SERVICE POLICIES, WATER SERVICE POLICIES AND SEWER SERVICE POLICIES, AND ADOPTING RECENT WATER AND SEWER PLANNING DOCUMENTS BY REFERENCE AS PART OF THE PUBLIC FACILITIES ELEMENT OF THE COMPREHENSIVE PLAN.

WHEREAS:

1. The Grants Pass and Urbanizing Area Comprehensive Community Development Plan was adopted December 15, 1982; and

2. Existing Sections 10.20 (Water Services) and 10.30 (Sewer Services) of Element 10 of the Comprehensive Plan determined service needs within the Urban Growth Boundary through the year 2000; and

3. The ordinance amends Sections 10.20 (Water Services) and 10.30 (Sewer Services) of Element 10 of the Comprehensive Plan, to incorporate information from recent water and sewer planning documents that determine service needs through at least the year 2020, and repeals existing Comprehensive Plan Sections 10.20 and 10.30; and

4. The ordinance adopts revisions to the Element 10 (Public Facilities and Services) General Service Policies, Water Service Policies and Sewer Service Policies; and

5. The ordinance adopts the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:
    a. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)
    b. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)
    c. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MHW/Montgomery Watson Harza, April 2004)
    d. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
    e. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
    f. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
    g. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999); and

6. The ordinance repeals previously-adopted water and wastewater plans which are rendered obsolete by adoption of the new plans; and

7. The proposal is consistent with the goals and policies of the Comprehensive Plan; and

8. The applicable criteria from the Comprehensive Plan are satisfied, and approval of the proposal is recommended by the Planning Commission to the City Council.

NOW, THEREFORE, THE CITY OF GRANTS PASS HEREBY ORDAINS:

Section 1: The amendments to the Comprehensive Plan and accompanying Policies document, as set forth in Exhibits 'A' and 'B', which are attached to and incorporated in this ordinance as follows, are hereby adopted:

A. Sections 10.20 (Water Services) and 10.30 (Sewer Services), Comprehensive Plan (***Exhibit A***)

B. Element 10, Comprehensive Plan Policies (***Exhibit B***)

Section 2: Existing Comprehensive Plan Sections 10.20 (Water Services) and 10.30 (Sewer Services) are hereby repealed.

Section 3: The water and sewer planning documents listed above are hereby adopted by reference.

Section 4: Any previously-adopted water and wastewater plans which are rendered obsolete by adoption of the new plans are hereby repealed.

ADOPTED by the Council of the City of Grants Pass, Oregon, in regular session this 16th day of July, 2008.

SUBMITTED to and Approved by the Mayor of the City of Grants Pass, Oregon, this 18 day of July, 2008.

Len Holzinger, Mayor

ATTEST:

David Reeves
Finance Director

Date submitted to Mayor: 7-18-08

Approved as to Form, Kris Woodburn, City Attorney Carl Huffman Deputy City Attorney

10.00 PUBLIC FACILITIES INDEX

10.20.0 WATER SERVICES INDEX

10.30.0 SANITARY SEWER SERVICES INDEX

10.40.0 STORM DRAINAGE SERVICES INDEX

10.50.0 SOLID WASTE SERVICES INDEX

10.60.0 POLICE PROTECTION SERVICES INDEX

10.70.0 FIRE PROTECTION SERVICES INDEX

10.80.0 SCHOOL SERVICES INDEX

EXHIBIT A
to Ordinance

10.20 WATER SERVICES INDEX

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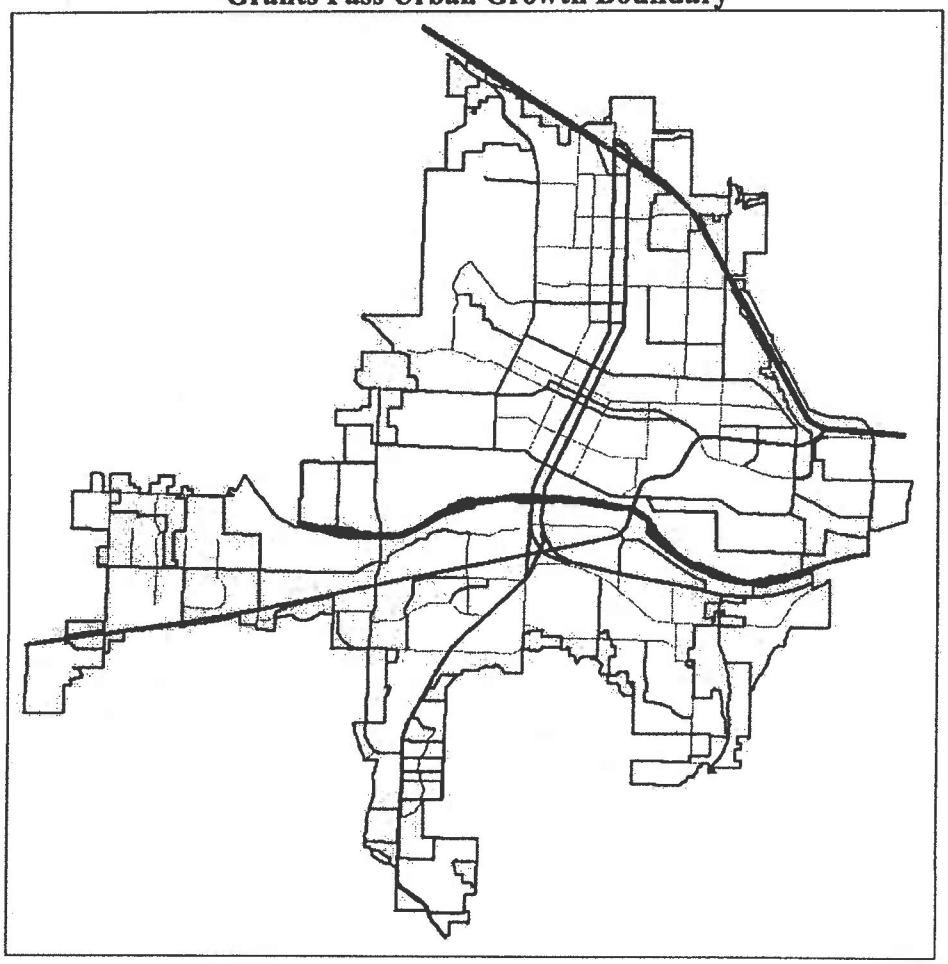
- 10.20.9.2 Water Treatment
- 10.20.9.3 Water Storage and Distribution

10.20.1 PURPOSE AND INTENT

10.20.1.1 Purpose.

The purpose of this section is to determine the domestic water demand requirements for the build-out of the Urban Growth Boundary (UGB), plus other areas served by municipal water, to assess the ability of the existing municipal water system to meet the projected requirements; to determine what capital improvements are necessary to serve the UGB at built-out, and to approximate costs; to suggest alternative methods for financing the required improvements; and to propose policies for the orderly provision of the required improvements.

Figure 10.20.1
Grants Pass Urban Growth Boundary



10.20.1.2 Intent.

The intent of this section is to enact the following public facilities water system master plans by ordinance as an update to the Public Facilities Element of the City of Grants Pass Comprehensive Plan:

1. *City of Grants Pass Water Management and Conservation Plan, Final Report,* West Yost and Associates, June 2002.
2. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report,* MWH/Montgomery Watson Harza, April 2004.
3. *City of Grants Pass Water Distribution System Master Plan,* West Yost and Associates, January 2001.

NOTE: The Dyer Partnership, Inc. prepared a "Water Master Plan" for Merlin and North Valley in April 2001. The recommended alternative was connection to the City of Grants Pass system. The "Water Master Plan" was adopted by Josephine County as part of its' Merlin / North Valley Community Plan (see Article 101.015 of the Josephine County Rural Land Development Code.) The County has not taken steps to implement the plan, and the City of Grants Pass has not adopted the plan. The City will continue to provide water service to specific properties through individual service agreements; however there are no additional obligations to provide service to properties other than those which are currently served (as of 2008.) The North Valley Industrial Park and some residential uses adjacent to the Merlin Landfill are currently served by City water, and facilities have also been extended to serve the Paradise Ranch Development.

NOTE: Several of the tables within this section contain numbers that have been updated since the above-listed plans were completed. The plans are based on data which was available at the time they were adopted, and have not been updated based on more recent data.

10.20.2 WATER SOURCES

10.20.2.1 Ground Water.

Within the Grants Pass area, an "alluvial deposit" geologic formation is the only reliable source of ground water. In this formation, however, the expected maximum yield from wells of standard construction is 50 gallons per minute, which is insufficient for a municipal supply. Due to lack of adequate quantity, ground water in the Grants Pass area has no potential for municipal use beyond that presently developed. (City of Grants Pass Water System Study, May 1974, Brown and Caldwell)

10.20.2.2 Surface Water.

In the Grants Pass area, surface waters have been in the past, and will continue to be, the only reliable source for large quantities of potable water required for municipal purposes. (Ibid.) The Rogue River is the principal supply of surface water; however, water rights to the Rogue River are limited. The Rogue River drains a large watershed extending from the Cascade Mountains to the Pacific Ocean. Grants Pass is located at approximately River Mile 100 and there are approximately 2,460 square miles of watershed area upstream of the City. As a result of this extensive drainage area, the Rogue River is a plentiful and reliable source of drinking water for the community.

The U.S. Geological Survey (USGS) maintains a river gauging station at the Grants Pass water treatment plant that provides extensive historical data on the flow characteristics of the Rogue River. Because the Lost Creek Reservoir was constructed upstream of Grants Pass in 1977, USGS statistical data for the river are typically based on records from 1978 to the present. Based on USGS data for this station, Table 10.20.2 presents the average, maximum, and minimum monthly flow rates for the Rogue River. Since construction of Lost Creek Reservoir, the lowest daily average flow at Grants Pass was 744 cubic feet per second (cfs) on October 10, 1994, and the lowest seven-day average flow was 799 cfs during the week of September 22, 1994. In general, dry weather flows are maintained by the combination of snow melt from the Cascades in the early summer and the release of stored water from Lost Creek Reservoir in the late summer.[1]

Table 10.20.2 Rogue River Average Monthly Flows at Grants Pass
USGS Data for the 28-Year Period Oct 1978 to Sept 2006

Month	Average Monthly Flow, cfs	Maximum Monthly Flow, cfs	Minimum Monthly Flow, cfs
January	5,094	16,600	1,348
February	4,500	10,960	1,250
March	4,020	8,119	1,099
April	3,950	6,843	1,211
May	3,750	6,428	1,857
June	2,790	4,572	1,549
July	2,120	3,485	1,059
August	2,080	3,080	1,620
September	1,780	2,642	1,333
October	1,450	2,282	1,008
November	2,530	7,669	1,160
December	4,910	17,620	1,557

Source: Grants Pass Water Management Plan, June 2002 (Updated 2/25/2008 by Jason Canady from USGS River Data.)

1 Source: Grants Pass Water Management Plan, June 2002, West Yost and Associates, LLC., page 2-1.

10.20.3 WATER RIGHTS

10.20.3.1 Rogue River.

The City has four separate permits for diverting water from the Rogue River for municipal use. The first is a "perfected right" of 12.5 cfs, dated 1888. The second and third are permits for 25 cfs each dated 1960 and 1965. The fourth permit, dated 1983, provides for an additional 25 cfs for a total of 87.5 cfs. The City's present water rights and water permits are shown in Table 10.20.3.

As water permits and rights are typically subject to cutbacks under conditions of low flow to serve parties with prior year rights, stored water can allow jurisdictions to augment flow otherwise cut back. Cutbacks during the 1977-78 drought years approached the 1965 level. Without Lost Creek Dam, cutbacks in 1981 would have reached back to the early 1900's, as released water accounted for 50% of stream flow that summer. However, since the City's point of diversion is downstream from the Savage Rapids Dam[2], and cutbacks are established by law as being those jurisdictions and individuals above the dam, there is some question as to whether the City could in fact be cut back, even under low water conditions.

TABLE 10.20.3
Grants Pass Water Rights

Permit Number	Priority Date	Permitted Use	Permitted Rate, cfs	Available Quantity, cfs	Source Availability Analysis		
					Reliability	Impact of ESA	Water Quality
D15839	1888	Municipal / Irrigation	12.5 cfs		High	Undefined	Good
S26901	1960	Municipal	25.0 cfs	735 cfs*	High	Undefined	Good
S45827	1965	Municipal	25.0 cfs		High	Undefined	Good
S47346	1983	Municipal	25.0 cfs		High	Undefined	Good

* Restriction that water can be diverted only when flow at the mouth of the Rogue exceeds 735 cfs.
Source: Grants Pass Water Management Plan, June 2002

Each Water Right and Permit has a specific geographical area within which the water may be used. The 1888 right stipulates "the city limits," which the City holds to be those city limits as they exist at any point in time. The 1960 permit shows an area approximating the City's 1979 Urban Growth Boundary (20 year expansion); the 1965 and 1983 permits show an area approximating a 50-year expansion.

10.20.3.2 Long-Term Reliable Yield.

Due to the nature of the City's surface water supply source, the long-term sustainability of drinking water supplies for Grants Pass is generally good. As noted earlier, the large size of the watershed drained by the Rogue River typically provides abundant water supplies throughout the year. Even during extreme dry weather periods when river flows are at their lowest, the reliable flow rate in the Rogue River is approximately 750 cfs or nearly fifty times larger than the highest drinking water demand ever experienced in Grants Pass. There are some special circumstances that may affect the

2 Savage Rapids Dam is slated for removal to resolve anadromous fish passage issues. Dam removal will occur once the GPID secures an appropriation of funding from both the federal and state sources. GPID will then pump water from the Rogue River and deliver water to its customers, including those within the City of Grants Pass.

long-term reliable yield for the Rogue River. For example, the listing of the Coho Salmon as an endangered species in the Rogue River may influence operational procedures at the Lost Creek Reservoir, which in turn may affect dry weather flow levels. Another issue is related to climate change and snow pack levels in the Cascade Range. Any reduction in average precipitation or the average snow pack will tend to reduce dry weather flow rates in the Rogue River. Since these factors are complex in nature, it is difficult to quantify their potential effect on the river's reliable yield at this time.[3]

10.20.3.3 Grants Pass Irrigation District.

The Grants Pass Irrigation District was organized by the local water users in January 1917. The area of the district was then about 6,000 acres. It originally was planned to irrigate by an extension of the Gravity Canal of the Gold Hill Irrigation District, which was further upstream on the Rogue River and was being organized at the same time. That plan was abandoned in 1920 and the present design was adopted to provide for a direct diversion system with permanent pumping units. The original works were constructed with private funds.

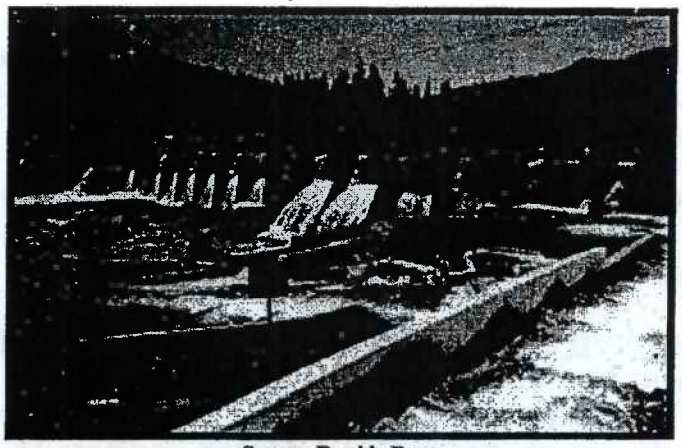
The Savage Rapids Diversion Dam was dedicated November 5, 1921, marking the beginning of the operating history of the district. Settlement and clearing of the undeveloped lands, which constituted a high proportion of the district's area, did not develop to the extent of the expectations upon which the district was founded and financed. As a result, just over one-half of the irrigable area was in production and therefore carried the entire tax burden.

The Savage Rapids Dam and the Northwest Unit pipeline were badly damaged by a flood in 1927. Emergency repairs were made at that time, but lack of sufficient funds prevented satisfactory completion of the work. The cost of maintenance on the pipeline had become almost prohibitive by 1949.

In 1949, the Bureau of Reclamation was requested to replace the old suspension pipeline and siphon with a new buried line under the Rogue River. Several years later Reclamation was asked to rehabilitate Savage Rapids Dam. After thorough investigations, both requests were undertaken and completed. In 1974, the Bureau of Reclamation and Bureau of Sport Fisheries and Wildlife investigated and prepared a report on anadromous fish passage improvements at Savage Rapids Dam.

On February 12, 1982, the Grants Pass Irrigation District perfected a right to 96.7 cfs with a priority date of 1916. In addition, the District "transports" 83 cfs for the Department of Fish and Game after use as irrigation water for stream enhancements, and was granted a "non-consumptive" right of 800 cfs of pass through water needed to drive the water turbines that lift irrigation water to the canals. The GPID point of diversion is at Savage Rapids Dam, see photo below.

3 Source: Grants Pass Water Management Plan, June 2002, West Yost and Associates, LLC., page 2-2.



Savage Rapids Dam
Source: U.S. Department of Interior, Bureau of Reclamation

10.20.3.4 Savage Rapids Diversion Dam.

The Savage Rapids Diversion Dam is on the Rogue River 5 miles east of Grants Pass. It is about 456 feet long and consists of a 16-bay spillway section and a hydraulic-driven pumping plant section at the right abutment. Maximum height of the spillway section is about 39 feet. The first seven bays at the right end of the dam are multiple arches with buttresses on 25-foot centers; the remaining nine bays have a concrete gravity section below the gates. Sixteen wooden-faced radial gates originally provided spillway control. Each of them constructed 23 feet wide and 10 feet high. During rehabilitation, the radial gates were replaced with metal stoplogs, and one double-gated river outlet with a capacity of 6,000 cubic feet per second was installed at the center of the dam. During the irrigation season, the stoplogs are used to raise the reservoir elevation 11 feet.

The Savage Rapids Diversion Dam diverts water from the Rogue River into the South Main Canal to serve the lowlands on the south side of the river. The main pumping plant pumps water from the reservoir to the Tokay Canal to serve lands on the north side of the river, and to the South Highline Canal to irrigate lands above the gravity-type South Main Canal. There are also four lateral relift pumping plants along the canals.

A study by the Bureau of Reclamation (October, 1979) showed that the District served 400 acres zoned exclusive farm use utilized by commercial growers, while serving 7,000 acres of "urban-suburban" lands to irrigate lawns, gardens and pastures. Diversion of water was between 180 to 220 cfs. The Bureau of Reclamation Study indicated that urban-suburban development posed a major problem for canal maintenance and water distribution, being a prime contributor to the District's loss of 2,600 acres of formerly irrigated lands. One of the key factors contributing to maintenance problems is the silting up of those parts of the system from winter runoff, as many of the system canals and laterals also serve to carry storm drainage. The City's Storm Drainage Master Plan calls for a continuance and intensification of this practice, and will require improvements and maintenance coordination between the City and the District. As of 2007, no agreement exists between the City and GPID.

Some 15% of the District's 55 miles of major canals are lined or enclosed in pipe, the rest being

unlined. The Bureau of Reclamation recommended either merger of the City and GPID into a water control district (Oregon Revised Statute 553) with both drainage and irrigation services, or some combination of improvements to the canal system to maintain or extend the provision of irrigation water through the system.

A 1974 Grants Pass Water System Study by Brown and Caldwell estimated that the combined water rights of the City and GPID would be sufficient to meet the needs of both the UGB and agricultural users outside the Boundary area.

While discussions with GPID and the Josephine County Water Advisory Board touched on this possibility, the present GPID Board policy is to continue to supply water through the irrigation delivery system, even though development should occur. The District supplies water to almost all of the urbanizing area, and in fact, much of the City, at this time. The District has elected to encourage the continued supply of irrigation water to urbanizing land as development proceeds within the UGB. The District feels this benefits the developer, as the cost of supplying water through a piped system does not often exceed the buy-out cost per acre; benefits the homeowner; and benefits the City, as it saves treatment costs for irrigation water, does not require an increase in the size of the distribution system piping to carry the additional water demand, and reduces the peak per capita usage of water in the summer, thus effectively expanding the capacity of the City's permits.

10.20.3.5 Dam Removal.

In 1995, the Bureau of Reclamation filed a Final Planning Report/Draft Environmental Statement to enhance the salmon and steelhead populations of the Rogue River by removing Savage Rapids Dam. Two pumping plants, one on the north bank and one on the south bank, would be constructed to lift water into Grants Pass Irrigation District's canal system. This plan and environmental law suits resulted in a court order to remove the dam by 2005. To date, the GPID has secured federal and state funding to accomplish dam removal by 2009, including the construction of a new pump station. The uncertainty of these mandated changes has the City and GPID uncertain about any future plans or agreements.

10.20.4 CITY OF GRANTS PASS WATER TREATMENT PLANT, DISTRIBUTION SYSTEM AND WATER DEMAND

10.20.4.1 System History.

The Grants Pass Water Treatment Plant, located at 821 SE"M" Street, was originally built in 1931 with a single basin and three filters for a designed capacity of approximately 3.5 mgd. The plant has undergone several upgrades and expansions through the years to incrementally adjust to a growing population and more stringent treatment standards, including:

- 1950 – Capacity increased to 9 mgd through the addition of second basin and two additional filters.
- 1961 – Minor improvements to treatment process.
- 1983 – Capacity increased to 20 mgd through addition of third basin and three additional filters, construction of a new raw water intake and new chemical feed systems.

- 1995-2001 – Filter media and gravel support replaced due to suspected gravel/under drain upset caused by excessive air in the backwash line.
- 1997 – Filter-to-waste added for improved CT-removal credit.
- 1998 – SCADA upgrade; VFD included on BW pump.
- 1999-2000 – Improvements to the Equalization basin pumping station.
- 2001 – Liquid sodium hypochlorite system installed to replace gas system.
- 2001 – Riverbank stabilization adjacent to the intake structure, in cooperation with US Army Corps of Engineers.
- 2002 – New PLC-based SCADA system and new monitoring devices were installed in the plant to replace outdated analog transmitters and to allow for more accurate and complete process performance monitoring and automated process control. The PLC replaced obsolete analogue loop-controllers and chemical feed controllers.
- 2006 - Installation of one additional booster pump/ replacement of one aging inefficient booster pump. Reconstruction of the Intake Structure and complete rebuild of the filters and Surface Wash System.

The *City of Grants Pass Water Distribution System Master Plan*, 2001 (West Yost & Associates) inventories and evaluates the performance of the City's water distribution system for critical service standards. This analysis identified system improvements necessary to maintain adequate performance through build-out of the UGB. (**NOTE:** In its' "Future Water Demand" analysis, the plan also acknowledges that there are some properties contiguous to the UGB that are likely candidates for receiving water in the future, as well as some additional areas within the North Valley- see Chapter 3 of the *2001 Water Distribution System Master Plan.*) These identified improvements were developed to either eliminate existing deficiencies in system performance or expand service to satisfy community growth. The elements of the water distribution system that were evaluated include water treatment plant capacity, treated water storage capacity, booster pumping capacity, and pipeline network performance. Table 10.20.4, *Estimated Capital Costs for CIP Projects*, presents the specific costs of reservoir, pump station, and pipeline projects that are targeted for implementation through build-out of the UGB. The costs shown are the City's estimated share of the pipeline extensions and do not include the cost which will need to borne by developers. Timing of projects is based on the City's review of the expansion plans and reflects developer interest and submitted development plans. Some adjustment of timing and priorities should be expected.

Table 10.20.4 Estimated Capital Costs for CIP Projects

Recommended Improvements	Capital Cost, $1,000
Period 2005-2010	
Pipelines	1,049
Pipeline Replacement	1,124
Total	2,173
Period 2010-2020	
Treated Water Storage	5,720
Pipelines	2,042
Pipeline Replacement	2,248
Total	10,010
Period Post 2020	
Treated Water Storage	1,560
Pump Stations	400
Pipelines	184
Total	2,144
Grand Total	14,327

Source: Grants Pass Water Distribution System Master Plan, January 2001

Per the *2001 Water Distribution System Master Plan*, the Grants Pass water supply system distributes water to developed properties covering an area of more than 3,500 acres. The majority of the properties currently connected to the water distribution system are within the present city limits, although the City does provide service to some areas outside the city limits, including the Redwood and Harbeck-Fruitdale areas within the UGB, as well as parts of North Valley. The overall system is composed of a water treatment plant, thirteen booster-pumping stations, eight reservoirs, three pressure-reducing valves, five altitude valves, and approximately 160 miles of pipelines ranging in size between 1" and 30"(most of which are made of cast iron or ductile iron and range in age up to approximately 80 years).

10.20.4.2 Service Pressures.

The urban growth boundary for the City of Grants Pass encompasses lands of wide ranging elevations. As a result, the water distribution system service area contains eight separate service pressure zones serving the UGB and North Valley (See Pressure Zone Map in the *City of Grants Pass Water Distribution System Master Plan*, West & Yost & Associates, 2001). Table 10.20.6 summarizes the service elevations and static range for each pressure zone. The lower end of the pressure range is based on reservoirs at 80 percent full and the upper end is based on full reservoirs. At this time, there are properties receiving City water service in each of the pressure zones except Zone 5.

In some areas, the pressure zone boundaries are modified slightly from these elevation ranges in order to accommodate special service pressure requirements. Pressure Zone 2A is a hybrid between Zones 1 and 2, as is the Rogue Community College's Zone 2B. The North Valley service area spans pressure Zones 1, 2, and 3, serving properties between the elevations of 995 and 1,165 feet. Due to the great range of elevations served in the North Valley, this pressure zone requires pressure reduction valves at service connections to maintain appropriate service pressures. Table 10.20.5 displays the water distribution system pipeline sizes and lengths.

Table 10.20.5
Water Distribution System Pipeline Network

Pipe Size (inches)	Length (miles)
2	5.23
4	1.80
6	40.89
8	43.63
10	7.64
12	19.49
14	0.38
16	7.88
20	2.40
24	1.02
30	0.95
36	0.01
	131.32

Table 10.20.6 Pressure Zone Ranges

Zone	Elevation (feet)	Pressure (psi)
1	900 - 1,020	36 – 90
2	1,020 – 1,140	41 – 95
2A	960 – 1,035	61 – 94
2B	1,000 – 1,060	35 – 60[a]
3	1,140 – 1,280	36 – 100
4	1,280 – 1,420	42 – 104
5	1,420 – 1,560	41 – 104
NV	995 – 1,165	101 - 177

Source: Grants Pass Water Distribution System Master Plan, January 2001

10.20.4.3 Fire Protection.

The Grants Pass Department of Public Safety provides fire protection for properties within the City, and some properties with Service and Annexation Agreements within the Urban Growth Boundary. Since the water distribution system is an integral part of the City's fire protection system, the Department of Public Safety has adopted the Oregon Fire Code recommendations as the required fire flows for the various land use classifications within the City. These fire protection requirements are discussed in detail in Chapter 4, "*Water Distribution System Service Standards,*" *Grants Pass Water Distribution System Master Plan,* January 2001.

10.20.4.4 Booster Pumping Stations.

The water distribution system includes the water treatment plant pumps and thirteen booster pumping stations that transfer water to the higher pressure zones. These pump stations either fill the reservoirs that serve these higher pressure zones or pump to maintain a minimum pressure in those areas that are not served by reservoirs. On the following page, Table 10.20.7 details the technical information for each of the system's pumping stations.

Table 10.20.7 Existing Booster Pumping Stations

Pump Station Location	Service Level	Feeds Reservoir	Number of Pumps	Horsepower and RPM	Rated Capacity	Rated Head (TDH)
Lawnridge	2	Yes (6)	4	1 – 25HP @ 1750 RPM 2 – 50 HP @ 1750 RPM 3 – 50 HP @ 1750 RPM 4 – 100 HP @ 1750 RPM	400 GPM 1000 GPM 1000 GPM 2000 GPM	120 TDH 120 TDH 120 TDH 148 TDH
Madrone	2	Yes (4)	3	1 – 60 HP @ 1750 RPM 2 – 30 HP @ 1750 RPM 3 – 40 HP @ 1750 RPM	2000 GPM 900 GPM 1200 GPM	170 TDH 170 TDH 170 TDH
Champion	3	Yes (8)	3	1 – 100 HP @ 1750 RPM 2 – 150 HP @ 1750 RPM 3 – 50 HP @ 1750 RPM	1600 GPM 2300 GPM 800 GPM	165 TDH 165 TDH 165 TDH
Hefley	3 & 4	Yes (13)	4	1 – 7.5 HP @ 3500 RPM 2 – 15 HP @ 3500 RPM 3 – 60 HP @ 3500 RPM 4 – 60 HP @ 3500 RPM	40 GPM 120 GPM 600 GPM 600 GPM	250 TDH 250 TDH 300 TDH 300 TDH
Starlite	3	No	5	1 – 5 HP @ 3461 RPM 2A – 7.5 HP @ 3525 RPM 2B – 7.5 HP @ 3525 RPM 3 – 60 HP @ 1760 RPM 4 – 30 HP @1760 RPM	30 GPM 84 GPM 84 GPM 1050 GPM 450 GPM	315 TDH 208 TDH 208 TDH 185 TDH 185 TDH
North Valley	5	Yes (15)	3	1 – 7.5 HP @ 3500 RPM 2 – 30 HP @ 3500 RPM 3 – 30 HP @ 3500 RPM	70 GPM 500 GPM 500 GPM	170 TDH 174 TDH 174 TDH
Harbeck	2	No	3	1 – 5 HP @ 3600 RPM 2 – 5 HP @ 3600 RPM 3 – 50 HP @ 3600 RPM	90 GPM 90 GPM 1200 GPM	112 TDH 112 TDH 125 TDH
Hilltop	2	No	3	1 – 5 HP @ 3600 RPM 2 – 7.5 HP @ 3600 RPM 3 – 40 HP @ 3600 RPM	100 GPM 150 GPM 750 GPM	120 TDH 120 TDH 120 TDH
New Hope	2	No	4	1 – 30 HP @ 3600 RPM 2 – 30 HP @ 3600 RPM 3 – 30 HP @ 3600 RPM 4 – 150 HP @ 1800 RPM 5A – 5 HP @ 3600 RPM 5B – 5 HP @ 3600 RPM	350 GPM 350 GPM 350 GPM 2000 GPM 53 GPM 53 GPM	212 TDH 212 TDH 212 TDH 200 TDH 227 TDH 227 TDH
Laurel Ridge	3	No	3	1 – 15 HP @ 3500 RPM 2 – 15 HP @ 3500 RPM 3 – 60 HP @ 3500 RPM	350 GPM 350 GPM 1100 GPM	127 TDH 127 TDH 160 TDH
Meadow Wood	2 & 3	No	6*	1 – 7.5 HP @ 3500 RPM 2A – 15 HP @ 3500 RPM 2B – 60 HP @ 3500 RPM 2C – 60 HP @ 3500 RPM 3 – 100 HP @ 3500 RPM 4 – 7.5 HP @ 3500 RPM	80 GPM 150 GPM 500 GPM 500 GPM 1500 GPM 63 GPM	280 TDH 155 TDH 275 TDH 275 TDH 150 TDH 341 TDH
Williams Crossing	3	No	2	1 – 5 HP @ 3500 RPM 2 – 5 HP @ 3500 RPM	70 GPM 70 GPM	145 TDH 145 TDH
Panoramic Loop	3	No	4	1 – 25 HP @ 3600 RPM 2 – 10 HP @ 3600 RPM 3 – 60 HP @ 3600 RPM 4 – 60 HP @ 1800 RPM	70 GPM 150 GPM 1000 GPM 1000 GPM	316 TDH 138 TDH 157 TDH 157 TDH
Treatment Plant	1	Yes	6	1 – 250 HP @ 1780 RPM 2 – 300 HP @ 1775 RPM 3 – 250 HP @ 1780 RPM 3A – 250 HP @ 1780 RPM 4 – 250 HP @ 1780 RPM 5 – 200 HP @ 1780 RPM	3500 GPM 3500 GPM 3500 GPM 3500 GPM 3500 GPM 2600 GPM	210 TDH 220 TDH 210 TDH 210 TDH 210 TDH 210 TDH

* Pumps 1, 2A and 3 service Zone 2 exclusively. Pump 4 services Zone 3 exclusively. Pumps 2B and 2C service both Zones 2 and 3.
Data source: Jason Canady- City of Grants Pass Water Treatment Plant Supervisor March 2008

10.20.4.5 Reservoirs.

There are eight water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999. Design information for these reservoirs is detailed in Table 10.20.8.

Table 10.20.8 Existing Reservoirs

Reservoir Location	Reservoir Number	Pressure Zone Served	Year Built	Construction Materials	Capacity (mg)	Bottom Elevation (ft)	Overflow Elevation (ft)
500 Block Woodson Drive	3	1	1946	Concrete	3.5	1,089.5	1,108.5
1500 Block Ridge Road	4	2	1953	Concrete	0.75	1,216	1,240
1400 Block Sherman Lane	5	1	1983	Concrete	3.5	1,079.5	1,108.5
2200 Block Crown Street	6	2	1982	Concrete	3.5	1,211	1,240
Heiglen Loop Road	8	3	1983	Concrete	2.0	1,341	1,370
1420 Denton Trail	11	1	1999	Concrete	4.5	1,080.1	1,108.5
1700 Block Sunset Lane	13	4	1980	Concrete	0.08	1,510	1,520
3900 Block Highland Ave.	15	4	1985	Concrete	1.2	1,374	1,403

Source: Grants Pass Water Distribution System Master Plan, January 2001

10.20.4.6 System Operation.

The general procedures for operation of the Grants Pass water distribution system are discussed in the following:

- The water treatment plant operates as necessary to fill storage reservoirs in the distribution system on a daily basis. Therefore, the operating schedule adjusts to seasonal variations in water demand. During winter months, the plant generally operates seven days per week for an eight-hour period. Operational hours are extended during the high demand summer months, when the plant must operate up to twenty four hours a day in order to keep the storage reservoirs full.
- Those booster pumping stations that fill storage reservoirs are automatically controlled to maintain preset water levels. When sensors show that the water level of a reservoir has fallen below a preset threshold, the lead pump will activate and begin filling the reservoir to a high water level. If water demand on the reservoir is such that a single pump cannot maintain the water level, a lag pump (or pumps) will activate as necessary until the reservoir fills to a high water level.
- Those booster pumping stations that serve areas without storage reservoirs are automatically controlled to maintain a minimum discharge pressure at the pumping stations. When pressure sensors show that the discharge pressure has fallen below a preset threshold, the lead pump activates and pumps until the discharge pressure exceeds a high pressure level. If water demand in the pump station's service area is such that a single pump cannot maintain the pressure level, a lag pump (or pumps) will activate as necessary until the system pressure is restored.
- The reservoirs in the water distribution system are generally maintained between 80 and 100 percent full. This fluctuating volume represents the operating storage. The remaining storage is

allocated to providing fire flow requirements and emergency reserves. In the case of Reservoir No. 15 in the North Valley, water levels are maintained at a much lower level due to limited demand in that portion of the distribution system. Altitude valves control the flow into and out of Reservoirs No. 3, No. 4, No. 5, No. 6, and No. 11. These valves are designed to close when the reservoir is full and open when the system pressure drops. The other reservoirs in the distribution system float on the system.

- There are three pressure reducing valve stations in the Grants Pass water distribution system. Two of the stations control the flow of water from Pressure Zone 2 to Pressure Zone 2A. Pressure Zone 2A extends to slightly lower elevations than Pressure Zone 2 and thus requires some pressure reduction. Each station contains a single pressure-reducing valve (one is a 10-inch valve and one is a 6-inch valve). The third pressure reducing valve station on NE Beacon Drive reduces Zone 4 water to Zone 3.
- The City upgraded the water distribution system to a Supervisory Control and Data Acquisition (SCADA) system in 1999. SCADA system monitors reservoir levels, pump operating status, and local pressures throughout the system. The central computer system for the man-machine interface is located at the water treatment plant.

10.20.4.7 Water Treatment Plant.

The City of Grants Pass Water Treatment Plant (WTP) has successfully met the City's drinking water needs for over 70 years. The Rogue River supply is typical of many Pacific Northwest surface waters with low mineral content, low pathogen concentrations, and normally low turbidity, but with seasonal increases in turbidity due to precipitation and runoff. The Rogue River quality and flow is also influenced by the operation of upstream reservoirs including Lost Creak Reservoir and Savage Rapids Dam (slated for removal). Peak withdrawals by the WTP to meet demands in the summer months coincide with minimum river flows and low turbidities.

The WTP's main purposes include removal of suspended particulates, removal and inactivation of pathogens, and production of non-corrosive, palatable water in accordance with Federal and State drinking water regulations. The plant has historically met all regulations and the few customer complaints are limited to occasional chlorinous tastes and odors. The plant appears well positioned to continue to meet current and future drinking water needs.

The plant's production has steadily increased over the last decade in response to increasing water demands within the City's service area. The City's service area has been expanding as areas previously served by small groundwater systems have been incorporated into the City's water system. Significant investments have been made to upgrade the distribution and storage systems over the past few years. Water production at the plant has increased by approximately 20% since 1995. In 2007, peak day water production was 11.87 mgd, and the average annual production was 5.85 mgd.

The plant has rated maximum capacity of 20 mgd with all raw water and finished water pumps in operation. The reliable plant capacity is approximately 15 mgd with one of the largest pumps out of service. The plant is operated in a start/stop mode each day, with the hours of production varying between 8 to 24 hours per day depending on demands and raw water quality. The plant occasionally operates at the peak production rate of 20 mgd (14,000 gpm) during the high demand season.

Recently the plant has had to increase its operating staff to allow for 24-hour operations, to more reliably meet demand during peak summer season.

The *Water Treatment Plant Facilities Plan*, April 2004 (WTPFP) provides guidance for improving this major element of the City's water system and recommends a capital improvement program (CIP) that will meet the City's water treatment needs for 20 to 25 years. Based on a water demand increase of 2.5 to 3 percent per year, it is expected that the plant will continue to be able to meet the City's water needs for at least the next 20 years, with some modifications and improvements. A major plant expansion is not envisioned until the middle to end of decade 2020. Although the existing plant site is extremely confined, the plant is capable of being expanded to approximately 30 mgd with major modifications. The plant expansion to provide 30 mgd of treatment would be required in 25 years (on-line by 2028) if demand growth is steady at 2.5 percent per year. The expansion would be required in 20 years (on-line by 2023) if demand growth is steady at 3 percent per year. The existing plant structures appear to have significant remaining useful life. However, the older plant structures are vulnerable to damage during a severe seismic event.

While the plant has been able to successfully meet the City's water demands and also produce good water quality, the *Water Treatment Plant Facility Plan* identified the following challenging issues which have regulatory compliance implications and which create production inefficiencies. Note that the WTPFP was adopted in 2004 and since then measures have been taken to address the following issues:

- The existing Rogue River intake does not comply with current Endangered Species Act (ESA) regulations to protect juvenile fish including salmonids, due to high approach velocities and screen deficiencies. **NOTE:** This problem has since been fixed. Completed in 2006, the intake structure screen was replaced and the intake inlet was increased in size to reduce intake velocity.[4]
- The backwash/sludge holding pond is completely full of solids and immediate action is required, including development of a long-term solids management plan, to ensure continued compliance with the City's NPDES permit for discharge to Skunk Creek. **NOTE:** A master plan for a long-term solution to this problem was being developed as of September 2007. Short-term solids handling procedures are in place until the plan is completed.[4]
- The filter media is in a degraded condition and the plant (specifically the filters and sedimentation basins) is operating inefficiently requiring frequent backwashing and excessive raw water pumping, resulting in higher operating costs. **NOTE:** The filter media has been upgraded and the resulting operational inefficiency is no longer a problem.[4]
- Proposed drinking water regulations, including the Disinfectants/Disinfection By-Products Rule and the Long-Term 2 Enhanced Surface Water Treatment Rule, have the potential to require significant plant modifications depending on the outcome of monitoring programs. **NOTE:** Increased monitoring programs will be underway in the near future. Depending on the outcome of these programs, plant modifications may be required to maintain drinking water compliance.[5]

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4 Per Dave Wright and Joey Wright, City of Grants Pass Public Works Department, September 2007

5 Per Jason Canady, City of Grants Pass Water Treatment Plant Supervisor, September 2007

These and other challenges require the City to implement near-term improvements to the plant. The plant also requires a longer-term capital improvement program to ensure reliability and redundancy of major equipment, including adding new equipment, replacement/repair of major equipment as it becomes less reliable, and to prepare for a major plant expansion.

10.20.4.8 Water Demand Analysis.

Analysis of the City of Grants Pass historic water production and demand data allows for the identification of the unique water use patterns that characterize the City and provides a basis for estimating future water demand in the community. Additional analysis relates the various measures of water demand (maximum monthly demand, maximum daily demand, and peak hour demand) to the average annual demand through the use of peaking factors.

The projection of future water demand is based on unit demand factors developed by land use type and corresponding customer classifications. These future demand projections provide the basis for assessing the adequacy of the existing water production and distribution system and planning for future improvements.

10.20.4.9 Recent Water Use Statistics.

There are several measures of water use that are important to analyze during the development of the water master plan. Following is a description of the influential water demand factors that will guide planning decisions with respect to the Grants Pass water systems:

- Annual average demand – A measure of the amount of water that must be obtained from the available sources of supply on an annual basis.
- Monthly average demand – Review of monthly average water demand illustrates seasonal variations in demand due to such factors as climate, irrigation, industrial production, and domestic use patterns.
- Maximum day demand – The maximum daily water demand is used to size booster pumping stations that serve areas with storage reservoirs. This measure of demand is also used along with fire demands to size storage reservoirs.
- Peak hour demand – The peak hour water demand is used to size pipelines and booster pumping stations that serve pressure zones without reservoirs.

Three water usage rate variations are generally used in the design of water system facilities. These are the average annual demand, maximum day demand and peak hour demand. The annual average, monthly average, and maximum day water demand are calculated from analyzing WTP daily operational data. The analysis allows for the identification of annual average, monthly average, and maximum day water demand based on the period from 1995 to 1999. The average annual demand increased from 3.73 mgd to 4.50 mgd. The highest peak daily demand was 9.47 in August of 1998.6

10.20.4.10 Per Capita Water Demand.

Per capita water demand is a useful demand measure that is derived from historical data. However, it was not used in determining demand in the *2001 Water Distribution Plan* (which uses land use-based demand) or *2004 Water Treament Plant Facility Plan* (which is based on a

6 Data excerpted from *2001 Water Distribution System Master Plan*. Water use has since increased substantially. See Table 10.20.16 for more recent water use data.

water demand growth rate factor.) Table 10.20.9 presents the population for Grants Pass along with the average annual demand during the years 1995 to 1999, which allows for calculation of the average demand in gallons per capita per day (gpcd). Ranging from 190 to 215 gpcd, the average daily water demand for the years 1995 to 1999 is 202 gpcd. Note that this unit demand factor is based on water production and includes all uses: residential, commercial, industrial, public/institutional, and unaccounted. The variation in per capita demand for the different years reflects the regular variation in water use patterns caused by seasonal conditions and possible changes in end user demand characteristics.

For reference, an engineering report of the water distribution system, prepared by CH2M Hill in 1979, identified an average annual demand of 253 gpcd.

Table 10.20.9 Grants Pass Water Use for 1995 to 1999, gpcd*

Year	Population (City Limits)	Average Demand, mgd	Average Demand, gpcd
1995	19,660	3.73	190
1996	20,255	4.11	203
1997	20,535	3.97	193
1998	20,590	4.17	203
1999	20,935	4.50	215
2000	23,170	4.55	196
2001	23,670	4.86	205
2002	23,870	4.98	209
2003	24,470	4.81	197
2004	24,790	5.00	202
2005	26,085	4.75	182
2006	30,930	5.26	170
2007	31,740	5.84	184
13-Year Average	--	5.66	196

Source: For years 1995-1999, Grants Pass Water Distribution System Master Plan, January 2001;
For years 2000-2007, updated 4/2008 by Jason Canady and Jared Voice using population estimates from Portland State University's Population Research Center)

*Demands include all uses, including residential, commercial, industrial, and public/institutional.

NOTE: Not all City residents are connected to the municipal water system. There are also users outside of the City limits that are not included in the population data in this table. Therefore, although the demand for all users is shown, the per capita demand shown likely overstates actual per capita use.

10.20.4.11 Unaccounted for Water.

All water distribution systems experience losses of water during transmission from the treatment plant to the end user. These losses, known as unaccounted for water, result from many situations including un-metered customers, transmission system leaks, main breaks, faulty meters, fire fighting activities, system flushing, and other miscellaneous hydrant uses. Thus, the total volume of water metered for all end users is always somewhat less than the volume of water produced at the WTP.

Since the City meters water use for all customers, a comparison of water billing records and treatment plant production data provides a good estimate of the volume of unaccounted for water in the system. Table 10.20.10 shows the estimated volume of unaccounted for water in millions of gallons and also as a percentage of total production during the period of 1998 through 2000. Since a water loss rate of 10 to 15 percent is considered good, the calculated unaccounted for water rate indicates that the distribution system is in good condition. The City is also conducting

several programs that will reduce the unaccounted for water rate. The programs include residential meter replacements, commercial meter upgrades, and improved monitoring of hydrant use.

Table 10.20.10 Unaccounted for Water; 1998-2000

Year	Million Gallons	Percent of Total Water Production
1998	146	9.6%
1999	190	11.6%
2000	177	10.9%

Source: Grants Pass Water Management Plan,. June 2002

10.20.4.12 Unit Demand Factors by Land Use Pattern.

Water demand factors related to land use patterns are used to analyze water demand in the community. Based on historical billing data provided by the City's Utilities Department for 1998 and 1999, Table 10.20.11 shows water demand within each land use pattern classification: commercial, single family residential, and multi-family residential showing the annual average and percent of total annual average demand for both years. The commercial classification includes general business, industrial, institutional and governmental-public land use categories. As indicated in the percentage summary of annual average demand by land use, the single-family residential classification accounts for nearly half of the water used in Grants Pass.

Table 10.20.11 Grants Pass Water Use by Customer Class for 1998 and 1999

	Demand			
	Commercial	Multi-Family Residential	Singe-Family Residential	Total
1998 Annual Average	1.36	0.61	1.80	3.77
Percent of total annual average demand	36.0%	16.2%	47.8%	100%
1999 Annual Average	1.40	0.67	1.91	3.98
Percent of total annual average demand	35.2%	16.8%	48.0	100%

Source: Grants Pass Water Management Plan, June 2002

To develop a unit demand factor for the three different land use patterns, the water demand data presented in Table 10.20.11, above, are combined with estimated areas for each of the land use classifications. Table 10.20.12, below, summarizes the acreages by land use classification and pressure zone for all areas receiving water service from the City of Grants Pass, including those within the Urban Growth Boundary and North Valley. This summary was derived from analysis of a distribution system for Pressure Zones 1 through 4. The acreage for Pressure Zone NV is based on an estimate of the properties connected to the system in the North Valley area. The quotient of water demand and acreage yields a unit demand factor for each land use classification in gallons per acre per day (gpad), as presented in Table 10.20.13 below. Table 10.20.11 above does not include unaccounted for water, the calculation of these unit demand factors includes an allowance for unaccounted for water.

Table 10.20.12 Land Use by Pressure Zone (Acres)

	Customer/Class	PZ/1	PZ/2	PZ/3	PZ/4	PZ/NV	Total	%
	Commercial	924	125	57	0	40	1,146	32%
	Residential	1,197	594	114	67	5	1,977	56%
	Multi-Family	358	42	35	0	0	435	12%
Total		2,479	761	206	67	45	3,558	100%
%		70%	21%	6%	2%	1%	100%	

*The Commercial customer class includes commercial, industrial, and public connections to the system.
Source: Grants Pass Water Distribution System Master Plan, January 2001

Table 10.20.13 Unit Demand by Customer Class

Customer Class	1999 Average Demand (mgd)	Land Use Area (Acres)	Average Unit Demand (gal/acre day)
Commercial/Industrial/Public	1.56	1,146	1,400
Multi-Family Residential	0.75	435	1,700
Single-Family Residential	2.13	1,977	1,100

*The 1999 average demand is based on billing records plus 11.6 percent to reflect unaccounted for water.
Source: Grants Pass Water Management Plan, June 2002

10.20.4.13 Peak Hourly Demand.

The peak hour demand on the water distribution system typically occurs during the hottest, driest period of the year when customers are heavily irrigating landscaped yards and parks. For the City of Grants Pass service area, the peak hour demand usually happens in the month of August during the peak day demand. In order to evaluate this peak hour demand, hourly water level data was collected from each of the reservoirs in the distribution system during the summer of 1999. This data was analyzed in combination with the water production rate for the water treatment plant to identify the peak hour demand on each day for which data was available. Based on this analysis, the peak hour demand is estimated to be 4.5 times the average annual demand.

10.20.4.14 Historical Peaking Factors.

The water demands observed over the five years from 1995-1999 can also be expressed as a ratio to the annual average demand known as peaking factors. Although peaking factors vary significantly from user to user, these historical peaking factors are useful for comparing system-wide water use patterns in Grants Pass to other communities and for projecting future water use patterns. Table 10.20.14 identifies Grants Pass water demand patterns for 1995 to 1999 and shows peaking factors based on ratios to annual average demand. The identified peak hour demands for the system are not actual measurements, but rather estimates derived from the reservoir level analysis conducted during the summer of 1999 as described above. Tables 10.20.14 and 10.20.15 below summarize the average peaking factors for the system. These values are fairly typical for a Western Oregon community. In general, they are slightly higher than the peaking factors for Corvallis and slightly lower than values for Portland.

Table 10.20.14 Grants Pass Maximum Month, Peak Day, and Peak Hour Demand Ratio

Year	Annual Average Demand, (mgd)	Maximum Month Demand, (mgd)	Peak Day Demand (mgd)	Peak Hour Demand, (mgd)	Ratio of Maximum Month to Annual Average Demand	Ratio of Peak Day to Annual Average Demand	Ratio of peak Hour to Annual Average Demand
1995	3.73	6.48	8.32	17.01	1.74	2.23	4.56
1996	4.11	7.22	9.09	18.60	1.76	2.21	4.52
1997	3.97	6.82	8.83	18.10	1.72	2.22	4.56
1998	4.17	7.62	9.47	19.40	1.83	2.27	4.66
1999	4.50	7.79	9.35	19.20	1.73	2.08	4.26
5 Yr. Avg.	4.10	7.18	9.01	18.46	1.76	2.20	4.51

Source: Grants Pass Water Distribution System Master Plan, January 2001

Table 10.20.15 Peaking Factor Summary

Description	Factor
Maximum month demand	1.8
Maximum daily demand Average for city	2.2
Peak hourly demand Average for city	4.5

NOTE: The average demand multiplied by the peaking factor yields the respective peak demand.
Source: Grants Pass Water Distribution System Master Plan, January 2001

The following table identifies Grants Pass Water demand patterns for the years 1995 through 2006. The data used in this table was obtained from the City Water Treatment Plant supervisor in 2007 to provide updated information that was not included in the previous planning documents referenced in this chapter. This more recent data shows that water demand has continued to increase since the previous plans were completed. Note that the more recent data was not contained in the *2001 Water Distribution System Master Plan*, the *2002 Water Management Plan* or the *2004 Water Treatment Plant Facility Plan*. The capital improvement and implementation plans developed in these documents and referenced later in this chapter pre-dated this more recent data.

Table 10.20.16 Grants Pass Maximum Month, Peak Day and Peak Hour Data, 1995-2006

[illegible]	Annual Average Flow (mgd)	Maximum Month Demand (mgd)	Peak Day Demand (mgd)	Peak Hour Demand (mgd)
	Finished	Finished	Finished	Finished
1995	3.93	[illegible]	8.32	17.69
1996	4.23	[illegible]	9.09	19.04
1997	4.03	[illegible]	8.83	18.14
1998	4.24	[illegible]	9.47	19.08
1999	4.56	[illegible]	9.35	20.52
2000	4.55	[illegible]	9.73	20.48
2001	4.86	[illegible]	9.25	21.87
2002	4.98	[illegible]	10.54	22.41
2003	4.81	[illegible]	10.31	21.65
2004	5.00	[illegible]	10.17	22.50
2005	4.75	[illegible]	10.52	21.38
2006	5.27	[illegible]	11.69	23.72
12Yr Avg	4.60	[illegible]	9.77	20.71

NOTE: Annual average flow, maximum month demand and peak day demand obtained from Water Treatment Plant data; Peak hour demand is an estimate calculated by multiplying the average flow by the actual 1995 to 1999 peaking factor average of 4.5 (see Tables 10.20.14 & 10.20.15)
Data Source: Jason Canady, City of Grants Pass Water Treatment Plant Supervisor

10.20.4.15 Future Water Demand.

NOTE: The information presented in this section was excerpted from the *Grants Pass Water Distribution System Master Plan*, prepared by West Yost and Associates in January 2001.

Committed Service Areas.

The land use demand factors developed in the previous sections provide a basis for projecting future water demand in the Grants Pass service area. The land use demand factors can be used in conjunction with land development projections to estimate water demand.

Although the timing of land use development within the UGB is unknown, information is available regarding the current zoning designation for all properties within the UGB. Table 10.20.17 summarizes the acreage of properties with the UGB according to land use, differentiating between properties that are were receiving water service as of January 2001 and those that will connect to the water distribution system in the future. Using the unit demand factors developed for these land use classifications, the table also projects average annual water demand at the UGB build-out condition. This analysis assumes the existing mix of residential/commercial properties will stay the same in the future.

Table 10.20.17 Land Use Based Water Demand Projections for UGB Build-Out

Land Use	Existing Acreage	Future Acreage	Total Acreage	Unit Demand, gallons/acre-day	Estimated Average Annual Demand, mgd
Commercial	1,146	598	1,744	1,400	2.4
Single-Family Residential	1,977	2,419	4,396	1,100	4.8
Multi-Family Residential	435	440	875	1,700	1.5
Total	3,558	3,457	7,015	4,200	8.7

Source: Grants Pass Water Distribution System Master Plan, January 2001

Based on this estimate of the UGB build-out average annual demand, the future maximum month, maximum day, and peak hour demand can be estimated using historical peaking factors. Table 10.20.18 summarizes water demand at the time of the *2001 Water Distribution System Master Plan* and projections for the build-out condition.

Table 10.20.18 Water Demand Summary for Grants Pass

	Current Water Demand, mgd	Future Total Water Demand, mgd
Average Annual	5.85	9
Maximum Month	10.13	16
Maximum Day	11.87	20
Peak Hour	23.72	40

Source: Grants Pass Water Distribution System Master Plan, January 2001 (Updated Jason Canady, 3/08)

Potential Service Areas Independent of UGB.

The City has also evaluated its potential to serve properties outside the Urban Growth Boundary, mainly related to the existing distribution system serving the Merlin / North Valley area, and some additional areas contiguous to the UGB.

One location with potential for future water system expansion is Merlin / North Valley. A portion of this area, including the North Valley Industrial Park and several residents along Merlin Road, is being serving by a service connection to the City at NW Highland Ave. near NW Vine St. (as of 2008.) The North Valley pump station was constructed at this connection to pump water from the City's Pressure Zone 3 to fill a 1.2 million gallon reservoir though an 8-inch pipeline in Highland Ave. The reservoir is located approximately one mile north of the North Valley pump station. The water coming out of the storage tank then feeds the Merlin / North Valley system by gravity through a 1.6-mile long 16-inch pipeline. The North Valley pump station consists of three booster pumps with a total capacity of 1,070 gallons per minute (gpm).

The existing North Valley Reservoir and Pump Station have the capacity to accommodate some additional demand in the area. Based on a Technical Memorandum issued by West Yost Associates in December 2007, in addition to serving existing users, there is adequate capacity to serve the airport, Manzanita Rest area, the North Valley schools and Paradise Ranch. To expand service beyond these specific users, including adding any additional residential users, would require significant upgrades to the system, including pipeline improvements and an upgrade of the North Valley pump station. Because of limited space within the existing pump station vault, it may be necessary to relocate the pump station to accommodate any expansion. Additionally, water storage issues will need to be addressed if the service expansion is large enough. The City can continue to provide water service to specific properties within Merlin / North Valley through

individual service agreements; however there are no additional obligations to provide service to properties other than those which are currently served (as of 2008.)

In addition to properties within the UGB, there are also properties contiguous to the UGB that are potential candidates to receive water service in the future. Based on information obtained by West Yost Associates from City staff in about 2001, a rough estimate of the area of these properties is 400 acres. Assuming that these properties largely fall within the single-family residential and industrial land use classifications, the additional acreage would increase the annual average demand on the system by approximately 0.5 mgd.

10.20.5 CITY OF GRANTS PASS WATER SYSTEM CAPITAL IMPROVEMENTS PROGRAM (CIP)

Based on the evaluation of existing system performance and future expansion requirements presented in Section 10.20.4, and upon projected improvements required for ensuring the Water Treatment Plant (WTP) continues to serve the City's needs for the next 20 years and beyond, this section integrates the projects into a staged Implementation Plan for WTP improvements and Capital Improvement Program (CIP) for water distribution system improvements (including pipes, pump stations, etc.)

Table 10.20.19 identifies the WTP facility improvements required to meet the build-out of the City of Grants Pass UGB. The Implementation Plan for WTP improvements was initially completed by MHW Americas, Inc. in 2004 as part of the *City of Grants Pass Water Treatment Plant Facility Plan.* The Plan was also included in the Technical Memorandum that was produced by Parametrix in March 2005 and adopted by City Council Resolution No. 4954 in May of 2005.

Table 10.20.19 City of Grants Pass Water System
Implementation Plan for WTP Improvements (2003 Dollars)

Fiscal Year	Improvements	Estimated Project Costs ($1000)
2004/2005	Intake Modifications (Engr. And Permitting)*	400
2005/2006	Intake Modifications (Engr and Construction)*	500
	Filter Upgrades (Engr. And Construction*	200
	Basin Modifications (Engr. and Construction)	200
2006/2007	Intake Modifications (Construction)*	700
	Filter Upgrades (Construction)*	400
	Basin Modifications (Construction)	400
2008/2009	Filter Gallery Upgrades (Engr. And Construction)	480
	Solids Handling	800
2009/2010	Filter Gallery Upgrades (Construction)	510
	Chemical System Upgrades (Engr. And Const.)	53
2010/2011	Chemical System Upgrades (Construction)	138
	Sludge Removal Systems (Engr. And Const.)	80
	New Storage Building (Engr. And Construction)	27
2011/2012	Sludge Removal Systems (Construction)	239
	New Storage Building (Construction)	53
2012/2013	Emergency Generator for 5 mgd (Engr. and Const.)	319
2024	Expand Capacity to 30 mgd	7,813

*Completed as of September 2007, per Jason Canady, City of Grants Pass Water Treatment Plant Supervisor
Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005,
Adopted By Council Resolution No. 4954, 5/6/2005

In addition to the capital improvements presented above, the City should also implement the following efforts for the Water Treatment Plant (WTP) over the next few years:

- Continue to explore alternative coagulation options to reduce solids production, improve plant performance and reduce operating costs;
- Continue collecting Cryptosporidium samples from the Rogue River to determine "bin classification" according to the LT2ESWTR;
- Develop a DBP sampling program based on the proposed regulations, in conjunction with State of Oregon DHS, to monitor for trihalomethanes (THMs) and haloacetic acids (HAAs), to verify compliance with the proposed Stage 2 D/DBP Rule;
- Complete a Seismic Vulnerability Study; and
- Assess the viability and costs of the sludge handling and disposal program, of which evaluation and implementation has been ongoing for several years and is anticipated to continue for the next 3 to 5 years (as of 2007.)

As of November 2007, it is anticipated that over the next 2 to 3 years the City will verify that it can meet the LT2ESWTR and D/DBP Rule with the existing plant. LT2ESWTR requires 2 years of data collection. The data has been collected once and the WTP is currently completing a second round to ensure compliance with the rule. The stage 2 D/DBP rule requires extensive planning and testing to determine new sampling points before compliance can be determined. If compliance is ultimately determined to be unlikely, then the City may have to implement an alternative disinfection scheme at the WTP.

The City should periodically monitor plant performance and water demands over the next 10 years as it makes capital improvements and to verify that planned improvements are still required. An update of the WTP Facilities Plan should be completed in 5 to 10 years depending on water demands and regulations, including a review of plant expansion requirements.

The WTP is capable of being expanded to approximately 30 mgd with major modifications. Based on current growth estimates, the plant expansion will not be required until the middle to end of decade 2020. The estimated project cost for a plant expansion to 30 mgd is $7.5 million dollars in 2003, which minimizes the use of additional footprint on the existing site. It is recommended that the City assess available property for a future new plant to expand and partially replace the existing plant within the next 50 years.

The following CIP identifies specific improvements for each of the first five years and for the full build-out of the City's service area. For each of the recommended projects, the CIP also presents cost estimates based on the unit construction costs identified on pages 7-1 of the *Grants Pass Water Distribution System Master Plan*, January 2001, West Yost & Associates. Cost estimates were adjusted by Parametrix in its Technical Memorandum dated March 2005 to reflect 2005 dollar amounts.

Table 10.20.20 City of Grants Pass Water System Capital Improvements Project List & Estimated Capital Costs 2000-2005 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Pump Stations	
Hilltop/Harbeck Heights Fire Pumps	60
Rogue Community College Pump Station	280
Subtotal	340
Pressure zone boundary modifications	114
Pressure reduction valves	149
P-101 West Harbeck to Allen Creek	62
P-103 Leonard Street Looping	46
P-104 Lower River Road	86
P-105 Prospect Avenue Looping	16
P-106 Hawthorne to Crescent	427
P-107 9th to 10th at Midland	139
P-108 Sherman Lane to Tokay Heights	259
P-109 Marion Lane	131
P-110 C St to D St Loop	56
P-113 Bridge to Brownell	325
P-114 Lincoln Rd Looping	95
P-115 10th and Savage Tie-In	8
P-202 Redwood Ave Looping North	200
P-203 Redwood Ave Looping South	202
P-204 Rogue Community College Extension	4,372
P-207 Williams Hwy Extension	53
P-220 Southeast North Street Extension	26
P-221 Shannon Lane Extension	55
P-222 Lincoln Road Extension	118
P-223 Ament Road Extension	754
P-224 Starlite Connector	290
Subtotal	7,983
Total	8,323

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005; Subtotal and total dollar amounts have been adjusted by City staff due to mathematical errors that appear in original document.

NOTE: Several of the above-listed projects have been fully or partially completed. Uncompleted projects will continue to be identified and included in the work plan / budget for completion.

Table 10.20.21 City of Grants Pass Water System Capital Improvements Projects List & Estimated Capital Costs 2005-2010 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Pipelines	
P-208 Williams Hwy Looping	336
P-214 Rogue River Hwy Extension	749
P-225 Starlite Extension	114
Subtotal	1,199
Pipeline Replacement	
12,500 feet total replacement	1,124
Subtotal	1,124
Total	2,323

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

Table 10.20.22 City of Grants Pass
Water System Capital Improvements Projects List &
Estimated Capital Costs 2010-2020 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Treated Water Storage	
Reservoir No 12	1,955
Reservoir No. 14	972
Reservoir No. 16	1,097
Reservoir No. 17	1,543
Reservoir No. 13 (replacement)	972
Subtotal	6,539
Pipelines	
P-206 Reservoir No. 12 Extension	488
P-209 Reservoir No. 17 Extension	111
P-215 Fruitdale Dr Extension	892
P-216 Cloverlawn Loop	64
P-218 Cloverlawn to Crestview Loop	162
P-219 Reservoir No.16 Extension	207
P-226 Greenfield Rd Loop	273
P-229 Reservoir No. 14 Extension	78
Subtotal	2,275
Pipeline Replacement	
25,000 feet total replacement	2,248
Subtotal	2,248
Total	11,062

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

Table 10.20.23 City of Grants Pass Water System
Capital Improvements Projects List and Estimated Capital Costs 2020 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Treated Water Storage	
Reservoir No. 10	1,783
Subtotal	1,783
Pump Stations	
Treatment Plant Pumps	457
Subtotal	457
Pipelines	
P-217 Reservoir No 10 Extension	210
Subtotal	210
Total	2,450

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

10.20.6 PRIVATE WATER UTILITIES

All domestic water services within the UGB area that are not served by the City of Grants Pass system derive their water supply from wells. Prior to requirements for subdivisions and other developments to connect to City water, developments were often served through private water systems. Private water companies supply water through small distribution systems mostly to subdivisions, motels, and mobile home parks. A list of private water systems and approximate populations served by each is available through the United States Environmental Protection Agency. Private utility companies have developed various areas within the City's Urban Growth Boundary,

especially south of the Rogue River. Most remaining systems were constructed prior to requirements that they be constructed to municipal system standards. These systems rely upon wells for their supply, and their distribution piping systems consist of small diameter pipe and cannot be used effectively in the Grants Pass Water System. In order for the City to accept these private systems into theirs, the private companies are required to meet or exceed the City's standards. In most cases, the private companies cannot afford the capital improvement costs to comply and naturally resist connecting with the City System.

There are three existing private water systems that are scheduled to be connected to the City Water System within the next two years. These include Bluegrass Park Water Company, Meadow Creek Subdivision and Twilight View Estates7. Per EPA records, the total population served by these three private water companies is approximately 245. At least two additional private water systems, College Oaks and Willow Glenn, were built to City standards and may eventually be connected in the future. However, only the originally-mentioned three systems were planned for connection to City water as of September 2007.

City policy prevents the development of any new private water systems within the UGB.

10.20.7 URBAN SERVICE MASTER PLANS AND MANAGEMENT AGREEMENTS FOR WATER

1. The Grants Pass City Council prepared the *Grants Pass Water Distribution System Master Plan*, 2001; *Water Treatment Plant Facility Plan*, 2004; and the *City of Grants Pass Water Management and Conservation Plan*, 2002 all of which are hereby incorporated into the City of Grants Pass Comprehensive Plan by reference and furthermore, establish the Capital Improvement Program (CIP) and associated costs for keeping pace with build-out of the UGB and serving additional areas outside of the UGB, including portions of the North Valley.

2. The City-County Urban Service Policies, adopted with the UGB in August, 1979, require a public water system with fire flow capacities to serve urban levels of development. A Management Agreement initially adopted January 1981, set out interim development standards to determine domestic and fire requirements for utilizing wells and storage tanks prior to municipal system extension. On August 8, 1998, Josephine County, City of Grants Pass, Harbeck-Fruitdale Sewer District and Redwood Sanitary Sewer Service District signed an Intergovernmental Agreement for the Orderly Management of the Grants Pass Urban Growth Boundary Area. This Intergovernmental Agreement replaces each of the earlier agreements.

3. The City has additional agreements to serve specific properties in the North Valley. These include residential properties in the vicinity of the Merlin Landfill; the North Valley Industrial Park, and Paradise Ranch.

7 Per Bob Hamblin and Kathy Mannon, City of Grants Pass Utilities Division, September 2007

10.20.8 CAPITAL IMPROVEMENT PROJECT IMPLEMENTATION PLAN AND FUNDING MECHANISMS FOR WATER

Implementation and funding plans for the City of Grants Pass Water System are found in each respective Master Plan identified below.

10.20.8.1 Water Treatment Plant.

Please refer to the April 2004, *City of Grants Pass Water Treatment Plant Facility Plan*, Chapter 7 - Implementation Plan, pages 7-1 through 7-8 (MWH).

10.20.8.2 Water Distribution System.

Please refer to the *City of Grants Pass Water Distribution System Master Plan*, January 2001, West Yost & Associates, Chapter 7 – Cost of Recommended Capital Improvement Program, pages 7-1 through 7-7 and associated maps locating each improvement.

10.20.9 WATER SERVICES FINDINGS

10.20.9.1 Water Source.

1. Groundwater from the area's alluvial deposit yields a maximum 50 gallons per minute, which is insufficient for municipal supply. Problems of salt intrusion and a dropping water table further limit the groundwater resource. The only reliable source for the large quantities of potable water required for municipal purposes is the Rogue River.

2. The Rogue River yearly flow is effectively fully subscribed, and can support additional subscriptions only by impounding winter flow behind Lost Creek Dam for dry season release.

3. The City has one "perfected right" (priority date 1888) and three permits (priority dates 1960, 1965, and 1983) for withdrawing 12.5 cubic feet per second (cfs), 25 cfs, 25 cfs and 25 cfs, respectively, for a total of 87.5 cfs from the river for municipal purposes.

4. The following information regarding the Grants Pass Irrigation District (GPID) was included in the previously adopted version of this chapter, and should be updated when more current information can be obtained from GPID.

    a. The Grants Pass Irrigation District has a "perfected right" of 96.7 cfs with a 1916 priority date, and in addition has a Fish and Game "transport right" of 83 cfs and an "as through" right for the turbine lifts of 800 cfs, also diverted at the Savage Rapids Dam Site. The GPID perfected right may be used for municipal purposes. One-third of this right could provide for 30,490 persons, and one-half could provide for 45,730 persons at maximum day demand levels.

    b. The GPID canal and delivery system serves 400 exclusive farm use acres and 7000 urban-suburban acres. The District has 55 miles of major canals and laterals of

which 85% are unlined or uncovered. Many of these canals and laterals serve as major drain ways of the City and urbanizing area, and have been incorporated into the Master Storm Drain Plan of the UGB area.

c. Some 2600 acres of irrigated lands have passed out of the District over the years, due mainly to urban level development and the silting and washout problems associated with winter drainage accommodation. The GPID has elected to continue supplying water through canals and laterals as development proceeds, citing as rationale that the improvements required by development often don't exceed "buy-out" costs, that the City's maximum day demand for water in the summer is thereby reduced, and that the irrigation water, being untreated and un-pressurized is cheaper for both the user and the provider.

10.20.9.2 Water Treatment.

5. The City began providing treated water for domestic use in 1931 (3.5/mgd), with expansions in 1950 (4.5 mgd), 1961 (11.5 mgd), and 1983 bringing the total current capacity to 20 mgd (as of 2007.) The WTP is capable of being expanded to approximately 30 mgd with major modifications. Based on a water demand increase of 2.5 to 3 percent per year, the plant expansion will not be required until the middle to end of decade 2020. The estimated project cost for a plant expansion to 30 mgd is $7.5 million dollars in 2003, which minimizes the use of additional footprint on the existing site. It is recommended that the City assess available property for a future new plant to expand and partially replace the existing plant within the next 50 years

10.20.9.3 Water Storage and Distribution.

6. Waters must be stored to allow for hourly fluctuation in demand ("equalizing storage" at 25% maximum daily demand), must meet fire flow demand when normal consumption is at the maximum daily rate ("fire storage" as per ISO tables), and must provide for water supply during a major disruption ("reserve storage," at 50% maximum day demand, or a 12 hour supply under maximum use conditions and 1 1/3 days supply under average use).

7. The annual average, monthly average, and maximum day water demand are calculated from analyzing WTP daily operational data. The analysis allows for the identification of annual average, monthly average, and maximum day water demand based on the period from 1995 to 1999. The average annual demand increased from 3.73 mgd to 4.50 mgd. The highest peak daily demand was 9.47 in August of 1998.

8. There are eight water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999.

10.30 SANITARY SEWER SERVICES INDEX

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10.30.1 PURPOSE AND INTENT

10.30.1.1 Purpose

The purpose of this section is to identify existing sanitary sewer service facilities and capacities, identify areas of immediate concern, project capacities needed through the planning period, present financial methods of paying for and regulating the service, and present policies of the orderly provision of services. Within and contiguous to the UGB, there are currently three systems providing sanitary sewer service: City of Grants Pass, Harbeck-Fruitdale Sanitary Sewer Service District, and the Redwood Sanitary Sewer Service District.

10.30.1.2 Intent

The intent of this section is to enact the following public facilities sanitary system master plans by ordinance as an update to the Public Facilities Element of the City of Grants Pass Comprehensive Plan:

1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001);
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices – Final Report* (Parametrix, June 2001)
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999) **(NOTE:** This report evaluated alternatives for either upgrading/expanding the Redwood Wastewater Treatment Plant, or conveying the waste water to the Grants Pass Water Restoration Plant (WRP) for treatment. The preferred alternative identified in the report was conveying the waste water to the WRP for treatment, and the work has since been completed.)

10.30.2 NATURAL ENVIRONMENT

The natural environment includes topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items in relation to sanitary sewer collection system planning and analysis. The information provided in this section was excerpted from the *City of Grants Pass Collection System Master Plan*, completed by Parametrix in September 2004.

10.30.2.1 Topography, Geology, and Soils

The topography, geology, and soils of a region can have a significant effect on the design and construction requirements of entire sanitary sewer system. Topography can determine the route and slope of sewer lines, as well as the need for and location of pumping stations. The geology and soil conditions in an area can affect construction costs for pipelines and determine locations for system components.

10.30.2.2 Topography

Grants Pass lies in the Rogue River Valley in the Klamath Mountain Range of Oregon. The Rogue River Valley begins at the base of the surrounding hills and exists as a well-defined stream terrace some 10 to 15 feet above the bed of the Rogue River. The valley slopes toward the river at an

average gradient of 1 to 2 percent. Elevations on the low-lying valley flow range from 880 to 1,100 feet above sea level. The Rogue River traverses the valley in a general east-west direction on an average slope of about 6 feet per mile. Away from the valley floor, the terrain grows steep relatively quickly. Beacon Hill (located northeast of the City) and Baldy Mountain (located southeast of the City) are 2,117 feet and 2,740 feet in elevation, respectively. The Siskiyou Mountains, part of the Klamath Mountains, lie to the south and west of Grants Pass. To the northeast, a spur connects the Klamath Mountains to the Cascade Range.

10.30.2.3 Geology

The service area contains several major geologic units, including alluvium deposits, diorites and granites, ultramafic and metavolcanic rocks, and gneisses and schists. The alluvium deposits, between 100 and 150 feet thick, formed the valley floor by eroding away from the surrounding rock units. The lava and metavolcanic rock composing Beacon Hill and Baldy Mountain does not weather easily. Its ruggedness has limited development in these areas. The softer granite of Dollar Mountain to the northwest, and various hills to the south and southwest of the city shows greater weathering. The rounded ridges and gentle slopes of these areas have encouraged development.

10.30.2.4 Soils

Weathering of the different geologic units has given the soils of this area a wide range of characteristics. The soils that underlie the developable portions of the Rogue River Valley are of the greatest importance to the collection analysis. Soils with poor drainage can increase the potential for infiltration and inflow (I/I) into the collection system, leading to increased flows to the Water Restoration Plant (WRP.) A survey conducted by the U.S Department of Agriculture (USDA, 1983) identified the soil types found in this area for agricultural purposes. A brief summary of the USDA survey with generalized engineering interpretations is presented below.

The most important soil types in the valley are Newberg fine sandy loam, Barron coarse sandy loam, and Clawson sandy loam. Newberg fine sandy loam is principal soil type in the floodplain and terraced areas of the valley. It occupies a strip along the Rogue River Valley that is generally about a mile in width; however, it narrows to about 2,500 feet at Grants Pass. The soil is well drained and presents no major problems for the collection system. Barron coarse sandy loam occupies extensive portions of the Rogue River Valley and underlies most of the city west of Gilbert Creek. The soil generally occurs upslope from Columbian fine sandy loam and extends as valley fill material into most of the minor tributary valleys. This soil has slightly higher clay content than the Newberg loam, but does not significantly impact drainage or increase I/I impacts.

Clawson sandy loam underlies a major portion of the city east of Gilbert Creek. This soil typically consists of about 1 foot of smooth-textured silt loam overlying a compact silty loam or clay loam subsoil. At a depth of about 30 inches, the subsoil assumes an extremely gritty texture, reflecting the presence of coarse granitic material. The subsoil terminates at shallow depths in coarse granitic rock. The soil is flat lying and poorly drained. Because of the impervious nature of the shallow bedrock, it is waterlogged during the winter and spring months. In some areas, the water table is less than 3 feet below the ground level well into the summer. The high groundwater conditions that accompany this soil type can be a problem when wastewater pipelines lying in the soil have cracks or leaks. Groundwater infiltrates into the cracks and leaks, significantly increasing the flow of liquid to

the WRP.

10.30.2.5 Climate

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of wastewater facilities. Rainfall is especially significant, because it can cause large flow increases in the collection system due to storm water runoff, illicitly connected roof drains, and raised groundwater levels.

10.30.2.6 General Climatic Conditions

The climate of Grants Pass is generally mild, although temperatures below freezing and above 100 degrees Fahrenheit occur for short periods annually. Climate is influenced by the Pacific Ocean, which is located about 60 miles west of the city. The intervening coastal mountains modify the effect of the marine air masses, causing this portion of the Rogue River Valley to receive less annual rainfall and to have fewer cloudy and rainy days than most other portions of Western Oregon. Monthly temperature and precipitation data for Grants Pass are summarized in Table 10.30.1.

Table 10.30.1 Climate Summary[a]

	Temperature, Degrees Fahrenheit					Precipitation in Inches	
Month	Mean Maximum	Mean Minimum	Mean	Highest Recorded	Lowest Recorded	Mean	Greatest Daily
January	47.4	31.1	39.3	69	13	5.0	3.4
February	54.1	32.7	43.4	76	12	4.4	2.2
March	59.8	34.1	47.0	81	22	3.7	2.2
April	65.6	35.8	50.7	93	24	2.0	1.4
May	73.1	40.5	56.8	102	26	1.2	1.5
June	80.8	45.4	63.1	106	33	0.5	1.0
July	88.8	49.5	69.2	109	39	0.4	1.3
August	89.0	48.9	69.0	110	36	0.5	0.8
September	82.7	43.0	62.9	108	29	0.9	2.9
October	70.4	37.4	53.9	98	20	2.1	1.7
November	53.3	34.6	44.0	77	12	5.1	4.8
December	45.7	31.3	38.5	67	-1	5.4	4.0
Annual	67.6	38.7	53.2	110	-1	31.2	4.8

[a] Source: Records of the Oregon Climate Services, 1971-2000

10.30.2.7 Precipitation

Nearly 75 percent of annual rainfall in Grants Pass occurs between the months of November through March. A majority of the annual precipitation is in the form of rain, although about 4 to 5 inches of snow falls each year. Seasonal snowfall rarely exceeds 10 inches, which usually melts immediately.

10.30.2.8 Temperature

Temperatures in Grants Pass usually remain moderate through the winter. Subfreezing temperatures may persist long enough to freeze water in aboveground facilities; however, it rarely lasts long enough to cause freezing in buried facilities. Summers are warm and dry. Temperatures exceed 100 degrees Fahrenheit on an average of 6 days a year. Nighttime temperatures are generally cool, averaging about 51 degrees Fahrenheit during July, the warmest month.

10.30.2.9 Other Climatic Factors

Sunshine is usually abundant during the spring, summer and fall, but the area is generally cloudy during the winter months. Early morning fog occurs frequently during November, December and January. Fog is less common in October and February and rarely occurs during the rest of the year. Wind speed and direction are not routinely measured at Grants Pass. The prevailing wind direction, however, is from the west, approximately parallel to the axis of the Rogue River Valley.

10.30.2.10 Water Resources

The principal water resources in the service area are surface water from The Rogue River and its tributaries and groundwater from the alluvium covering the river valley. The water resource most important for this Plan, the Rogue River, drains 2,460 square miles above Grants Pass before traversing the study area. The Rogue River is used for the City's potable water supply, irrigation, and recreation.

A number of individual wells rely on area groundwater. The alluvium is the major aquifer, with typical yield of 40 gallons per minute (gpm). Volcanic formations usually yield less water, but in a few highly fractured areas wells have yields as high as 60 gpm. Many of the high yields are not sustainable, as the aquifers are small and substantial drawdown occurs.

10.30.2.11 Water Quality

The *2004 Collection System Master Plan* discusses regulatory framework for both existing water quality and the quality of water discharged from the Water Restoration Plant. The City's NPDES Permit takes into account existing temperature and pollutant loading in setting discharge limits from the Water Restoration Plant. Oregon Administrative Rules (OAR), Division 41, Section 340-41-365, sets standards for water quality in the Rogue River basin. The rules cover dissolved oxygen concentrations, temperature increases, pH values, coliform counts, creation of tastes or odors, toxic conditions that harm aquatic life or affect drinking water, and the formation of sludge. The regulations and associated water quality information relating to wastewater treatment and disposal are discussed in Section 10.30.7.

10.30.2.12 Water Quantity

The most recent 30 years of Rogue River flow data collected by the U.S Geologic Survey (USGS) are summarized in Table 10.30.2.

Table 10.30.2 Historical Rogue River Stream Flow Data

Month	Monthly Stream Flow Data (1973-2002)[a]		
	Maximum (ft^3/s)	Minimum (ft^3/s)	Mean (ft^3/s)
January	16,610	1,348	5,123
February	10,960	1,162	4,512
March	10,760	1,099	4,364
April	8,395	997	4,076
May	6,428	1,538	3,703
June	4,572	1,016	2,796
July	3,484	974	2,060
August	3,080	878	2,009
September	2,642	1,098	1,724
October	2,282	1,008	1,503
November	9,086	1,160	2,833
December	17,620	1,386	4,866

[a] Data from US Geologic Survey Rogue River Monitoring Station (14361500) at Grants Pass, Oregon.

Flows in the Rogue River can fluctuate widely from year to year. The largest recorded discharge, 152,000 cubic feet per second (cfs), occurred during a December 1964 flood. The lowest recorded discharge was minimum day of 606 cfs during 1968. Reservoirs have since been constructed in the Rogue River basin to provide storage of high wet weather flows for release during dry weather periods.

10.30.2.13 Flood Potential

It is necessary to identify flood-prone areas in order to safely locate wastewater collection and treatment system components. A detailed description of the flood history, mapped locations, and an evaluation of the degree of hazard are found in the Natural Hazards Element of the City's Comprehensive Plan, 1983. The City has adopted Ordinance 4471, which prohibits development in the floodway without a certified "No-Rise" Analysis. Development in the floodway fringe is permitted provided the main living floor is elevated at least one foot above the 100-year Base Flood Elevation, or for nonresidential structures, floodproof construction techniques are utilized.

10.30.3 DEMAND FACTORS

Sewerage demand is driven by factors such as population growth, land use, and to a lesser extent, infiltration from GPID. This section examines the factors driving demand. Unless otherwise noted, the information provided in this section was excerpted from the *City of Grants Pass Collection System Master Plan*. This plan, completed by Parametrix in September 2004, used a planning horizon through the year 2020 and a horizon of 2060 for sizing the distribution system.

10.30.3.1 Population

Consideration of population trends is crucial to long-term sewerage planning. In order to size new facilities and expansions, historical population trends must be examined to predict future population growth. The Grants Pass area has experienced steady population growth since the 1920s. This increase has been in line with the national population trend of people moving to the west and

southwest from the northeastern and central states and to rural areas from urban areas. The population in Josephine County rose during the 1970s (5.11 percent annually), but slowed dramatically during the 1980s (0.6 percent annually). The City of Grants Pass grew more rapidly than the county during the last decade (1.5 percent annually).

10.30.3.1.1 Estimate of Population Equivalent for Sewer Service Area

The 1990 census lists the population of Grants Pass at 17,424. Population within the city limits in 2003 was estimated at 22,444. This is the base year population estimate used for the *2004 Collection System Master Plan.* Population estimates completed after the *2004 Collection System Master Plan* are presented in the Population element of the Comprehensive Plan.

The population within the sewer service area consists primarily of the population within the city limits, and the Harbeck-Fruitdale and Redwood areas outside the city limits. Prior to 2001, the City's Water Restoration Plant (WRP) treated sewage only from within the city and Harbeck-Fruitdale area. The *2004 Collection System Master Plan* estimated a 2003 population of 4,620 for Harbeck-Fruitdale, based on sewer connection and billing records. In addition, the City began serving the Redwood Sanitary Sewer Services District (RSSSD) in 2001, and has subsequently converted the Redwood Treatment Plan to a pump station and sewage is now pumped to and treated at the City's WRP. The *2004 Collection System Master Plan* used a 2003 estimated population of 5,714 for the Redwood area. Based on the estimates for the City, Harbeck-Fruitdale and Redwood, the sewer service area population was estimated to be 32,778 in 2003. It is also necessary to take into account the commercial and industrial contribution as a form of population equivalent. The *2004 Collection System Master Plan* assumed that commercial/industrial population equivalent is 35 percent of the total residential population, which equals 11,472. Therefore, the total sewer service area population equivalent was estimated to be 44,250 in 2003.

The *2001 Wastewater Facilities Plan Update* projected the future sewerage area to also include Merlin / North Valley. This area lies outside the UGB to the north of the city and has largely been dependent on on-site sewage treatment systems. The subsequent *2004 Collection System Master Plan* acknowledged that although this area may begin contracting with the City for wastewater services in the future, there are no current plans for service expansion. Therefore, consideration of flows from the North Valley area was not addressed in the *2004 Collection System Master Plan.* In summary, the *2001 Wastewater Facilities Plan Update* would provide sufficient capacity of the WRP to accommodate sewage from Merlin / North Valley. However, the *2004 Collection System Master Plan* does not plan for facilities to collect and convey sewage from this area to the WRP.

10.30.3.1.2 Projection of Population Equivalent for Sewer Service Area

The *2004 Collection System Master Plan* developed a future equivalent population projection for the service area by applying sub-area growth rates from the City's Comprehensive Plan to the base population estimates for the sub-areas and adding 35% to the resulting population for commercial/industrial population equivalent. Estimated growth rates were respectively: 1.5% for Grants Pass, 1.6% for Harbeck-Fruitdale and 3.1% for Redwood. Maintaining the commercial/industrial equivalent as 35% of the population share in 2020 resulted in an estimated 1.8% growth rate in commercial/industrial equivalent population. As of 2003, the Grants Pass WRP was serving an estimated population equivalent of 44,250 (including the 35 percent

commercial/industrial equivalent.) After applying these growth rates, year 2020 service area population equivalent was estimated as 60,157. For years beyond 2003, it is likely that the city limits and UGB would have enlarged from what existed in 1997. The Redwood and Harbeck-Fruitdale populations would probably then be reflected in the city limit population.
A summary of the population projection used in the *2004 Collection System Master Plan* is shown in Table 10.30.3. In addition to projections for the years 2020 and 2025, the plan also created a projection assuming all available land was at saturation levels of development, which was referred to as Year 2060. The 2060 projections provide a good representation of the build-out conditions that are typically used to determine the size of long-term infrastructure improvements, such as collection system pipelines.

Table 10.30.3 Population Growth by Area

Area	1997 Population	Average growth rate (% per year)	Population Projection			
			2003	2020	2025	2060
City Limits	20,526	1.5	22,444	28,908	31,143	52,440
Harbeck-Fruitdale	4,200	1.6	4,620	6,053	6,550	11,417
Redwood	4,758	3.1	5,714	9,600	11,186	32,563
Commercial / Industrial Equivalent	10,319	1.8	11,472	15,596	17,107	33,747
Total Service Area Population Equivalent:	39,803		44,250	60,157	65,986	130,167

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004

10.30.3.2 Land Use

The *2004 Collection System Master Plan* used land use estimates to generate wastewater flow estimates. These estimates were used to determine the size of long-term collection system improvements. However, land use data was not used in determining plant capacity. Plant capacity was determined exclusively by utilizing the population projections found in the previous section, and applying a 35 percent commercial / industrial equivalency factor. The land use data used in developing the flow analysis can be found in Section 2 of the *2004 Collection System Master Plan.*

10.30.3.3 Grants Pass Irrigation District

The Grants Pass Irrigation District was formed in the early 1920s to supply irrigation water to land located between the town of Rogue River and the confluence of the Applegate and Rogue Rivers. Currently about 7,700 acres of agricultural and residential lands are irrigated. The district has water rights to divert up to 150 cfs from the Rogue River during irrigation season. A series of canals constructed by the district carries the water from Savage Rapids Dam throughout the area covered by the district. Many residents in the sewerage service area use water from the canals to irrigate landscaping and gardens. The canals are also used to carry storm water away from these lands.

The irrigation season typically lasts from about April 15 to October 1. Examination of flows received at the treatment plant shows that the influent flows increase during those months, even though little precipitation occurs during that period. It appears that one source of infiltration and inflow in the Grants Pass sewage treatment plant is irrigation water that has seeped into the ground and infiltrated the sewer system. Although there is a court order to remove Savage Rapids Dam

(now underway), GPID will continue to function as it does now through the installation of pumps that will deliver water to the district's canal system.

10.30.3.4 Hydraulic and Biologic Loading

The *2001 Wastewater Facilities Plan Update* made flow projections based on existing wastewater flow, plus a typical flow per capita assumed for all future connections. These future flow factors in gallons per capita per day (gpcd) are shown in Table 10.30.4. The per capita flow assumed for future connections was typical for new construction at the time. Table 10.30.5 shows the wastewater flow projections made through the 2020 planning period. **NOTE:** The population figures shown in Table 10.30.5 were extracted from the *2001 Wastewater Facilities Plan Update* and include the Harbeck-Fruitdale Sewer Service District, the Redwood Sanitary Sewer Service District, and the Merlin / North Valley area.

Table 10.30.4 Future Flow Factors

Flow Type	Contribution (gpcd)
Average Flow	105
Maximum Month Dry Weather Flow (MMDWF)	100
Maximum Month Wet Weather Flow (MMWWF)	130
Peak Day	275
Peak Wet Weather Flow	360
Summer Average	85
Winter Average	125

Source: Waste Water Facilities Plan Update, Parametrix, 2001

Table 10.30.5 Wastewater Flow Projections

	YEAR					
Types of Flow	1999 (Pop. 42,900)		2010 (Pop. 52,200)		2020 (Pop. 62,700)	
	Calculated (mgd)	Calculated (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)
Summer Max. Month	7.3	173	8.4	160	9.4	150
Winter Max. Month	10.8	256	12.2	233	13.5	216
Peak Day	21.5	514	24.6	471	27.5	438
PWWF	30.1	715	34.0	652	37.8	603
Summer Average	5.0	120	6.0	113	6.8	109
Winter Average	6.4	155	7.80	149	9.1	145
Annual Average	5.7	137	6.9	131	8.0	127

Source: Waste Water Facilities Plan Update, Parametrix, 2001

Table 10.30.6 Organic and Solids Loading (pounds per day)

Type of Loading	Year		
	1999	2010	2020
Average BOD_5	7,490	9,400	11,400
Maximum Monthly BOD_5	10,985	13,700	16,800
Average NH_3-N	327	390	450
Maximum Monthly NH_3-N	636	760	870
Average TSS	8,564	10,500	12,700
Maximum Monthly TSS	11,623	14,300	17,300

Source: Waste Water Facilities Plan Update, Parametrix, 2001

Because the Rogue River is used for salmon spawning, fish passage, and rearing Coho, wastewater discharge requirements are strict. To properly evaluate upgrading/expanding the Grants Pass WRP, an analysis of the facilities wastewater discharge permit limit is necessary. Table 10.30.6 presents the anticipated year 2020 BOD_5 and TSS treatment requirements as mandated by OAR 340-41-375. These values represent the dry weather season BOD_5 and TSS effluent concentrations that the plant should meet monthly.

Table 10.30.7 Anticipated Year 2020 BOD_5 and TSS Treatment Requirements Based on OAR 340-41-375

	Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge, lbs/day		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based on Effluent Quality							
Summer, Dry Weather	8	10	15	-	670	1,000	1,300
Winter, Wet Weather	10	30	45	-	2,500	3,750	5,000

Source: Waste Water Facilities Plan Update, Parametrix, 2001

The DEQ has determined that the Rogue River is a high priority concern for implementing management strategies to attain compliance with water quality standards. The first step in this process is to develop the total maximum daily load (TMDL) pollutant loading allocations. This TMDL will include both point and non-point sources.

The TMDL process is now underway and is currently scheduled to be completed in the next several years. Following TMDL allocation, new discharge permit requirements may be established for the Grants Pass WRP that will modify Table 10.30.7. These new permit requirements may also impact the capacity of the existing plant.8

8 Per Steve Gilbert, Parametrix, June 2007

10.30.4 CITY OF GRANTS PASS SANITARY SEWER SERVICES

10.30.4.1 Service Area

Situated near the Rogue River, the Grants Pass Water Restoration Plant (WRP) serves the City and the Harbeck-Fruitdale area. In the fall of 2000, the plant began receiving sewage from the Redwood area.

The *2004 Collection System Master Plan* estimated that the Grants Pass WRP had a service area population equivalent of approximately 44,250 in 2003. With estimated growth rates of 1.5 percent for Grants Pass and 1.6 percent for Harbeck-Fruitdale, and the addition of Redwood at a 3.1 percent population growth rate, it was estimated that the Grants Pass WRP would be serving an equivalent of 60,157 people by the year 2020.

10.30.4.2 Treatment Plant

Since 1935, the City of Grants Pass WRP has been operating at its current site. Subsequent plant additions occurred in 1953 and 1962. In 1974, the treatment plant was renovated and expanded. More improvements occurred from 1994-96 in response to the Department of Environmental Quality's concerns about the effluent toxicity, and the discharging directly into the Rogue River. The improvements consisted of a fourth raw sewage pump, a temporary belt filter press to improve wet weather solids disposal, two rectangular primary clarifiers, a computer based supervisory control and data acquisition system, and an ultraviolet disinfection system. In January 1999, Brown & Caldwell completed a facilities plan for the Grants Pass Water Restoration Plant (WRP). Also in 1999, the addition of a second Ultraviolet channel was implemented. This allowed the increase of disinfection capacity from 21.5 million gallons per day (mgd) to 43 mgd.9

10.30.4.3 Treatment Level

At the time of the *2001 Wastewater Facilities Plan Update*, the Grants Pass WRP treated 4.5 mgd of average dry weather flow (ADWF), with a record peak storm flow of 26.5 mgd. More recent data has also been compiled. As of April 2008, the WRP treated an ADWF of 5.5 mgd and peak wet weather flow of 30.0 mgd. The WRP is comprised of numerous unit treatment processes for both liquid and solid streams, a control/laboratory building, and a maintenance shop. There is a 27 mgd hydraulic capacity for influent pumping10, screening, and primary treatment, a 13 mgd hydraulic capacity for secondary treatment, and a 43 mgd hydraulic capacity for UV disinfection. Flow exceeding the secondary treatment capacity receives only primary treatment and disinfection. This occurs only a few days a year during wet weather storm conditions.

10.30.4.4 Biosolids Handling and Disposal

Currently, the Grants Pass WRP sends the Class B Biosolids to JO-GRO™, a green waste and biosolid waste composting plant.

To produce Class B biosolids, processing must occur. The process starts with a primary clarifier. Solids, which are thickened in the gravity thickener and the gravity belt thickener, are dewatered by

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9 Information from this section was excerpted from the *2001 Wastewater Facilities Plan Update*. Additional WRP upgrades have occurred since that time (see Water Restoration Plant Capital Improvement Plan, Table 10.30.11).
10 As of January 2008 a project was completed to increase the hydraulic capacity for influent pumping to 45 mgd.

the secondary clarifier to produce waste activated sludge. The primary and secondary sludge are then combined and sent to the 50-foot-diameter anaerobic digester. Dewatered biosolids are loaded into a dumpster and hauled to the Merlin Landfill for the JO-GRO™ composting process.

10.30.4.5 Collection System

Per the *2001 Wastewater Facilities Plan Update*, it was believed that prior to 1927 sewer pipes were constructed of vitrified clay, although records are not available to confirm this. From 1927 to 1964, sewers were constructed of non-reinforced concrete pipe with bell and spigot joints caulked with cement mortar. Since 1964, sewers have been constructed of concrete pipe with bell and spigot joints and rubber ring gaskets. The concrete pipes are caulked at the joints with cement mortar. Over time, the cement caulking dissolves leaving the joints vulnerable to cracking and resulting in infiltration of groundwater and penetration of tree roots.

At the time of the *2001 Wastewater Facilities Plan Update* the City's collection system consisted of approximately 110 miles of gravity sewers, one force main (approximately 2,000 feet long), and three pumping stations. As of 1983, three hundred fifty acres of the downtown area were still served by vitrified clay pipe installed prior to 1927. The portion of the collection system constructed prior to 1964 has shown severe deterioration in both pipe materials and joint integrity. From 1992 to 2001, the city reported 17 sinkholes due to pipe failure.

The City of Grants Pass completed a *Collection System Master Plan* in 1983 (James Montgomery, 1983). A number of the 1983 recommendations for improving the collection system have been implemented. The City has, however, experienced sewer line failures and occasional overflows due to sewer line obstructions. The City has also expanded its collection system to include the Redwood Sanitary Sewer Service District (RSSSD) collection system located west of the City. The Oregon Department of Environmental Quality made completion of an updated Collection System Master Plan a provision in the National Pollutant Discharge Elimination System (NPDES) permit for the City's Water Restoration Plant, issued in December of 2000.

The *2004 Collection System Master Plan* is the most recent in a series of reports/studies focusing on wastewater infrastructure. In 2000, the City commissioned a *Wastewater Facilities Plan Update* (Parametrix, 2001) for improvements to the City WRP. The *Facilities Plan* focused on the WRP but also provides some analysis of the existing collection system to assess the impact of infiltration and inflow (I/I) on peak flow events at the WRP. Additional collection system analyses that were presented in the *RSSSD Facilities Plan* (prepared in 1999 by Parametrix) assessed the RSSSD wastewater collection system. In September 2004, Parametrix prepared the City's latest *Collection System Master Plan*, which provides for the capital improvement programming (see tables 10.30.11, 10.30.14, and 10.30.15) to accommodate orderly and cost-effective methods to operate, maintain and expand the collection system while reducing the risk of system failures.

10.30.4.6 Pump Stations

The topography of the City's service area is such that most of the system is operated under gravity flow conditions. As such, there are only a few pump stations in the collection system. The Webster No. 1 Lift Station, Webster No. 2 Lift Station, and Bridge Street Pump Station are all located in the southwestern portion of the city. Under an intergovernmental agreement, the City also operates two

pump stations serving the RSSSD: the RSSSD pump station, located at the site of the abandoned RSSSD wastewater treatment plant at 4960 Leonard Road, and the Darneille Pump Station at 3100 South River Road.

10.30.4.6.1 City Pump Stations: The collection system includes three pumping stations, two of which are simply lift stations. Design data for all three pump stations is included in Table 10.30.8. The lift stations, Webster Lift Station 1 and Webster Lift Station 2, are both located on Webster Lane and serve the mobile home park in the western section of Basin A. Each station is equipped with two vertical, non-clog, and centrifugal pumps. The pumps in Station 1 are 7.5 hp pumps, each with a capacity of 100 gallons per minute (gpm) at 23 feet total dynamic head (TDH). Station 2 pumps are 3 hp pumps, each with a capacity of 100 gpm at 10 feet TDH.

The third station, designated the Bridge Street Station, is located at the intersection of Bridge Street and Tami Court. Dual (4- and 8-inch diameter) force mains travel east from the pump station along Bridge Street about 1,900 feet and discharge in Manhole B111. The station is equipped with two submersible non-clog centrifugal pumps. Each of the 20 hp pumps has a capacity of 650 gpm at 75 feet TDH. Air injection is also provided for the force main to control sulfides. A 50 kW natural gas fueled engine generator is provided standby power.

10.30.4.6.2 Redwood Sanitary Sewer Service District (RSSD) Pump Stations: The Redwood Conveyance System, which transfers all flow from the old Redwood Wastewater Treatment Plan (WWTP) to the Grants Pass WRP, includes the Redwood Pump Station, the Redwood force main, the Darneille force mains and Darneille Pump Station, and a gravity sewer which brings the transferred flow to the Grants Pass WRP. The Redwood Pump Station is located at the old Redwood WWTP. It is a duplex submersible pump station with a capacity of 0.48 mgd, based on one pump in operation. The station has a Bioxide chemical injections system with a 3,000-gallon chemical storage tank. From the Redwood Pump Station, flow is routed through approximately 10,300 feet of 6-inch-diameter force main to an influent manhole at the Darneille Pump Station.

The Darneille Pump Station accepts the majority of the flow from the RSSSD, as well as the flow pumped from the Redwood Pump Station. The Darneille Pump Station has a firm capacity of 4.2 mgd, based on operation of two of three pumps. The station is a wet well/dry well type station with above-grade electrical panels, generator, and chemical feed system. The chemical feed system is identical to that provided at the Redwood Pump Station, except that the chemical feed pumps are slightly larger. From the Darneille Pump Station, flow is pumped through approximately 17,740 feet of dual 12-inch force main and the 1,000 feet of single 14-inch force main. From the pump station, the dual force mains are routed south to South River Road; then east through road rights-of-way and easements to the south side of the Pedestrian Bridge. The dual 12-inch force mains join into a single 14-inch force main that crosses the Rogue River on the Pedestrian Bridge and then discharges into a gravity sewer, which flows to the WRP. Table 10.30.8 provides pump station data.

Table 10.30.8 Pump Station Data

Location	Year Built	No. of Pumps	Pump Type	Horsepower (hp)	Drive Type	Capacity/Head (gpm)/(feet)
Webster No. 1 Lift Station: East edge of Roguelea Estates	1967	2	Self-priming, vertical close-coupled, non-clog centrifugal	7.5	Constant Speed	100/23
Webster No. 2 Lift Station: All Sports Park -Basin A	1967	2	Self-priming, vertical close-coupled, non-clog centrifugal	3	Constant Speed	100/10
Bridge Street Pump Station: Bridge Street and Tami Court – Basin A	1994	2	Submersible, non-clog centrifugal	20	Variable Speed	650/75
Redwood Pump Station: 4960 Leonard Road – RSSSD	2000	2	Submersible, non-clog, centrifugal	40	Variable Speed	335/163
Darneille Pump Station: 3100 South River Road - RSSSD	2000	3	Immersible dry-pit, screw centrifugal	110	Variable Speed	1,460/180

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004

10.30.5 TREATMENT ALTERNATIVES CONSIDERED

In January 1999, Brown and Caldwell (BC) completed a Facilities Plan (FP) for the Grants Pass Water Restoration Plant (WRP). A concern regarding the future flow and population projections prompted a Value Engineering (VE) Workshop in June 1999. During this VE Workshop with Parametrix, Inc. and the City, these projections were recalculated and other treatment alternatives for both the liquid and solid streams were conceptualized. Also during the VE Workshop, the projected population, flow, Biological Oxygen Demand (BOD), and Total Suspended Solids (TSS) loadings were recalculated. These recalculations and treatment alternatives were incorporated into the *2001 Grants Pass Wastewater Facilities Plan Update* (WFP).

To select the best method for meeting Grants Pass treatment needs in the year 2020, the WFP evaluated three liquid stream alternatives and four solids streams alternatives. The liquid stream treatment includes a conventional expansion alternative, a ballasted sedimentation alternative, and a Zenon process alternative. The biosolids disposal and handling alternatives propose a Merlin Landfill co-compost facility, hauling dewatered biosolids to the Dry Creek Landfill, land application of Class B biosolids, and using an Aerobic Thermophilic Pretreatment (ATP) component. The alternatives are summarized below.

10.30.5.1 Liquid Stream Alternatives

10.30.5.1.1 Upgrades Common to the Liquid Stream Alternatives. All of the liquid stream treatment alternatives have several component upgrades in common. At the headworks, an

additional mechanical bar screen would be installed with a 23.5 mgd capacity. This provides redundancy and a maximum capacity of 47 mgd, which is well beyond the projected future flows. An outfall diffuser would be added to improve and reduce the ammonia toxicity into the Rogue River. Several miscellaneous plant improvements, which existed in the Facility Plan, have also been included in the value engineering alternatives. These improvements include laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion.

10.30.5.1.2 Alternative One – Conventional Expansion. Alternative One is the recommended improvement from the WFP using a conventional expansion approach. By replicating the existing components, this option allows continuing ease of operation due to staff familiarity of the process. This alternative has been modified to treat the peak wet weather flow. A summary of the upgrades include:

- Additional mechanical bar screen with a capacity of 23.5 mgd.
- Removal of the four existing influent pumps and replacement with three 19 mgd pumps
- Additional rectangular primary clarifier. Rehabilitate existing primary clarifiers.
- Two additional aeration basins. Rehabilitate existing aeration basins.
- Two additional 115-foot secondary clarifiers. Rehabilitate existing secondary clarifiers.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, and instrumentation and control system expansion.

10.30.5.1.3 Alternative Two – Ballasted Sedimentation. Alternative Two uses ballasted sedimentation to treat peak flows greater than 13.5 mgd. It is proposed to convert the existing gravity thickener into the ballasted sedimentation tank. The peak flows would be conveyed to this system and then recombined with the main treatment train to receive UV disinfection. Other upgrades to the Grants Pass WRP include:

- Install additional influent pumping capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.
- Odor containment at the influent pump station and mechanical bar screen area,
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary clarifiers would be rehabilitated.
- Addition of a bioselector in the aeration basin to provide filamentous bacteria control, which would improve the settling performance in the secondary clarifiers. The aeration basin would also be modified with fine bubble diffusers, dissolved oxygen control, and motorized gates.
- Two additional 90-foot secondary clarifiers. Rehabilitate the secondary clarifiers.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

10.30.5.1.4 Alternative Three – Zenon Process. Alternative Three incorporated the use of Zenon for secondary treatment. The wastewater can flow directly into the primary clarifiers, thereby eliminating the need for secondary clarifiers. Zenon is a microfiltration membrane system located in a suspended growth biological reactor. For this alternative, the membranes would be placed into the aeration basin to serve as a biological reactor. Other upgrades to the Grants Pass WRP for this alternative include:

- Additional influent pump to meet the projected firm capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.
- Odor containment at the influent pump station and mechanical bar screen area.
- For peak overflows greater than 13.5 mgd, the existing gravity thickener would be converted into a ballasted sedimentation tank.
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary and secondary clarifiers rehabilitated for enhanced performance.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements include: Laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

10.30.5.1.5 Liquid Stream Alternatives Cost Estimate Comparison. A preliminary cost estimate for these liquid stream alternatives is summarized in Table 10.30.9. Engineering, administration, and contingency are included in these costs.

Table 10.30.9 Comparison of Liquid Stream Alternatives

Alternative	Capital Cost (millions)
One- Conventional Expansion	$12.23
Two- Ballasted Sedimentation	$13.59
Three- Zenon Process	$23.96

Source: City of Grants Pass Water Restoration Plan Update, June 2001, Parametrix Inc.

10.30.5.2 Biosolids Disposal and Handling Alternatives

The following subsections generally describe the alternatives for biosolids disposal and handling proposed at the Merlin Landfill co-compost facility, for hauling dewatered biosolids to the Dry Creek Landfill, for applying to land the Class B biosolids, and for using an Aerobic Thermophilic Pretreatment (ATP) component.

10.30.5.2.1 Alternative One– Merlin Landfill Co-compost Facility. Alternative One would co-compost digested primary and raw secondary biosolids. The existing digester would be rehabilitated and used only for digesting primary biosolids. A new component for dewatering digested primary biosolids to 15 percent solids would be installed in an existing building located on site. The biosolids would be hauled to the co-composting facility located at the Merlin Landfill. This facility would produce Class A biosolids, which would be available for public purchase.

10.30.5.2.2 Alternative Two- Dry Creek Landfill. Alternative Two hauls raw primary and secondary dewatered biosolids to the Dry Creek Landfill. These biosolids would be dewatered at the Grants Pass WRP to 15 percent solids using a belt filter press (BFP). The existing press and one new additional BFP would be installed in an existing building located on site. Extra hauling equipment would be required for handling the transport of all the biosolids to the landfill.

10.30.5.2.3 Alternative Three- Land Applying Class B Biosolids. Alternative Three is the recommended alternative from the WFP. It is a continuation of the current biosolids management program of long hauling Class B biosolids to be land applied. However, because of an increase in future biosolids production, contracting with landowners of large parcels of land in Eastern Oregon to expand the land application area would be necessary. Biosolids would be dewatered to 15 percent, hauled to the site in large tractor/trailers, and applied with a manure spreader. During the winter, the biosolids would be stored near the application site. An additional gravity thickener, gravity belt, anaerobic digester, and belt filter press are necessary to meet the future solids production. The existing anaerobic digester would need to be rehabilitated. Class A biosolids can be produced by adding low cost aeration equipment to the future storage building to create a pilot-scale co-composting facility.

10.30.5.2.4 Alternative Four- Aerobic Thermophilic Pretreatment (ATP). Alternative Four is to produce Class A biosolids and distribute them to the public as fertilizer. To accommodate increasing loads in the future, an ATP would be installed instead of adding a new digester. To reduce plastics in the biosolids, a Muffin Monster would be added prior to the ATP. The existing digester would need to be rehabilitated to enhance performance, increase capacity, and repair deficiencies.

10.30.5.2.5 Biosolids Alternatives Cost Estimate Comparison. Preliminary cost estimates for the four biosolids disposal alternatives are in Table 10.30.10. These values include engineering, administration, and contingency costs.

Table 10.30.10 Comparison of Biosolids Disposal and Handling Alternatives

Alternative	Capital Cost (millions)
One – Merlin Landfill Co-compost Facility	$3.60
Two – Dry Creek Landfill	$1.40
Three – Land Apply Class B Biosolids	$11.00
Four – Aerobic Thermophilic Pretreatment (ATP)	$2.30

Source: City of Grants Pass Water Restoration Plan Update, June 2001, Parametrix Inc.

10.30.5.3 Miscellaneous Plant Improvements

- Plant Equipment Audit. A full analysis of the existing component conditions would be conducted. This audit would analyze the remaining life span of each component and develop an operations and maintenance schedule for the 20-year planning period.
- Additional Plant Landscaping. Currently a substantial amount of landscaping at the treatment plant has occurred to promote a good-neighbor environment. However, to continue this effort, additional landscaping would be necessary.

10.30.6 PREFERRED TREATMENT ALTERNATIVES

10.30.6.1 Biosolids Handling and Disposal

For the biosolids handling and disposal, the *2001 Wastewater Facilities Plan Update* found the Merlin Landfill Co-compost Facility (Alternative One) to be the preferred alternative. This alternative would consist of the existing digester, a new dewatering device, and a co-composting facility. Under this solid waste handling system, rehabilitated digesters would be used to treat only primary clarifier solids. The secondary clarifier solids would be combined with the digested primary solids, dewatered in a new dewatering component, and trucked to the new co-composting facility located at the Merlin Landfill.

10.30.6.2 Liquid Stream Treatment

The *2001 Wastewater Facilities Plan Update* found the ballasted sedimentation alternative to be the preferred alternative for the liquid stream treatment. Table 10.30.11 lists the anticipated costs and time-line of when the components or upgrades would occur at the Water Restoration Plant, and a proposed construction schedule follows. Both are excerpted from the *2001 Wastewater Facilities Plan Update*.

10.30.6.3 Capital Improvements Water Restoration Plant

Table 10.30.11 City of Grants Pass
Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001-2004	2005-2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping*	$560,000		$560,000	
Raw Sewage Pipeline to Ballasted Sedimentation	$240,000		$240,000	
Screening Odor Control*	$390,000	$390,000		
Mechanical Bar Screen No. 2*	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000		$3,510,000	
Modify Existing Primary to Combination Clarifier/Thickener	$610,000		$610,000	
Yard Piping*	$270,000		$270,000	
SECONDARY TREATMENT				
Aeration Basin Fine Bubble*	$530,000	$530,000		
Aeration Basin Selector*	$260,000	$260,00		
Blowers and DO Control*	$870,000	$870,000		
Rehabilitate Existing Clarifiers*	$770,000	$770,000		
New Secondary Clarifiers*	$2,400,000	$1,200,000		$1,200,000

Yard Piping*	$770,000	$400,000		$370,000
Motorized Gates*	$340,000	$340,00		
FINAL TREATMENT				
Outfall Diffuser*	$540,000	$540,000		
OTHER PLANT IMPROVEMENTS				
Lab Improvements*	$100,000	$100,000		
Operation Building Repairs and Office*	$130,000	$130,000		
SCADA System Expansion*	$670,000	$250,000	$250,000	$170,000
Equipment Improvements from Audit Results*	$250,000	$100,000	$75,000	$75,000
Plant Landscaping*	$100,000	$40,000	$30,000	$30,000
Public Education*	$50,000	$20,000	$20,000	$10,000
SOLIDS THICKENING AND DIGESTION				
Rehabilitate Existing Digester*	$740,000	$740,000		
Dewatering Centrifuge*	$1,000,000	$1,000,000		
SOLIDS HANDLING OFF-SITE				
Co-composting Facility*	$1,850,000	$1,850,000		
COLLECTION SYSTEM				
Pine Street*	$1,050,000	$1,050,000		
2nd Street	$700,000	$700,000		
Western Avenue	$580,000	$580,000		
Master Plan*	$170,000	$170,000		
TOTALS	$19,680,000	$12,030,000	$5,795,000	$1,855,000

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc. *(1999-2000 dollars)
*Item has been completed or partially completed as of April 2008 (Per Public Works Dept.)

Year 2000

- Install odor containment and control at the influent pump station and mechanical screening areas of the plant.
- Add an anoxic selector basin to the aeration basin.
- Modify the existing aeration basin, convert the existing aeration system to fine bubble diffusers, and add dissolved oxygen control and motorized gates. This would require new or modified aeration blowers.
- Rehabilitate the existing secondary clarifiers to correct short circuiting and flow distribution problems.
- Add a third secondary clarifier.
- Begin laboratory upgrades and improvements.
- Begin operations building repairs and modifications.
- Begin instrumentation and control system expansion.
- Conduct a plant equipment audit.
- Continue to install plant landscaping.
- Develop and institute a public education program.

Year 2005

- Install additional influent pumping capacity.
- Add a second mechanical bar screen.
- Convert the existing circular primary clarifier to a combination primary clarifier/gravity thickener.
- Modify the existing gravity thickener to a ballasted sedimentation tank to treat peak flow.

Year 2010

- Add a fourth secondary clarifier.

10.30.7 RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

The recommended collection system improvements presented below are based upon deficiencies in the pipeline hydraulic capacity that were identified in the Hydraulic Analysis found in Section 6 of the *2004 Collection System Master Plan* (Parametrix) and the needed collection system improvements that were identified in the Maintenance and Reliability Analysis presented in Section 7. A Capital Improvement Program (CIP) for the collection system was developed based on a priority analysis of these improvements.

10.30.7.1 Collection System Goals

Three goals were used in developing a CIP for the collection system, identifying the improvements required and a schedule for implementation.

- Service to Saturation-Level Populations: All of the needed improvements were selected to serve the 2060 populations that could occur in the collection system service area.
- Attention to Critical Improvements: Attention was given to collection system pipelines and sub-basin service areas that City staff identified as problems areas. City staff experience in the frequency of maintenance of various pipelines and witnessing surcharged pipelines (hydraulic capacity deficiencies) have been used, particularly to schedule needed improvements.
- Distribution of Capital Expenditures: In the selection and scheduling of the required collection system improvements, the goal was to develop a CIP that is financially viable for the City of Grants Pass.

10.30.7.2 Hydraulic Capacity Improvements

Based on the hydraulic analysis conducted on the wastewater collection system serving the City of Grants Pass, five capital improvement projects have been identified that need to be completed in the next 20 years. These improvements are necessary to maintain adequate conveyance system capacity in the collection system and prevent sewer system overflows. These five projects are found in Section 8 of the *2004 Collection System Master Plan.*

10.30.7.3 Maintenance and Reliability Improvements

Very old and small pipelines serve the downtown area of Grants Pass. The pipelines are greater than 60-years-old, and often only 6-inches in diameter. Based on the Maintenance and Reliability Analysis conducted on the wastewater collection system, six areas of the City are served by these old small-diameter collection lines need further investigation and will likely require repair or replacement. These areas are described in Section 8 of the *2004 Collection System Master Plan.*

10.30.7.4 Estimated Cost of Improvements

By using the gravity sewer pipeline construction cost unit prices, total project costs for each of the recommended pipeline improvement projects are listed in the following table.

Table 10.30.12 Cost of Recommended Pipeline Improvement Projects (in $1M)

Project	Length (feet)	Preliminary Diameter (inches)	Construction Cost ($/foot)	Base Construction Cost	Construction Contingency 40%	Engineering Legal and Administration 30%	Total Estimated Project Cost
Pine Street*	7,010	24	$200	$1.40	$0.56	$0.59	$2.55
Western Avenue	4,720	18	$170	$0.80	$0.32	$0.34	$1.46
Mill Street	9,140	21	$185	$1.69	$0.68	$0.71	$3.08
7th Avenue	4,530	18	$170	$0.77	$0.31	$0.32	$1.40
Nebraska	2,710	18	$170	$0.46	$0.18	$0.17	$0.81

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004

*Item has been completed or partially completed as of April 2008 (Per Joey Wright, Public Works Dept.)

Similarly, the total project costs for the recommended structural repair areas have been prepared. A minimum diameter of 8-inch sewer pipeline has been assumed to estimate project cost. This is because replacing existing 6-inch-diameter sewer pipe with 6-inch does not meet generally accepted sewer design criteria. Given that different repair/replacement technologies will be implemented in these areas, rather than just assume replacement of all old pipelines with new pipelines to estimate the total project costs in each of these areas, it was assumed that total cost would equal the cost to replace one-half of all pipelines in these areas. These costs are developed and presented in Table 10.30.13 below.

Table 10.30.13 Cost of Recommended Structural Repair Areas (in $1M)

Structural Repair Area	Pipeline Diameter (inches)	Length (feet)	Construction Cost ($/foot)	Base Construction Cost	Construction Contingency 40%	Engineering (Legal and Administration) 30%	Total Estimated Cost*
Pine Street (*completed*)	8 10 12	7,667 402 48	$150 $150 $155	$1.22	$0.49	$0.51	$1.11
5th Street	8 10 12	11,672 1,244 2,606	$150 $150 $155	$2.34	$0.94	$0.98	$2.13
7th Street	8 10 12 18	9,326 400 692 260	$150 $150 $155 $170	$1.61	$0.64	$0.67	$1.46
Subbasin B/C	8 10 12	7,409 945 644	$150 $150 $155	$1.35	$0.54	$0.57	$1.23
Subbasin H	8 10	7,919 1,005	$150 $150	$1.34	$0.53	$0.56	$1.21
Lawnridge-Washington	8 10 12	11,362 612 16	$150 $150 $155	$1.80	$0.72	$0.76	$1.64

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004
*Total of base cost, construction contingency, and engineering divided by two

10.30.7.5 Collection System Capital Improvement Program

Using the prioritization of Collection System Improvements described in Section 8.4 of the *2004 Collection System Master Plan* (Parametrix) and the estimated cost of these improvements presented in Table 10.30.12 and Table 10.30.13, a recommended Capital Improvement Program for the City collection system has been developed and is presented in Table 10.30.14.

Table 10.30.14 Recommended Capital Improvement Program
Grants Pass Collection System

Project	Schedule	Cost ($1M)
Pine Street Sewer*	2004-2006	$2.55
Western Avenue Sewer	2006-2009	$1.46
Pine Street Structural Repair*	2009-2011	$1.46
5th Street Structural Repair	2010-2012	$1.11
7th Street Structural Repair	2013-2015	$1.46
Lawnridge-Washington Structural Repair		$1.64
Mill Street Sewer	2016-2018	$3.08
Sub-basin Structural Repair		$1.23
Nebraska Avenue Sewer	2022-2024	$0.81
Total Collection System Capital Improvement Program:		**$18.08**

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004
*Item has been completed or partially completed as of April 2008 (Per Joey Wright, Public Works Dept.)

10.30.8 REGULATORY AND PROCEDURAL ISSUES

Federal, state, and local regulatory agency policies and procedures affect the installation, upgrades, and operation of the City's wastewater collection system and treatment facility. The impacts of these policies and procedures on wastewater management planning in the Grants Pass area are described below. The discussion of regulations presented is not exhaustive and is focused on those regulations and laws that are relevant to wastewater conveyance and treatment.

10.30.8.1 FEDERAL POLICY

Federal policies that will affect the planning process include the Federal Water Pollution Control Act/Clean Water Act; Safe Drinking Water Act; and the proposed Capacity, Management, Operation and Maintenance Rules.

10.30.8.1.1 Federal Water Pollution Control Act/Clean Water Act

Since its enactment, the Federal Water Pollution Control Act, also known as the Clean Water Act (CWA), has formed the foundation for regulations detailing specific requirements for pollution prevention and response measures. The CWA requires states to adopt water quality standards consistent with federal limitations on pollutant and thermal loading. The standards are to take into consideration the use of the waters for public water supplies; propagation of fish and wildlife; recreational purposes; and agricultural, industrial, and other beneficial uses.

The City's collection system conveys wastewater to the City WRP, where it is treated before discharge to the Rogue River. State policies specifically regulate pollutant and thermal loading to comply with federal policy detailed in the CWA.

10.30.8.1.2 Safe Drinking Water Act

The Safe Drinking Water Act (SDWA) authorizes the Environmental Protection Agency (EPA) to set national health-based standards for drinking water to protect against contaminants that may be found in drinking water. Wastewater flows that are collected in or travel through substandard collection systems have the potential to contaminate drinking water systems. State and local regulations are designed to comply with the SDWA, and prohibit activities that could cause an adverse impact on existing or potential beneficial use of groundwater.

10.30.8.1.3 Proposed Capacity, Management, Operation, and Maintenance Rule

EPA is proposing rules that will govern the manner in which municipalities and special service districts manage and operate wastewater collection systems. The proposed Capacity, Management, Operation, and Maintenance (CMOM) Rule, depending on its final promulgated form, may have a significant affect on collection system development and operation and maintenance (O&M) for the City.

Under the proposed rule, sanitary sewer overflows (SSOs) would be prohibited unless caused by severe natural conditions such as widespread flooding, earthquakes, or other natural disasters. Owners of collection systems would be required to provide adequate capacity for peak flows in all parts of the system, monitor and report on SSOs, and make the SSO control program and reports available for public review.

There are two aspects of the proposed rule that are under close scrutiny of the reviewing community. First, satellite sewer systems would be operated under separate NPDES permits. Satellite sewer systems are loosely defined in the proposed CMOM rule as any agency that conveys wastewater to another agency for additional conveyance and final treatment and discharge. For Grants Pass, the RSSSD would be considered a satellite system requiring its own NPDES permit.

Second, the proposed CMOM rule is vague on the design threshold to which SSOs must be controlled. For instance, the proposed rule is silent on the recurrence interval of a storm event above which SSOs would be allowed (e.g., would SSOs be permitted during storms greater than 1-in-5 year event?). As such, EPA offers wastewater agencies little or no clear guidance regarding the amount of additional pipeline and pump station construction that would be required under CMOM, nor understanding about the amount of additional maintenance effort required to ensure elimination of SSOs.

As of February 2008, the CMOM rule had not yet been implemented. Based on an April 2004 statement on the EPA SSO web page, "SSO Proposed Rule was with-drawn from publication in the Federal Register," the timeline for implementation of the CMOM rule is uncertain. How the CMOM rule will eventually be interpreted and applied in Oregon is also uncertain. One possibility is that Oregon's "bacteria rule," will be used to set a minimum threshold for SSO prevention.

10.30.8.2 STATE POLICY

10.30.8.2.1 National Pollutant Discharge Elimination System

Section 402 of the CWA provides the legal basis for the NPDES permit program, which regulates point and non-point source discharges. Oregon Department of Environmental Qualtiy (ODEQ) is authorized by EPA to administer the NPDES program through Oregon Revised Statute 468B and associated OARs. These rules and statutes include regulations for wastewater collection, treatment, control, and disposal. Under the conditions of the NPDES permit, permittees are allowed to construct, install, modify, or operate these systems only in conformance with the Federal Clean Water Act and the above-mentioned State statues that set forth requirements, limitations, and conditions for such activities.

The Grants Pass WRP plant operates under NPDES Permit Number 101985 issued December 29, 2000.

10.30.8.2.2 Bacterial Control Management Plan

As noted previously, EPA is currently considering proposed CMOM rules that will limit the number of allowable SSOs. While the proposed CMOM rule is silent on collection system design criteria, Oregon has already adopted a rule that addresses the "bacteria rule," therefore, indirectly offers some guidance to design engineers and collection system owners. The OAR seeks to protect receiving waters and drinking water sources by prohibiting discharge of untreated wastewater to waters of the state except during the following conditions:

- During the period November 1 though May 21, except during a storm event greater than the 1-in-5-year, 24-hour duration storm.
- During the period of May 22 through October 31, except during a storm event greater than the 1-in-10-year, 24-hour duration storm.

The State Environmental Quality Commission may approve a change to these rules on a case-by-case basis as described in OAR 340-41-120. Determining the causes and preventing against SSOs usually requires a municipality to thoroughly evaluate both the collection and treatment system to determine the extent of extraneous weather-related flow, system structural condition and reliability, system hydraulic and treatment capacity, and the efficiency of operation and maintenance practices. The City has already performed a thorough analysis of the treatment and collection system.

10.30.8.2.3 Groundwater Regulation

The Federal SDWA requires that state underground injection control programs be established to ensure that underground injection will not endanger drinking water sources. In Oregon, groundwater regulations, including regulatory requirements for injection controls, are administered by ODEQ. On-site drain fields and septic systems that serve 20 or more persons are considered injection wells by ODEQ. The City Development Code requires that all new development within the service area be connected to the wastewater collection and treatment system. Existing development using septic systems is required to connect to the public sewer system at such time as repair or replacement of existing facilities is necessary, if public sewer is within 300 feet of the property.

10.30.8.3 LOCAL ORDINANCES, POLICY, AND MANAGEMENT AGREEMENT

Local requirements of particular concern to the planning process are related to the City Municipal Code, City Development Code, and the Sanitary Sewer Lateral Replacement Policy.

10.30.8.3.1 City of Grants Pass Municipal Code

City Ordinances 4861 and 5028 have been adopted by the City as Chapter 8.50 of the City Municipal Code. Chapter 8.50 is intended to protect public health and safety; protect the environment; and ensure compliance with all applicable state and federal laws as they pertain to wastewater collection, conveyance, treatment, and discharge.

Sewer use requirements set forth in Chapter 8.50 include general and specific prohibited discharge standards. The general prohibitions state that no user shall introduce or cause to be introduced into the City WRP any pollutant or wastewater which causes pass-through or interference, or which will cause the WRP to violate its NPDES permit or harmfully impact the receiving water quality standards. Chapter 8.50 also sets forth procedures for the allowance of intentional occurrences, and the reporting of unanticipated bypass.

10.30.8.3.2 City of Grants Pass Development Code

Title 10 of the City Municipal Code may also be cited as the City Development Code. The purpose of the Development Code is to coordinate City regulations governing the development and use of land. Standards for sewer and septic systems are set forth in the Development Code to ensure compliance with state and federal statutes, policies, and laws designed to prevent harmful impact to

receiving waters.

10.30.8.3.3 Sanitary Sewer Lateral Replacement Policy.

Substandard or combined sewer laterals discovered during public sewer, water, or storm drain projects are required to be replaced. The City considers the sewer lateral to be the responsibility of the private property owner from the point of connection to the main to the building being served. Replacement of substandard sewer laterals may often include work within the public right-of-way, with the possibility of additional costs such as pavement patching, traffic control, and other construction items not usually associated with work within private property boundaries. To assist the property owner in the cost of lateral replacement, the City has adopted a Sanitary Sewer Lateral Replacement Policy. Under this policy, the City Manager can authorize payment of 50 percent of the cost of replacing failed or otherwise substandard laterals.

10.30.8.3.4 Urban Growth Boundary Management Agreement.

The City-County Urban Service Policies, adopted with the UGB in August 1979, require a public sewer system with capacities to serve urban levels of development. On August 8, 1998, Josephine County, City of Grants Pass, Harbeck-Fruitdale Sewer District and Redwood Sanitary Sewer Service District signed an Intergovernmental Agreement for the Orderly Management of the Grants Pass Urban Growth Boundary Area.

10.30.9 FINANCING PLAN

Various funding alternatives exist for the City of Grants Pass to implement the Capital Improvement Plan (CIP). The purpose of this section is to determine the best option for financing. Unless otherwise noted, the information from this section was excerpted from the *2001 Wastewater Facilities Plan Update*.

As part of the work in developing a financing plan, a computer model was created to assist the City in future modifications to both the CIP and operation/maintenance funding. This model is the basis for the conclusions presented herein.

10.30.9.1 Capital Costs

The CIP includes three major elements: Treatment Plant (liquid treatment), Biosolids, and Collection System. The proposed phasing plan is shown in Table 10.30.15.

Table 10.30.15 Capital Improvement Plan (Year 2000 dollars)

	YEAR 2001-2004	Year 2005-2006	Year 2010-2011
Treatment Plant	$5,940,000	$5,795,000	$1,855,000
Biosolids	$3,590,000		
Collection System	$2,500,000		
TOTAL	$12,030,000	$5,795,000	$1,855,000

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc.

In addition to capital costs, the City must be able to fund the ongoing cost of operating and adequately maintaining its sewer utility. A breakdown of these operation and maintenance costs is shown in Table 10.30.16.

Table 10.30.16 Operation and Maintenance Expenses (Year 2000 dollars)

Wastewater Collection Services	$333,488
Wastewater Treatment Services	$887,513
Customer Services	$167,910
General Program Operations	$361,393
TOTAL	$1,750,304

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc.
NOTE: Updated operation and maintenance expenses can be obtained from annual city budgets.

10.30.9.2 Current Funding

The City of Grants Pass has two main revenue sources: monthly user charges and system development charges. The monthly user charge is collected for all system customers, and has increased since the *2001 Wastewater Facilities Plan Update*.

All new connections to the system pay a Sewer System Development Charge (SSDC). This charge is designed so new customers pay their share of wastewater collection and treatment infrastructure costs. At the time of the *2001 Wastewater Facilities Plan Update*, SSDCs averaged about $1,000 per new connection. SSDCs have increased substantially since that time, and vary depending upon use and location (properties within the Redwood Sanitary Sewer District are subject to a different SDC schedule than other properties connecting to the system.)

10.30.9.3 Capital Funding Mechanisms

To fund the CIP, the City will need to consider other funding mechanisms. These other funding mechanisms include: Revenue Bonds; Low interest loans – State Revolving Fund (SRF); and, Grants.

A revenue bond is a very common tool to fund capital improvements. Rates are determined by market conditions and currently are around 5 to 6 percent. Due to the significant financing cost and associated coverage, this funding method is normally a last resort if other funds are no available.

Grants used to be the only way to finance wastewater treatment improvements. Many of the upgrades to secondary treatment across the United States were funded by grants, sometimes up to 75 percent of the costs. Wastewater construction grants, while still available, are insufficient compared with the current demand.

One of the best governmental programs available is the State Revolving Fund loan program. This fund, seeded by money from the Environmental Protection Agency (EPA), is giving state governments, including Oregon, funds to loan to municipalities for treatment and collection system improvements. These loans, administered by the Oregon Department of Environmental Quality Wastewater Finance Office, are "low interest" with current rates at less than 4 percent. Unlike bonds, normally these loans have a smaller reserve amount and no coverage requirements. Due to

federal funding, cities like Grants Pass seeking SRF funding must comply with EPA requirements for a Facility Plan, thus this plan has been developed according to said requirements. The City secured approximately $7M in SRF loans in 2003-2004 for work on Phase I of the CIP. The work was completed as of July 2004, and the loans were in the process of being repaid as of January 2008.11

10.30.9.4 Projected Cash Flow

The major components of both revenue and expenses in a simplified view of the wastewater utility are:

- Revenue: Monthly Sewer Service Charges, SDCs, Loan Proceeds, and Interest
- Expenses: Operation and Maintenance, Capital Expenditures, Debt Service, and Reserve Amounts

In reality, these revenues and expenses are tracked in separate accounts; however, for the purpose of this simplified analysis, all revenue and expenses will be considered as one amount. Table 10.30.17 projects the annual cash flow for the Grants Pass wastewater utility. This projection is for the period of the CIP. With the funding from the SRF, and the current cash on hand (based on the City's construction fund only), it appears that the City will be able to fully fund the CIP. In addition, the City may be able to fund an ongoing sewer rehabilitation program. This amount, shown in the cash flow during non-CIP funded years, is $250,000. It will have to be determined if this amount is sufficient to address the long-term needs of the City.

11 Per Joey Wright, City of Grants Pass Public Works Department, January 2008

Table 10.30.17 Simplified Cash Flow Analysis

	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011
Beginning Balance	$6,800,000	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	1,401,552$	$931,076
Rate Revenue	$2,650,000	$2,689,750	$2,730,096	$2,771,048	$2,812,613	$2,854,803	$2,897,625	$2,941,089	$2,985,205	$3,029,983	$3,075,433	$3,121,565
SDC Revenue		$100,000	$101,500	$103,023	$104,568	$106,136	$107,728	$109,344	$110,984	$112,649	$114,339	$116,054
O&M Cost	$1,750,000	$1,802,500	$1,856,575	$1,912,272	$1,969,640	$2,028,730	$2,089,592	$2,152,279	$2,216,848	$2,283,353	$2,351,854	$2,422,409
Loan Funding		$2,000,000	$2,000,000		$2,000,000							
Debt Service		$145,435	$290,871	$290,871	$290,871	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306
Reserve Account		$150,000	$150,000			$150,000						
Capital Cost		$6,195,400	$6,195,400	$100,000[a]	$100,000[a]	$2,547,000	$2,547,000	$100,000[a]	$100,000[a]	$100,000[a]	$921,000	$921,000
Investment Income		$290,000	$243,728	$111,106	$56,396	$82,298	$92,184	$50,301	$19,036	$34,521	$48,912	$46,653
Ending Balance	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	$1,401,552	$931,076	$435,633

Source: 2001 Grants Pass Wastewater Facilities Plan Update, Parametrix

[a] Balances reflect both Capital and Operations Accounts

10.30.9.5 Conclusion.

The financial situation of the City's wastewater utility is good. Given the ability to secure loans from the SRF program, and the sound fiscal management of the utility, the City will be able to fully fund the CIP as outlined in the Facilities Plan.

10.30.10 SANITARY SEWER SERVICES FINDINGS

10.30.10.1 Existing Sewer Capacity

- At the time of the *2001 Wastewater Facilities Plan Update*, the Grants Pass Water Restoration Plant (WRP) treated 4.5 mgd of average dry weather flow (ADWF) to a record peak storm flow of 26.5mgd. More recent data has also been compiled. As of April 2008, the WRP treated an ADWF of 5.5 mgd and peak wet weather flow of 30.0 mgd. The plant is comprised of numerous unit treatment processes for both liquid and solid streams, a control/laboratory building, and a maintenance shop. The Grants Pass WRP has a 27 mgd hydraulic capacity for influent pumping12, screening, and primary treatment. There is a 13 mgd hydraulic capacity for secondary treatment. In addition, there is a 43 mgd hydraulic capacity for UV disinfection. Flow exceeding the secondary treatment capacity receives only primary treatment and disinfection. This occurs only a few days a year during wet weather storm conditions. For the final phase of disposal, dewatered biosolids are loaded into a dumpster and hauled to the Merlin Landfill for the JO-GRO composting process.

- The Harbeck-Fruitdale Sewer Service District uses the Grants Pass WRP plant for the processing of its waste. The sewer collection system has a capacity that can accommodate the population equivalent of approximately 14,000 persons.

- The Redwood Sanitary Sewer Service District uses the Grants Pass WRP for the processing of its waste. The sewer collection system has a capacity that can accommodate the population equivalent of approximately 16,000 persons.

10.30.10.2 Future Need

- The City and County have already established pumping of Redwood's wastewater to Grants Pass WRP and are not likely to extend services to the North Valley during this planning period.

- The *2004 Collection System Master Plan* developed a future equivalent population projection for the service area by applying sub-area growth rates from the City's Comprehensive Plan to the base population estimates for the sub-areas and adding 35% to the resulting population for commercial / industrial population equivalent. As of 2003, the Grants Pass WRP was serving an estimated population equivalent of 44,250. Estimated growth rates are respectively: 1.5% for Grants Pass, 1.6% for Harbeck-Fruitdale and 3.1% for Redwood. By the year 2020, it was anticipated that the Grants Pass WRP could be serving a total population of 60,157. Year 2060

12 As of January 2008 a project was completed to increase the hydraulic capacity for influent pumping to 45 mgd.

service area population, assumed by the *2004 Collection System Master Plan* to represent build-out, was estimated at approximately 130,167.

- The *2001 Wastewater Facilities Plan Update* identified a need for several improvements: At the head works, an additional mechanical bar screen to be installed with a 23.5 mgd capacity this provides redundancy and a maximum capacity of 47 mgd, which is well beyond the projected future flows); An outfall diffuser to be added to improve and reduce the ammonia toxicity into the Rogue River; Several miscellaneous plant improvements, which existed in the Facility Plan, were also included in the value engineering alternatives. These improvements include laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion. All of these improvements have since been completed.

- Plant Equipment Audit. The *2001 Wastewater Facilities Plan* recommended that a full analysis of the existing component conditions be conducted. This audit would analyze the remaining life span of each component and develop an operations and maintenance schedule for the 20-year planning period. The audit has since been completed.

- Additional Plant Landscaping. As of the *2001 Wastewater Facilities Plan*, a substantial amount of landscaping at the treatment plant had occurred to promote a good-neighbor environment. However, to continue this effort, the *WFP* recognized that additional landscaping would be necessary. The installation of additional landscaping has been ongoing.

- The City of Grants Pass continues upgrading its sanitary sewer system facilities to provide the appropriate level of services for the impending growth and development of its service area. Through its master facility planning, the City is prepared to meet its growth challenges in compliance with federal, state and local regulations, provided it continues to implement the respective Capital Improvements Plans as scheduled.

- Feasibility for Effluent Reuse, (Chapter 7 – *Grants Pass Facilities Plan*, Parametrix, 1999) concluded that reclaimed water (treated effluent) from the Grants Pass Water Restoration Plant (WRP) could be used beneficially instead of being discharged. Low river levels coinciding with the period of peak crop water use make irrigation a potentially feasible alternative for the City of Grants Pass and local agricultural water users. Reclaimed water can irrigate agricultural land, parks, highway landscaping, and golf courses. It can be used to grow wood fiber for fuel, pulp, or lumber. Other potential uses include increased flow to wetlands and storage in reservoirs or other impoundments specifically designated for effluent reuse only.

    - An extra incentive to "get out of the river" during the summer months is provided by concerns over meeting temperature requirements, potential future nutrient limitations, and listing of certain fish species as endangered. Reclaimed WRP effluent can also be viewed as a valuable resource that could help bridge the gap created by the planned removal of the Savage Rapids Dam. Dam removal has sparked a controversy about the availability of irrigation water, and new costs associated with pumping irrigation water from the Rogue River.

- Effluent reuse is complex and requires further planning to address implementation issues of effluent reservoir and storage siting, piping route selection, right-of-way acquisition, system ownership, financing and management.

10. PUBLIC FACILITIES & SERVICES

Goal

To provide needed facilities and services for the Urban Growth Boundary area in a timely, orderly, efficient, economic and coordinated manner.

Policies

10.1 General Service Policies

10.1.1 Urban levels of development shall require urban levels of service, as defined by the Implementing Ordinances.

10.1.2 Those who benefit most from the extension of urban services shall be those who pay most of the cost of service extension. Citizens in the developed areas with a full range of services already provided should pay little if any of the costs of extending urban services. Various techniques should be utilized to mitigate the economic impact of service extension to those residents in developing areas who already provide certain of their own services, and to mitigate the economic impact of service extension to those persons on fixed and/or low incomes.

10.1.3 Services shall be provided in an orderly and economic manner. Services provided at public expense should be provided first to those areas most heavily committed to urban development and those areas most actively developing, before extension to less committed areas or to those areas less actively developing. The extension of services with similar physical and/or programmatic requirements should be coordinated where economies will result. The involvement of the private sector is essential in the provision of services, and will determine to a great extent the timing, location and financing means of service extensions.

10.1.4 The division of lands and development of property within the Urban Growth Boundary shall be in accordance with the phased provision of urban services, as provided in the Implementing Ordinances. The type, location and phasing of public facilities and services shall be used by the City and County in a coordinated fashion as factors to direct urban expansion, and to implement land use policies.

10.1.5 Neither the City nor the County shall create special districts within the Urban Growth Boundary for the provision of water, sewer, storm drainage or street improvement services, unless approved by both parties and managed by either the City Council or the Board of County Commissioners. Overlapping and competing layers of political control of the provision of services shall be discouraged.

10.1.6 Services shall be resource effective. Services shall not be extended past the carrying capacity of the resource base of that service, and shall utilize the resource in the most effective way practicable.

EXHIBIT B
to Ordinance

10.1.7 The City and County recognize that the provision of necessary services to accommodate the projected growth and land use allocations is a mutual responsibility. The City and County will continue to cooperate with other and with the private sector in the development and use of financial mechanisms and programs that are effective, efficient and equitable. The County recognizes its need to develop new techniques and resources for financing urban level public facilities.

10.1.8 The City and County will develop, adopt and maintain Capital Improvement Programs to meet the needs of the service area. These programs will be used as a guide in the decision making process regarding the expenditures of local public funds on capital projects as well as seeking State and Federal funds.

10.2 Water Service Policies

10.2.1 The City and County shall follow the adopted Water Facilities Plan for the Urban Growth Boundary area when extending and improving water service. Key factors to be utilized in growth management include:

(a) the number, size, location and approximate costs of water treatment, storage and distribution facilities deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) water sources and treatment and distribution modes;

(c) continued input from all segments of the community;

(d) implementation and financing strategies for acquiring, developing and maintaining needed water treatment, storage and distribution; and

(e) determination of the areas of greatest need, including techniques of funding and prioritization for these areas of need.

10.2.2 The City and County shall maintain a continuously updated computerized model of the municipal distribution system. This model shall be available for use at cost by public agencies and private organizations in order to determine questions of service capacity, improvement requirements and improvement cost.

10.2.3 The City and County shall adopt an official Water Facilities Plan Map, showing the location, size and type of existing and future water treatment, storage and distribution facilities called for by the Water Facilities Plan, and such map shall be keyed to the computerized model of the distribution system.

10.2.4 The Development Code shall facilitate these water service policies, and shall contain a balanced mix of positive incentives (which may include density transfers, density bonuses, rapid review procedures, etc.) as well as exactive requirements (which may include dedication or easement requirements, system charges, development requirements, etc.) as needed to assure the realization of these policies.

10.2.5 The City and County shall maintain a Capital Improvement Program (CIP) which shall include timely and adequate funding to realize the development of facilities required by the Water Facilities Plan, and shown on the Water Facilities Plan Map.

10.2.6 The Water Facilities Plan shall be reviewed and updated periodically as necessary, with major revisions at five year intervals.

10.2.7 Urban level development shall require a public water system, or shall meet requirements of interim development standards as provided by the Implementing Ordinances. Interim development standards shall allow development to proceed in a timely and economical manner, prior to full public water system extension, provided the requirements of public safety, health and welfare are met, and the future extension of the public water system is safeguarded.

10.3. Sewer Service Policies

10.3.1 The City and County shall follow adopted Sanitary Sewer Facility and Management Plans for the Redwood, Fruitdale-Harbeck and City service districts, including all parts of the Urban Growth Boundary area. The Sanitary Sewer Facility and Management Plans:

(a) determine the number, size, location and approximate costs of sanitary sewer facilities and improvements deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) base the facilities and improvements determination upon a thorough analysis of the Urban Growth Boundary service districts, including present treatment plan capacity, treatment levels and Department of Environmental Quality requirements, collection system age, construction and function, and infiltration and inflow characteristics of the system;

(c) recommend implementation and financing strategies for acquiring, developing and maintaining needed sanitary sewage facilities;

(d) demonstrate continuity with past sanitary sewer plans, as adopted and developed by the City and County;

(e) provide for adequate coordination between the City and County as needed in the expansion and maintenance of the sewer service districts;

(f) determine the areas of highest priority.

10.3.2 The City and County shall maintain an official Sanitary Sewer Facilities Plan Map, showing the location, size and type of existing and future collection and treatment facilities called for by the Sanitary Sewer Facilities and Management Plan. The map shall also show Service District boundaries.

10.3.3 The Development Code and Development Standards shall act to facilitate these sanitary sewer service policies, and shall contain a balanced mix of positive incentives (which may include density transfers, public funding of oversized lines, rapid review procedures, etc.) as well as exactive requirements (which may include dedication or easement requirements, system charges, development requirements, etc.) as needed to assure the realization of these policies.

10.3.4 The City and County shall maintain a Capital Improvement Program (CIP) which shall include timely and adequate funding to realize the development of facilities required by the adopted Sanitary Sewer Facility and Management Plans, and as shown on the Sewer Facilities Plan Map.

10.3.5 The Sanitary Sewer Facility and Management Plans shall be reviewed and updated periodically as necessary, with major revisions at five year intervals. The revisions to the Sanitary Sewer Facilities and Management Plans shall be used as a basis for revising these policies.

10.3.6 The City and County shall encourage sanitary sewer design that minimizes the cost of sanitary service extensions, and that minimizes the cost of maintaining such extensions.

10.3.7 Urban level development shall require a public sanitary sewer system, or shall meet the requirements of interim development standards as provided by the Implementing Ordinances. Interim development standards shall allow development to proceed in a timely and economical manner, prior to full extension of the sanitary sewer system, provided the requirements of public safety, health and welfare are met.

10.4 Storm Drain Service Policies

10.4.1 The City and County shall follow the adopted Master Storm Drain Facilities and Management Plan for the Urban Growth Boundary area when extending the improving drainage service. Key factors to be utilized in growth management include:

(a) the number, size, location and approximate costs of storm drainage facilities and improvements deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) the analysis of the UGB drainage basins, using generally accepted runoff projection techniques, including appropriate computer modeling, if possible;

(c) implementation and financing strategies for acquiring, developing and maintaining needed storm drainage facilities;

(d) maintaining continuity with past drainage plans, as adopted and developed by the City and County; and

(e) determination of the areas of highest priority, including techniques of funding and prioritization for these high priority areas.

10.4.2 The City and County shall adopt an official Storm Drainage Facility Map showing the location, size and type of existing and future storm drainage facilities called for by the Storm Drainage Plan. The Storm Drainage Map shall be used to determine service district jurisdiction, and the location of future storm drainage facilities and improvements.

10.4.3 The Development Code shall act to facilitate these storm drainage policies, and shall contain a balanced mix of positive incentives (which may include density transfers, public funding of oversized lines, rapid review procedures, etc.), as well as exactive requirements, system charges, development requirements, etc.), as needed to assure the realization of these policies.

10.4.4 The City and County shall develop a Capital Improvement Program (CIP) within 12 months of adoption of the Comprehensive Plan, which program shall include timely and adequate funding to realize the development of facilities required by the adopted Storm Drainage Plan, and shown on the Storm Drainage Facilities Map.

10.4.5 The Storm Drain Plan shall be reviewed and updated, and revised if necessary, at one year intervals, with major revisions at five year intervals. The revisions to the Storm Drain Plan shall be used as a basis for revising these policies.

10.4.6 The City and County working with the Grants Pass Irrigation District shall explore an agreement that will ensure that the storm drainage use of, and the necessary repairs, improvements and maintenance of the irrigation canal system, are made in a manner consistent with the Storm Drain Plan, and in a timely and cost-effective manner.

10.4.7 The City and County shall encourage storm drainage design that minimizes storm water runoff, including retention areas or devised, use of vegetative open space, and the preservation of natural waterways.

10.4.8 The City and County shall coordinate the provision of storm drain facilities with the provision of open space called for by the Park Facilities Plan, wherever possible, and to the extent practicable. This coordination shall include retaining drainage channels as close as possible to their natural state, and the use of plan materials and maintenance techniques in storm water retention.

10.4.9 Urban level development shall require urban levels of storm drainage, as provided in the Implementing Ordinances. Interim Development Standards shall allow development to proceed in a timely and economical manner, prior to full extension and development of the storm drain system, provided the requirements of public safety, health and welfare are met.

10.5 Solid Waste Service Policies

10.5.1 The City and County shall encourage the collection of solid waste within the Boundary area by private, commercial collection services.

10.5.2 The City and County Agreements with the commercial franchise service managing the solid waste landfill at the Merlin site shall include measures to successfully reduce leachate produced at the landfill site, such as uphill trenching and draining, and importation of suitable topsoil to reduce erosion and promote revegetation.

10.5.3 Within 16 months of adoption of the Comprehensive Plan, the City and County shall adopt a Solid Waste Management Implementation Plan, including relevant sections of the Solid Waste Management Plan (1975), which plan shall include:

(a) an ongoing assessment of landfill disposal techniques, with provisions for correction of those techniques as required.

(b) a yearly estimate of landfill capacity and the rates of solid waste generation, including all areas within the landfill site service district as well as the UGB area, and an estimate of when landfill site capacity will be reached.

(c) a recommendation of financing strategies for adequately maintaining and preparing the landfill site, as well as providing for alternative methods of solid waste disposal.

10.6 Police Protection Service Policies

10.6.1 Urban levels of development shall require urban levels of police protection. As the urbanizing area converts from rural to urban levels and intensities of land use over time, police protection should be increased to meet the increased service need.

10.6.2 The City and County shall explore an agreement establishing responsibility for the provision of police protection services within the Urban Growth Boundary over time. This agreement shall consider the costs and benefits of various methods of providing police protection, and shall include financing techniques to mitigate the costs of increased service.

10.7 Fire Protection Service Policies

10.7.1 Municipal water systems shall provide water at fire flow capacities.

10.7.2 Urban levels of development shall require urban levels of fire protection as stipulated by the Implementing Ordinances. The minimum urban level of fire protection for fully developed residential, commercial and industrial areas shall be that qualifying for the insurance underwriters relative classification rating of 5. Provision of fire protection should be phased over time as urban level development proceed without a minimum of a Class 8 rating, nor shall commercial industrial development proceed without a minimum of a Class 9 rating.

10.7.3 The City and County shall explore an agreement establishing responsibility for the provisions of fire protection services within the Urban Growth Boundary area over time. This agreement shall consider the costs and benefits of various methods of providing fire protection, and shall include financing techniques to mitigate the costs of increased service.

10.8 Health Services

10.8.1 Health services should be provided by the private sector. The City and County shall encourage the provision of health services in appropriate locations throughout the Boundary area.

10.9 School Service Policies

10.9.1 The City and County shall maintain an open, ongoing dialogue with the School Districts in a manner that will facilitate the planning efforts of all agencies.

10.9.2 The City and County shall notify the respective School Districts of all residential land use actions within that district in a timely and complete manner, and make development data available to the districts on a regular basis.

10.9.3 The School Districts shall be notified in a timely manner regarding revisions and updates to the Comprehensive Plan that may affect the Districts, and shall be encourage to participate in the revision process.

CITY OF GRANTS PASS COMMUNITY DEVELOPMENT DEPARTMENT

PUBLIC WATER AND SEWER SERVICE
COMPREHENSIVE PLAN TEXT AMENDMENT
FINDINGS OF FACT- CITY COUNCIL

Procedure Type:	Type IV: Planning Commission Recommendation and City Council Decision
Project Number:	08-40500002
Project Type:	Comprehensive Plan Text Amendment
Applicant:	City of Grants Pass
Planner Assigned:	Jared Voice / Tom Schauer
Application Received:	April 18, 2008
Application Complete:	April 18, 2008
Date of Planning Commission Staff Report:	June 4, 2008
Date of Planning Commission Hearing:	June 11, 2008
Planning Commission Findings of Fact:	June 25, 2008
Date of City Council Staff Report:	July 8, 2008
Date of City Council Hearing:	July 16, 2008
City Council Findings of Fact:	August 6, 2008

I. PROPOSAL:

The purpose of the proposal is to update the water and sewer sections of Comprehensive Plan Element 10 (*Public Facilities and Services*), including the policies for each, based on recent water and sewer master plans.

Additionally, the proposal would adopt the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:

Water

City of Grants Pass Water Distribution System Master Plan (West Yost and Associates, January 2001)

City of Grants Pass Water Management and Conservation Plan, Final Report (West Yost and Associates, June 2002)

City of Grants Pass Water Treatment Plant Facility Plan, Final Report (MWH/Montgomery Watson Harza, April 2004)

Sewer

1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999)

II. AUTHORITY AND CRITERIA:

Section 13.5.3 of the Grants Pass and Urbanizing Area Comprehensive Plan provides that the City Council may initiate a text amendment.

Sections 13.5.5 and 13.8 of the Comprehensive Plan provide that joint review by the City Council and Board of County Commissioners shall be required for amendment and revision to Comprehensive Plan findings, goals, and policies.

The review shall be in accordance with the procedures of Section 13.8.3 of the Comprehensive Plan, which provides for a recommendation hearing by the Urban Area Planning Commission prior to a joint hearing of the City Council and Board of County Commissioners.

However, with adoption of the 1998 Intergovernmental Agreement, this provision requiring a joint hearing is modified with the result that City Council will make the decision, and the County will have automatic party status, as summarized below:

> Section III of the 1998 Intergovernmental Agreement (IGA) provides for transfer of authority for provision and management of planning services from the County to the City for the Urbanizing Area. It provides:
>
> > The City is hereby vested with the exclusive authority to exercise the County's legislative and quasi-judicial powers, rights, and duties within the Urbanizing Area...
>
> Section V of the IGA contains provisions pertaining to notification and appeals for quasi-judicial and legislative decisions within the Urbanizing Area. For legislative decisions, the IGA provides:
>
> > The City agrees to provide written notice of all proposed legislative actions to the County at least 45 days prior to the public hearing at which the action is first considered. The County shall be deemed to have automatic party status regarding all such decisions for the purposes of standing for appeals.

Section 13.8.3 of the Comprehensive Plan provides that notice shall be as provided in Section 2.060 of the Development Code for a Type IV procedure. Section 13.8.3 further provides that the hearing shall be conducted in accordance with the Legislative Hearing Guidelines of Section 9 of the Development Code.

Therefore, the application was processed through a "Type IV" procedure, with a recommendation from the Urban Area Planning Commission and a final decision by City Council. The County has automatic party status for appeals.

The text of the Comprehensive Plan may be recommended for amendment and amended provided the criteria in Section 13.5.4 of the Comprehensive Plan are met.

III. APPEAL PROCEDURE:

The City Council's final decision may be appealed to the State Land Use Board of Appeals (LUBA) as provided in state statutes. A notice of intent to appeal must be filed with LUBA within 21 days of the Council's written decision.

IV. PROCEDURE:

A. The application for a Comprehensive Plan text amendment was submitted on April 18, 2008. The application was deemed complete on April 18, 2008, and processed in accordance with Section 2.060 of the Development Code.

B. Notice of the proposed amendment and the initial June 11, 2008 public hearing was mailed to the Oregon Department of Land Conservation and Development (DLCD) on April 18, 2008.

C. Notice of the proposed amendment was mailed to Josephine County on April 23, 2008 in accordance with the 1998 Intergovernmental Agreement for the Urbanizing Area.

D. Notice of the June 11, 2008 Planning Commission public hearing was mailed to potentially-interested parties and public agencies on May 21, 2008.

E. Public notice of the June 11, 2008 Planning Commission public hearing was published in the newspaper on June 4, 2008 in accordance with Sections 2.053 and 2.063 of the Development Code.

F. A public hearing was held by the Urban Area Planning Commission on June 11, 2008 to consider the request and make a recommendation to City Council.

G. Public notice of the July 16, 2008 City Council public hearing was mailed to potentially-interested parties and public agencies on June 26, 2008.

H. Public notice of the July 16, 2008 City Council public hearing was published in the newspaper on July 9, 2008 in accordance with Sections 2.053 and 2.063 of the Development Code.

I. A public hearing was held by the City Council on July 16, 2008 to consider the request.

V. SUMMARY OF EVIDENCE:

A. The basic facts and criteria regarding this application are contained in the City Council staff report and exhibits, which are attached as Exhibit "A" and incorporated herein.

B. A printed copy of the Power Point presentation given by staff at the July 16, 2008 City Council hearing is attached as Exhibit "B" and incorporated herein.

C. The minutes of the July 16, 2008 public hearing held by the City Council, which are attached as Exhibit "C", summarize the oral testimony presented and are hereby adopted and incorporated herein.

VI. GENERAL FINDINGS- BACKGROUND AND DISCUSSION:

The proposed new documents will replace the existing Water and Sewer sections of the Comprehensive Plan *Public Facilities* element, which were adopted in 1982 and determined service needs within the UGB through the year 2000.

Work on updating the Water and Sewer sections of the Comprehensive Plan Public Facilities element began in 2004, at the request of the City's former Utilities Department Director. The updated Comprehensive Plan element was intended to incorporate data from several recently-completed water and sewer master plans, and adopt the plans by reference.

In May of 2005, City Council adopted the Capital Improvement Programs (CIPs) recommended within the recently-completed water and sewer plans (see Resolution No's 4954 and 4955, corresponding City Council background sheets and Technical Memorandum from Parametrix dated April 25, 2005); however, the plans were not adopted by reference in their entirety, nor were updated Comprehensive Plan *Water Service* and *Sewer Service* sections adopted. The CIPs shown in the technical memo (and water and sewer plans) and adopted by Council are identical to those shown in the currently-proposed water and sewer section updates. Therefore, adoption of the current proposal would not change any of the planned projects for public water and sewer facilities over the twenty-year planning period. Note that several of the items identified within the CIPs have since been completed. Remaining items will continue to be identified for completion within annual budget reports as funds become available.

The proposal incorporates the previously-listed water and sewer facility plans into the Comprehensive Plan, to serve current and future needs within the existing Urban Growth Boundary. Additional facility planning is expected to occur as part of the anticipated Urban Growth Boundary expansion. The current update does not account for areas outside the existing UGB, except for those within special sewer districts (i.e. Redwood Sanitary Sewer Service District, Harbeck Fruitdale Sewer Service District) or which are already committed or planned to receive City water services (i.e. Paradise Ranch, North Valley Industrial Park, etc.)

Throughout the water and sewer service policies, it is stated that the City and County are jointly obligated for managing water and sewer services within the Urbanizing Area (UA). However, under the 1998 Intergovernmental Agreement (IGA), the County agreed to transfer to the City "all of the County's authority to provide and manage...facility financing and development within the UA." Therefore, as long as the IGA is in effect, it is the City's responsibility to adopt and maintain public facility and service plans for the UA, including water and sewer service plans.

VII. FINDINGS OF FACT- CONFORMANCE WITH APPLICABLE CRITERIA:

For amending the findings, goals, policies, and Land Use Map of the Comprehensive Plan, the City Council and Board of County Commissioners shall base their conclusions upon, and adopt findings in consideration of, all of the following criteria:

CRITERION (a): Consistency with other findings, goals and policies in the Comprehensive Plan.

> **City Council Response: Satisfied.** The proposal is consistent with other findings, goals and policies in the Comprehensive Plan.
>
> The existing *General Service*, *Water Service* and *Sewer Service* policies would be amended to eliminate language that is out of date, as the "Public Facilities and Services" element of the Comprehensive Plan *Policies Manual* was last amended in 1984. All amended policies are consistent with the goal of Element 10, which is: "To provide needed facilities and services for the Urban Growth Boundary area in a timely, orderly, efficient, economic and coordinated matter."
>
> Other goals and policies within the Comprehensive Plan that relate specifically to water and sewer facilities include the goal of *Element 4: Environmental Resource Quality* ("To maintain and improve the quality of the air, water and land resources of the area."), Policy 4.3 (d) ("The City and County shall affect water quality by increasing the hydraulic capacity of the City's wastewater treatment plant.") and the goal of *Element 8: Economy* ("To improve, expand, diversify and stabilize the economic base of the community.") The proposal is consistent with these goals and policies.

CRITERION (b): A change in circumstances, validated by and supported by the data base or proposed changes to the data base, which would necessitate a change in findings, goals and policies.

> **City Council Response: Satisfied.** The proposed amendment is necessary due to a change in circumstances that is supported by proposed changes to the database. The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated.
>
> The updated findings and policies are validated by and supported by the updated data base. As discussed in the response to the previous criterion, this policy is consistent with other findings, goals and policies found in the Comprehensive Plan.

CRITERION (c): Applicable planning goals and guidelines of the State of Oregon.

> **City Council Response: Satisfied.** The proposed amendment is consistent with applicable planning goals and guidelines of the State of Oregon. Applicable goals and guidelines include Goal 1 (Citizen Involvement), Goal 2 (Land Use Planning), Goal 6 (Air, Water and Land Resources Quality), Goal 11 (Public Facilities and Services), ORS 197.712(2)(e) (which requires a city to adopt a

public facility plan for areas within a UGB containing a population of over 2,500) and OAR 660, Division 11 (Public Facilities Planning.)

Goal 1- Citizen Involvement (OAR 660-015-0000(1)): See response to Criterion (d) below.

Goal 2- Land Use Planning (OAR 660-015-0000(2)): Although Goal 2 does not specifically address public facility planning, the explanation of Part I (Planning) of Goal 2 states the following: "All land-use plans and implementation ordinances shall be adopted by the governing body after public hearing and shall be reviewed and, as needed, revised on a periodic cycle to take into account changing public polices and circumstances..." The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated. Revisions to these sections and the associated policies are necessary to account for circumstances that have changed over the past 26 years. Further, subsection (C) of the "Guidelines" for Goal 2 states that: "Inventories and other forms of data are needed as the basis for the policies and other decisions set forth in the plan", including those for "man-made structures and utilities, their location and condition." Existing utilities inventories within the current Comprehensive Plan are not useful for any practical purpose and must be updated to be in full compliance with Goal 2.

Goal 6- Air, Water and Land Resources Quality (OAR 660-015-0000(6)): Goal 6 states that "all waste and process discharges from future development, when combined with such discharges from existing developments shall not threaten to violate, or violate applicable state or federal environmental quality statutes, rules and standards." To assure full compliance with Goal 6, an update of the City's sewer facility plans is necessary.

Goal 11- Public Facilities and Services (OAR 660-015-0000(11)) / ORS 197.712(2)(e) / OAR 660-011-0000: Goal 11 requires "a timely, orderly and efficient arrangement of public facilities and services to serve as a framework for urban and rural development." OAR 660, Division 11 interprets Goal 11 requirements and implements ORS 197.712(2)(e), which requires that a city develop and adopt a public facility plan for areas within a UGB containing a population greater than 2,500.

OAR 660-011-0005 defines public facilities plan as "a support document or documents to a comprehensive plan. The facility plan describes the water, sewer and transportation facilities which are to support the land uses designated in the appropriate acknowledged comprehensive plans within an urban growth boundary containing a population greater than 2,500..." The proposal contains three (3) separate supporting water documents and four (4) separate supporting sewer documents that would be adopted as supporting documents to the Comprehensive Plan. (Note that this proposal amends only the water and wastewater / sewer portion of the City's public facilities plan, and that other existing elements of the public facilities plan, including the Master Transportation Plan (adopted in 1997) and Master Storm Drain Plan (adopted in 1982) are not amended by this proposal.) The proposal also includes an update of the Comprehensive Plan itself, to reflect the supporting data contained in the separate water and sewer facility documents.

OAR 660-011-0010 through -0035 outlines the items that must be contained within a public facility plan. These include an inventory / assessment of existing facilities, a list of projected public facility projects (including rough cost estimates, locations, and time estimates for each), applicable urban growth management agreements and funding mechanisms. The proposals for both water and sewer contain each of these required items.

The proposed amendment is also consistent with remaining sections of Goal 11 and OAR 660 Division 11, including OARs 660-011-0045 ("Adoption and Amendment Procedures for Public Facility Plans"), 660-011-0060 ("Sewer Service to Rural Lands") and 660-011-0065 ("Water Service to Rural Lands").

ORS 197.610: Notice of the proposed amendment was mailed to the Oregon Department of Land Conservation and Development on April 18, 2008, in accordance with ORS 197.610.

CRITERION (d): Citizen review and comment.

City Council Response: Satisfied. Notice of the proposed amendment has been posted in accordance with the procedure required for a Type IV-B procedure. In addition, the agendas and packets for the June 11th, 2008 Planning Commission meeting and July 16th, 2008 City Council meeting were posted on the City's website in advance of the hearing. No written comments were submitted by citizens prior to or during the Planning Commission or City Council hearings.

CRITERION (e): Review and comment from affected governmental units and other agencies.

City Council Response: Satisfied. 45-day notice was provided to the Department of Land Conservation and Development (DLCD) in accordance with OAR 660 Division 18 and ORS 197.610. OAR 660-18-0035 provides that if DLCD is participating in the proceeding, they shall notify the local government 15 days prior to the first evidentiary hearing. DLCD has not provided notification to the City.

45-day notice was provided to Josephine County in accordance with the 1998 Intergovernmental Agreement for the Urbanizing Area. The County has replied and has no comments regarding the proposal.

Notice of the proposed amendment was also provided to affected agencies and governmental units, including the Oregon DHS Drinking Water Program, the Oregon Department of Environmental Quality (DEQ), Oregon Watermaster District 14, and Grants Pass Irrigation District. No additional comments regarding the proposal have been received.

CRITERION (f): A demonstration that any additional need for basic urban services (water, sewer, streets, storm drainage, parks, and fire and police protection) is adequately covered by adopted utility plans and service policies, or a proposal for the

requisite changes to said utility plans and service policies as a part of the requested Comprehensive Plan amendment.

> **City Council Response: Satisfied.** The proposal does not create the need for any additional basic urban services. The proposed utility plans contain facility improvements and upgrades necessary to serve the water and sewer needs for existing and future residents of the existing Urban Growth Boundary, as well as for a limited number of service areas outside the existing UGB, through at least the year 2020.

CRITERION (g): Additional information as required by the review body.

> **City Council Response: Satisfied.** No additional information was requested by the Planning Commission or City Council.

CRITERION (h): In lieu of item (b) above, demonstration that the Plan as originally adopted was in error.

> **City Council Response: Not Applicable.** Criterion (b) is applicable. The Plan was not adopted in error. The proposed amendment is being adopted in response to a change in circumstances. See Criterion (b) for discussion of the change in circumstances.

VIII. DECISION AND SUMMARY:

The City Council found the applicable criteria were satisfied and **APPROVED** the Comprehensive Plan text amendment. The vote was 8-0-0, with Councilors Berger, Cummings, Hyde, Kangas, Patterson, Renfro, Richardson and Wendle in favor, and none opposed.

IX. ADOPTED BY THE GRANTS PASS CITY COUNCIL this 6th day of August 2008.

Len Holzinger, Mayor

NOTE: *This is a legislative decision. State law does not require that a decision be made on the application within 120 days.*

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CITY OF GRANTS PASS COMMUNITY DEVELOPMENT DEPARTMENT

PUBLIC WATER AND SEWER SERVICE
COMPREHENSIVE PLAN TEXT AMENDMENT
CITY COUNCIL STAFF REPORT- TYPE IV

Procedure Type:	Type IV: Planning Commission Recommendation and City Council Decision
Project Number:	08-40500002
Project Type:	Comprehensive Plan Text Amendment
Applicant:	City of Grants Pass
Planner Assigned:	Jared Voice / Tom Schauer
Application Received:	April 18, 2008
Application Complete:	April 18, 2008
Date of Planning Commission Staff Report:	June 4, 2008
Date of Planning Commission Hearing:	June 11, 2008
Planning Commission Findings of Fact:	June 25, 2008
Date of City Council Staff Report:	July 8, 2008
Date of City Council Hearing:	July 16, 2008

I. PROPOSAL:

The purpose of the proposal is to update the water and sewer sections of Comprehensive Plan Element 10 (*Public Facilities and Services*), including the policies for each, based on recent water and sewer master plans.

See Exhibits A and B to adopting ordinance for text of proposed amendments.

See Exhibit 3 to the Planning Commission Staff Report for legal strike-through copy of proposed amendments to Policies Manual.

See Section VIII of the Planning Commission Findings of Fact (attached as Exhibit 1 to this document) for changes recommended by Planning Commission, which have been incorporated into Exhibit B to the adopting ordinance.

Additionally, the proposal would adopt the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:

Water

City of Grants Pass Water Distribution System Master Plan (West Yost and Associates, January 2001)

City of Grants Pass Water Management and Conservation Plan, Final Report (West Yost and Associates, June 2002)

City of Grants Pass Water Treatment Plant Facility Plan, Final Report (MWH/Montgomery Watson Harza, April 2004)

EXHIBIT A
to CC FOF

Sewer

1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999)

NOTE: To conserve resources, the above-listed water and sewer planning documents (Exhibits 4 - 9 to the Planning Commission Staff Report) have not been included in the Council packet. Please contact Jared Voice at the Community Development Department (474-6355, ex. 6317) if you would like copies of these exhibits made for you. Binders containing these exhibits are available for viewing in the City Administration and Community Development offices, located on the second floor of the Municipal Building, and will be available at the public hearing.

Each plan may be viewed in its entirety at the Public Works office, located on the first floor of the Municipal Building. Additionally, each plan is available for viewing on the City's website at www.grantspassoregon.gov. Follow the links "Your Government" > "Public Works" to "Water" or "Wastewater". Please contact Jared Voice (474-6355, ex. 6317) if you would like assistance in locating the plans on the web.

II. AUTHORITY AND CRITERIA:

The authority and criteria are provided in the Planning Commission's Findings of Fact.

III. APPEAL PROCEDURE:

The City Council's final decision may be appealed to the State Land Use Board of Appeals (LUBA) as provided in state statutes. A notice of intent to appeal must be filed with LUBA within 21 days of the Council's written decision.

IV. BACKGROUND AND DISCUSSION:

Detailed background and discussion is provided in the Planning Commission's Findings of Fact.

V. CONFORMANCE WITH APPLICABLE CRITERIA:

Detailed findings of conformance with applicable criteria are provided in the Planning Commission's Findings of Fact.

VI. RECOMMENDATION:

The Urban Area Planning Commission finds the applicable criteria are satisfied and **RECOMMENDS APPROVAL** of the proposal to City Council, which:

1) amends the sewer and water sections of Comprehensive Plan Element 10 by adopting new Comprehensive Plan Sections 10.20 (Water Services) and 10.30 (Sewer Services), as presented in Exhibit A to the adopting ordinance, and repealing existing Comprehensive Plan Sections 10.20 and 10.30, and
2) adopts revisions to the Element 10 (Public Facilities and Services) General Service Policies, Water Service Policies and Sewer Service Policies, as presented in Exhibit B to the adopting ordinance, and

3) adopts the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:
   i. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)
   ii. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)
   iii. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MHW/Montgomery Watson Harza, April 2004)
   iv. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
   v. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
   vi. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
   vii. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999); and

4) repeals previously-adopted water and wastewater plans which are rendered obsolete by adoption of the new plans.

VII. CITY COUNCIL ACTION:

A. Positive Action:
   1. approve the proposal recommended by the Planning Commission.
   2. approve the proposal recommended by the Planning Commission with modifications (list):

B. Negative Action: Deny the request and make no amendment for the following reasons (list):

C. Postponement: Continue item
   1. indefinitely.
   2. to a time certain.

NOTE: *This is a legislative decision. State law does not require that a decision be made on the application within 120 days.*

VIII. INDEX TO EXHIBITS:

1. Planning Commission's Findings of Fact and the Attached Record

    A. Planning Commission Staff Report
        1. Proposed Water Services Section 10.20
        2. Proposed Sewer Services Section 10.30
        3. Proposed Amended Public Facilities and Services Policies
        4. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)*
        5. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)*
        6. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MHW / Montgomery Watson Harza, April 2004)*
        7. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)*
        8. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)*
        9. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999)*
        10. City Council Resolution No. 4954, Adopting Water CIP, 5/9/05
        11. City Council Resolution No. 4955, Adopting Wastewater CIP, 5/9/05
        12. City Council Background Sheet for Water CIP, 5/4/05
        13. City Council Background Sheet for Wastewater CIP, 5/4/05
        14. Parametrix Technical Memo Dated 4/25/2005
        15. E-mail Message from Josephine County Planning Director, 6/3/08

    B. Planning Commission Power Point Presentation

    C. Minutes of 6/11/08 Planning Commission Hearing

***To conserve resources, Exhibits 4 through 9 to the Planning Commission Staff Report (the individual water and sewer planning documents) have not been included in the packet.** Please contact Jared Voice at the Community Development Department (474-6355, ex. 6317) if you would like copies of these exhibits made for you. Binders containing these exhibits are available for viewing in the City Administration and Community Development offices, located on the second floor of the Municipal Building, and will be available at the public hearing.

Each plan may be viewed in its entirety at the Public Works office, located on the first floor of the Municipal Building. Additionally, each plan is available for viewing on the City's website at www.grantspassoregon.gov. Follow the links "Your Government" > "Public Works" to "Water" or "Wastewater". Please contact Jared Voice (474-6355, ex. 6317) if you would like assistance in locating the plans on the web.

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CITY OF GRANTS PASS COMMUNITY DEVELOPMENT DEPARTMENT

PUBLIC WATER AND SEWER SERVICE
COMPREHENSIVE PLAN TEXT AMENDMENT
FINDINGS OF FACT- URBAN AREA PLANNING COMMISSION

Procedure Type:	Type IV: Planning Commission Recommendation and City Council Decision
Project Number:	08-40500002
Project Type:	Comprehensive Plan Text Amendment
Applicant:	City of Grants Pass
Planner Assigned:	Jared Voice / Tom Schauer
Application Received:	April 18, 2008
Application Complete:	April 18, 2008
Date of Staff Report:	June 4, 2008
Date of Planning Commission Hearing:	June 11, 2008
Planning Commission Findings of Fact:	June 25, 2008

I. PROPOSAL:

The purpose of the proposal is to update the water and sewer sections of Comprehensive Plan Element 10 (*Public Facilities and Services*), including the policies for each, based on recent water and sewer master plans.

Additionally, the proposal would adopt the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:

Water

1. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)
2. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)
3. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MWH/Montgomery Watson Harza, April 2004)

Sewer

1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999)

EXHIBIT 1
to CC. Staff Report

II. AUTHORITY AND CRITERIA:

Section 13.5.3 of the Grants Pass and Urbanizing Area Comprehensive Plan provides that the City Council may initiate a text amendment.

Sections 13.5.5 and 13.8 of the Comprehensive Plan provide that joint review by the City Council and Board of County Commissioners shall be required for amendment and revision to Comprehensive Plan findings, goals, and policies.

The review shall be in accordance with the procedures of Section 13.8.3 of the Comprehensive Plan, which provides for a recommendation hearing by the Urban Area Planning Commission prior to a joint hearing of the City Council and Board of County Commissioners.

However, with adoption of the 1998 Intergovernmental Agreement, this provision requiring a joint hearing is modified with the result that City Council will make the decision, and the County will have automatic party status, as summarized below:

> Section III of the 1998 Intergovernmental Agreement (IGA) provides for transfer of authority for provision and management of planning services from the County to the City for the Urbanizing Area. It provides:
>
> > The City is hereby vested with the exclusive authority to exercise the County's legislative and quasi-judicial powers, rights, and duties within the Urbanizing Area...
>
> Section V of the IGA contains provisions pertaining to notification and appeals for quasi-judicial and legislative decisions within the Urbanizing Area. For legislative decisions, the IGA provides:
>
> > The City agrees to provide written notice of all proposed legislative actions to the County at least 45 days prior to the public hearing at which the action is first considered. The County shall be deemed to have automatic party status regarding all such decisions for the purposes of standing for appeals.

Section 13.8.3 of the Comprehensive Plan provides that notice shall be as provided in Section 2.060 of the Development Code for a Type IV procedure. Section 13.8.3 further provides that the hearing shall be conducted in accordance with the Legislative Hearing Guidelines of Section 9 of the Development Code.

Therefore, the application will be processed through a "Type IV" procedure, with a recommendation from the Urban Area Planning Commission and a final decision by City Council. The County has automatic party status for appeals.

The text of the Comprehensive Plan may be recommended for amendment and amended provided the criteria in Section 13.5.4 of the Comprehensive Plan are met.

III. APPEAL PROCEDURE:

The City Council's final decision may be appealed to the State Land Use Board of Appeals (LUBA) as provided in state statutes. A notice of intent to appeal must be filed with LUBA within 21 days of the Council's written decision.

IV. PROCEDURE:

A. The application for a Comprehensive Plan text amendment was submitted on April 18, 2008. The application was deemed complete on April 18, 2008, and processed in accordance with Section 2.060 of the Development Code.

B. Notice of the proposed amendment and the initial June 11, 2008 public hearing was mailed to the Oregon Department of Land Conservation and Development (DLCD) on April 18, 2008.

C. Notice of the proposed amendment was mailed to Josephine County on April 23, 2008 in accordance with the 1998 Intergovernmental Agreement for the Urbanizing Area.

D. Notice of the June 11, 2008 Planning Commission public hearing was mailed to potentially-interested parties and public agencies on May 21, 2008.

E. Public notice of the June 11, 2008 Planning Commission public hearing was published in the newspaper on June 4, 2008 in accordance with Sections 2.053 and 2.063 of the Development Code.

F. A public hearing was held by the Urban Area Planning Commission on June 11, 2008 to consider the request and make a recommendation to City Council.

V. SUMMARY OF EVIDENCE:

A. The basic facts and criteria regarding this application are contained in the Planning Commission staff report and exhibits, which are attached as Exhibit "A" and incorporated herein.

B. A printed copy of the Power Point presentation given by staff at the June 11, 2008 Planning Commission hearing is attached as Exhibit "B" and incorporated herein.

C. The minutes of the June 11, 2008 public hearing held by the Urban Area Planning Commission, which are attached as Exhibit "C", summarize the oral testimony presented and are hereby adopted and incorporated herein.

VI. GENERAL FINDINGS- BACKGROUND AND DISCUSSION:

The proposed new documents will replace the existing Water and Sewer sections of the Comprehensive Plan *Public Facilities* element, which were adopted in 1982 and determined service needs within the UGB through the year 2000.

Work on updating the Water and Sewer sections of the Comprehensive Plan Public Facilities element began in 2004, at the request of the City's former Utilities Department Director. The updated Comprehensive Plan element was intended to incorporate data from several recently-completed water and sewer master plans, and adopt the plans by reference.

In May of 2005, City Council adopted the Capital Improvement Programs (CIPs) recommended within the recently-completed water and sewer plans (see Resolution No's 4954 and 4955, corresponding City Council background sheets and Technical Memorandum from Parametrix dated April 25, 2005); however, the plans were not adopted by reference in their entirety, nor were updated Comprehensive Plan *Water Service* and *Sewer Service* sections adopted. The CIPs shown in the technical memo (and water and sewer plans) and adopted by Council are identical to those shown in the currently-proposed water and sewer section updates. Therefore, adoption of the current proposal would not change any of the planned projects for public water and sewer facilities over the twenty-year planning period. Note that several of the items identified within the CIPs have since been completed. Remaining items will continue to be identified for completion within annual budget reports as funds become available.

The proposal incorporates the previously-listed water and sewer facility plans into the Comprehensive Plan, to serve current and future needs within the existing Urban Growth Boundary. Additional facility planning is expected to occur as part of the anticipated Urban Growth Boundary expansion. The current update does not account for areas outside the existing UGB, except for those within special sewer districts (i.e. Redwood Sanitary Sewer Service District, Harbeck Fruitdale Sewer Service District) or which are already committed or planned to receive City water services (i.e. Paradise Ranch, North Valley Industrial Park, etc.)

Throughout the water and sewer service policies, it is stated that the City and County are jointly obligated for managing water and sewer services within the Urbanizing Area (UA). However, under the 1998 Intergovernmental Agreement (IGA), the County agreed to transfer to the City "all of the County's authority to provide and manage…facility financing and development within the UA." Therefore, as long as the IGA is in effect, it is the City's responsibility to adopt and maintain public facility and service plans for the UA, including water and sewer service plans.

VII. FINDINGS OF FACT- CONFORMANCE WITH APPLICABLE CRITERIA:

For amending the findings, goals, policies, and Land Use Map of the Comprehensive Plan, the City Council and Board of County Commissioners shall base their conclusions upon, and adopt findings in consideration of, all of the following criteria:

CRITERION (a): Consistency with other findings, goals and policies in the Comprehensive Plan.

Planning Commission Response: Satisfied. The proposal is consistent with other findings, goals and policies in the Comprehensive Plan.

The existing *General Service*, *Water Service* and *Sewer Service* policies would be amended to eliminate language that is out of date, as the "Public Facilities and Services" element of the Comprehensive Plan *Policies Manual* was last amended in 1984. All amended policies are consistent with the goal of Element 10, which is: "To provide needed facilities and services for the Urban Growth Boundary area in a timely, orderly, efficient, economic and coordinated matter."

Other goals and policies within the Comprehensive Plan that relate specifically to water and sewer facilities include the goal of *Element 4: Environmental Resource Quality* ("To maintain and improve the quality of the air, water and land resources of the area."), Policy 4.3 (d) ("The City and County shall affect water quality by increasing the hydraulic capacity of the City's wastewater treatment plant.") and the goal of *Element 8: Economy* ("To improve, expand, diversify and stabilize the economic base of the community.") The proposal is consistent with these goals and policies.

CRITERION (b): A change in circumstances, validated by and supported by the data base or proposed changes to the data base, which would necessitate a change in findings, goals and policies.

Planning Commission Response: Satisfied. The proposed amendment is necessary due to a change in circumstances that is supported by proposed changes to the database. The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated.

The updated findings and policies are validated by and supported by the updated data base. As discussed in the response to the previous criterion, this policy is consistent with other findings, goals and policies found in the Comprehensive Plan.

CRITERION (c): Applicable planning goals and guidelines of the State of Oregon.

Planning Commission Response: Satisfied. The proposed amendment is consistent with applicable planning goals and guidelines of the State of Oregon. Applicable goals and guidelines include Goal 1 (Citizen Involvement), Goal 2 (Land Use Planning), Goal 6 (Air, Water and Land Resources Quality), Goal 11 (Public Facilities and Services), ORS 197.712(2)(e) (which requires a city to adopt a public facility plan for areas within a UGB containing a population of over 2,500) and OAR 660, Division 11 (Public Facilities Planning.)

Goal 1- Citizen Involvement (OAR 660-015-0000(1)): See response to Criterion (d) below.

Goal 2- Land Use Planning (OAR 660-015-0000(2)): Although Goal 2 does not specifically address public facility planning, the explanation of Part I (Planning) of Goal 2 states the following: "All land-use plans and implementation

ordinances shall be adopted by the governing body after public hearing and shall be reviewed and, as needed, revised on a periodic cycle to take into account changing public polices and circumstances..." The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated. Revisions to these sections and the associated policies are necessary to account for circumstances that have changed over the past 26 years. Further, subsection (C) of the "Guidelines" for Goal 2 states that: "Inventories and other forms of data are needed as the basis for the policies and other decisions set forth in the plan", including those for "man-made structures and utilities, their location and condition." Existing utilities inventories within the current Comprehensive Plan are not useful for any practical purpose and must be updated to be in full compliance with Goal 2.

Goal 6- Air, Water and Land Resources Quality (OAR 660-015-0000(6)): Goal 6 states that "all waste and process discharges from future development, when combined with such discharges from existing developments shall not threaten to violate, or violate applicable state or federal environmental quality statutes, rules and standards." To assure full compliance with Goal 6, an update of the City's sewer facility plans is necessary.

Goal 11- Public Facilities and Services (OAR 660-015-0000(11)) / ORS 197.712(2)(e) / OAR 660-011-0000: Goal 11 requires "a timely, orderly and efficient arrangement of public facilities and services to serve as a framework for urban and rural development." OAR 660, Division 11 interprets Goal 11 requirements and implements ORS 197.712(2)(e), which requires that a city develop and adopt a public facility plan for areas within a UGB containing a population greater than 2,500.

OAR 660-011-0005 defines public facilities plan as "a support document or documents to a comprehensive plan. The facility plan describes the water, sewer and transportation facilities which are to support the land uses designated in the appropriate acknowledged comprehensive plans within an urban growth boundary containing a population greater than 2,500..." The proposal contains three (3) separate supporting water documents and four (4) separate supporting sewer documents that would be adopted as supporting documents to the Comprehensive Plan. (Note that this proposal amends only the water and wastewater / sewer portion of the City's public facilities plan, and that other existing elements of the public facilities plan, including the Master Transportation Plan (adopted in 1997) and Master Storm Drain Plan (adopted in 1982) are not amended by this proposal.) The proposal also includes an update of the Comprehensive Plan itself, to reflect the supporting data contained in the separate water and sewer facility documents.

OAR 660-011-0010 through -0035 outlines the items that must be contained within a public facility plan. These include an inventory / assessment of existing facilities, a list of projected public facility projects (including rough cost estimates, locations, and time estimates for each), applicable urban growth management agreements and funding mechanisms. The proposals for both water and sewer contain each of these required items.

The proposed amendment is also consistent with remaining sections of Goal 11 and OAR 660 Division 11, including OARs 660-011-0045 ("Adoption and Amendment Procedures for Public Facility Plans"), 660-011-0060 ("Sewer Service to Rural Lands") and 660-011-0065 ("Water Service to Rural Lands").

ORS 197.610: Notice of the proposed amendment was mailed to the Oregon Department of Land Conservation and Development on April 18, 2008, in accordance with ORS 197.610.

CRITERION (d): Citizen review and comment.

Planning Commission Response: Satisfied. Notice of the proposed amendment has been posted in accordance with the procedure required for a Type IV-B procedure. In addition, the agenda and packet for the June 11th, 2008 Planning Commission meeting was posted on the City's website in advance of the hearing. No written comments were submitted by citizens prior to or during the Planning Commission hearing.

CRITERION (e): Review and comment from affected governmental units and other agencies.

Planning Commission Response: Satisfied. 45-day notice was provided to the Department of Land Conservation and Development (DLCD) in accordance with OAR 660 Division 18 and ORS 197.610. OAR 660-18-0035 provides that if DLCD is participating in the proceeding, they shall notify the local government 15 days prior to the first evidentiary hearing. DLCD has not provided notification to the City.

45-day notice was provided to Josephine County in accordance with the 1998 Intergovernmental Agreement for the Urbanizing Area. The County has replied and has no comments regarding the proposal.

Notice of the proposed amendment was also provided to affected agencies and governmental units, including the Oregon DHS Drinking Water Program, the Oregon Department of Environmental Quality (DEQ), Oregon Watermaster District 14, and Grants Pass Irrigation District. No additional comments regarding the proposal have been received.

CRITERION (f): A demonstration that any additional need for basic urban services (water, sewer, streets, storm drainage, parks, and fire and police protection) is adequately covered by adopted utility plans and service policies, or a proposal for the requisite changes to said utility plans and service policies as a part of the requested Comprehensive Plan amendment.

Planning Commission Response: Satisfied. The proposal does not create the need for any additional basic urban services. The proposed utility plans contain facility improvements and upgrades necessary to serve the water and sewer needs for existing and future residents of the existing Urban Growth Boundary, as well as for a limited number of service areas outside the existing UGB, through at least the year 2020.

CRITERION (g): Additional information as required by the review body.

Planning Commission Response: Satisfied. No additional information was requested by the Planning Commission.

CRITERION (h): In lieu of item (b) above, demonstration that the Plan as originally adopted was in error.

Planning Commission Response: Not Applicable. Criterion (b) is applicable. The Plan was not adopted in error. The proposed amendment is being adopted in response to a change in circumstances. See Criterion (b) for discussion of the change in circumstances.

VIII. RECOMMENDATION TO CITY COUNCIL:

The Urban Area Planning Commission recommends that City Council **APPROVE** the proposal, which:

1) amends the sewer and water sections of Comprehensive Plan Element 10 by adopting new Comprehensive Plan Sections 10.20 (Water Services) and 10.30 (Sewer Services), attached as *Exhibits 1 and 2* to the Planning Commission staff report, and repealing existing Comprehensive Plan Sections 10.20 and 10.30, which were adopted by Ordinance 4471 in 1982, and

2) adopts revisions to the Element 10 (*Public Facilities and Services*) sewer and water policies, attached as *Exhibit 3* to the Planning Commission staff report, with the following revisions:

    i. Un-strike and retain existing Policy 10.2.8 (to become Policy 10.2.7) and un-capitalize the words Interim Development Standards, so the policy reads as follows: *Urban level development shall require a public water system, or shall meet requirements of interim development standards as provided by the Implementing Ordinances. Interim development standards shall allow development to proceed in a timely and economical manner, prior to full public water system extension, provided the requirements of public safety, health and welfare are met, and the future extension of the public water system is safeguarded.*

    ii. Un-strike and retain existing Policy 10.3.7 and un-capitalize the words Interim Development Standards, so the policy reads as follows: *Urban level development shall require a public sanitary sewer system, or shall meet the requirements of interim development standards as provided by the Implementing Ordinances. Interim development standards shall allow development to proceed in a timely and economical manner, prior to full extension of the sanitary sewer system, provided the requirements of public safety, health and welfare are met.*; and

3) adopts the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:
    i. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)
    ii. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)
    iii. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MWH/Montgomery Watson Harza, April 2004)
    iv. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
    v. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
    vi. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
    vii. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999), and

4) repeals previously-adopted water and wastewater plans which are rendered obsolete by adoption of the new plans.

The vote was 6-0-0, with Commissioners Arthur, Sackett, Wickham, Kellenbeck, Fitzgerald and Fowler in favor, and none opposed. Commissioners Berlant and Fedosky were absent.

IX. ADOPTED BY THE URBAN AREA PLANNING COMMISSION this 25th day of June, 2008.

Gary Berlant, Chairperson
Urban Area Planning Commission

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6.4.1.5 Additional Clearwell Volume

Additional clearwell volume is required to provide adequate on-site storage for chlorine disinfection, flow equalization and backwash pumping capacity. 800,000 gallons of additional storage is recommended for the 30 mgd expansion to bring the total clearwell volume to 1,250,000 gallons to provide one hour of storage at the peak flow rate. The existing 450,000 gallon clearwell volume is barely adequate for the current 20 mgd plant flow, but additional volume does not have to be added until the plant capacity is expanded, or if future regulations disallow CT credit prior to filtration.

The new buried clearwell addition can be located in the front of the plant or to the west of the existing Basin 3. It should be inter-connected with the existing clearwell to maximize chlorine contact time for disinfection and to allow the existing HSPS to be expanded to pump the full 30 mgd peak capacity. Approximately 150,000 gallons of the required additional volume can be provided underneath the three new filters and filter gallery. Care must be taken to ensure that the deep excavation does not undermine the adjacent contact basin foundation. Further analysis of locations for additional clearwell volume are required during preliminary design.

If addition of an alternative form of disinfection is required in the future, either for *Cryptosporidium* inactivation or to control DBP formation or both, then the clearwell volume for the expanded plant could possibly be reduced. No costs for alternative disinfection systems are included in the expansion costs.

6.4.1.6 New High Service Pumps

The existing five high service pumps provide a total pumping capacity of 21 mgd and a firm, reliable capacity of 16.7 mgd with one large pump out of service. There is currently space to add two more pumps for capacity expansion. Assuming neither of these spaces is used for a spare backwash pump, then all of the plant's total pumping capacity can probably remain located in the existing High Service Pump Room for 30 mgd.

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There are a number of options for increasing the pumping capacity. If two additional 5 mgd pumps are added, then the total installed pumping capacity will be 31 mgd. The firm capacity with this arrangement would be 26 mgd +/-. To develop a firm 30 mgd pumping capacity, some of the smaller pumps would have to be replaced with larger pumps. Upsizing existing pumps requires careful evaluation of the electrical equipment and motor control center. The plant currently has two of the high service pumps equipped with VFDs and this is adequate for the ultimate pumping capacity. Ideally, the two pumps with VFDs will not be replaced with larger pumps for the expansion.

For planning purposes, it is assumed that two new 5 mgd pumps will be added to the High Service Pump Room. As discussed previously, at least one of the new pumps should be added prior to the full plant expansion, when demands exceed 15 mgd, to provide a firm/reliable pumping capacity of 20 mgd. Based on current growth projections, the new raw water pump will be required in the next 10 to 15 years. This new pump should be added at the same time as the new raw water pump addition.

6.4.1.7 Surge Control Improvements

The existing high service pumps discharge into a 36-inch finished water pipe that exits the plant property. At 30 mgd, the velocity in the 36-inch pipe header is approximately 6.5 feet per second (fps). At this velocity and under the high discharge pressure conditions, potential surge damage to the piping system which could be caused due to a sudden loss of pumping power. Surge could also damage plumbing system of nearby customers if the pressure wave is strong enough.

The plant discharge piping system is already equipped with a 11,000 gallon buried hydropneumatic surge control tank which has served the plant well for the past 20 years. At the higher flows for the 30 mgd expansion, the surge tank will likely be required to be replaced with a larger tank to provide adequate protection. For planning purposes, it is assumed that a new 15,000 gallon tank would be installed to replace the existing tank.

6.4.1.8 Yard Piping Improvements

Various yard piping buried around the plant site may need to be modified or replaced or relocated as part of the plant expansion project to accommodate new construction including the filters and clearwell and basin modifications. Also, additional raw water pipe will be required to deliver 15 mgd total to Basin #3. An allowance of $200,000 is provided for this work for planning purposes.

6.4.1.9 Increase Site Electrical Service

The existing plant transformer is rated at 1,500 kVa and is considered at capacity. The service, including the transformer and feeder cable(s) will have to be expanded to supply power for the new pumps and other mechanical equipment.

Normally, the power provider will replace the electrical equipment and cabling to serve a higher load without a capital charge to the City. The City's power rate structure might be adjusted to account for the larger service equipment. For planning purposes, an allowance of $200,000 is provided for this work.

6.4.1.10 Site Electrical Improvements to Support Expansion

In addition to the expanded power supply improvements, various site electrical improvements are require to support the new electrical, mechanical and control systems to be added for the plant expansion. These improvements include new motor control centers, feeders and cables, cable trays, ductbanks and terminations.

6.4.1.11 Instrumentation and Control Improvements for Expansion

The City will need to integrate certain features of the plant expansion components into its existing WTP SCADA/control system. Specifically, these improvements would integrate the new raw water pumps, new basin equipment, filters, and high service pumps. Various programming, software and hardware work items will be required. It is also assumed that the existing plant SCADA system will be upgraded/replaced due to technology advancements.

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6.4.1.12 Items Not Included

Items not included in the lists of recommended improvements and costs presented in ***Table 6-8*** are shown below. The City should review this list as it gets closer to expanding the plant capacity to verify no additional improvements are required.

- Alternative disinfection systems for *Cryptosporidium* inactivation and/or DBP formation control (such as UV, ozone, chlorine dioxide, or ammonia for chloramines)
- Costs for property acquisition for a new WTP site, or for solids disposal site
- Chemical feed system modifications for lime alternatives
- Structural modifications to existing basins, filters and control buildings for seismic protection, if required

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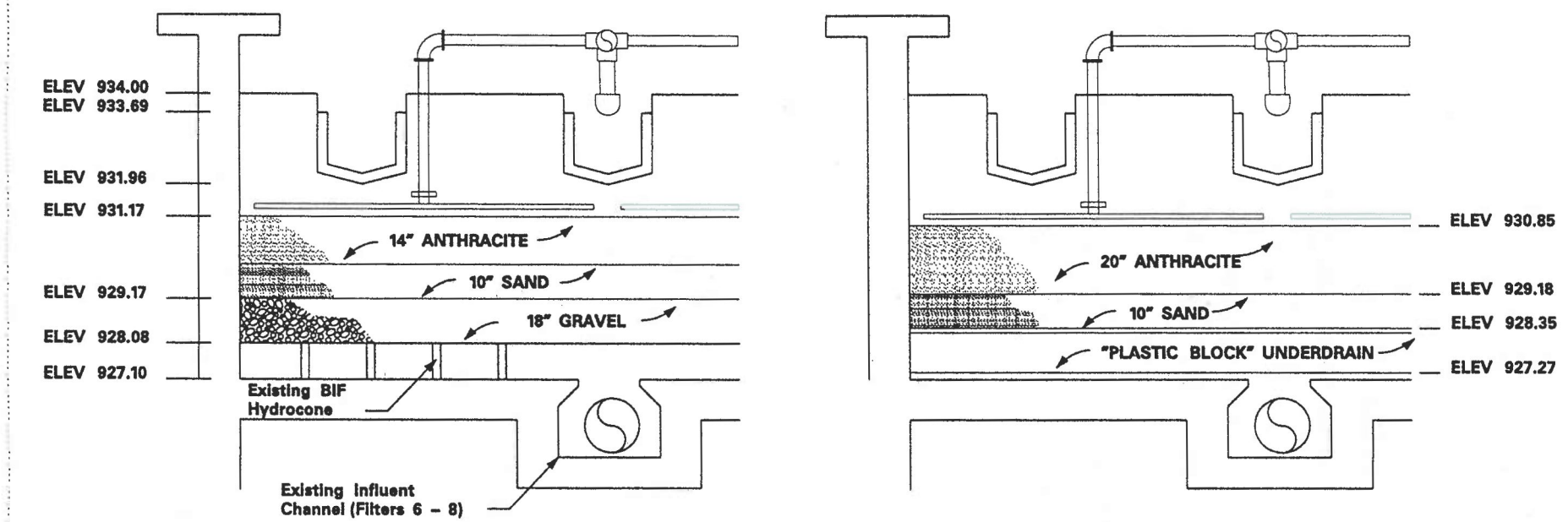
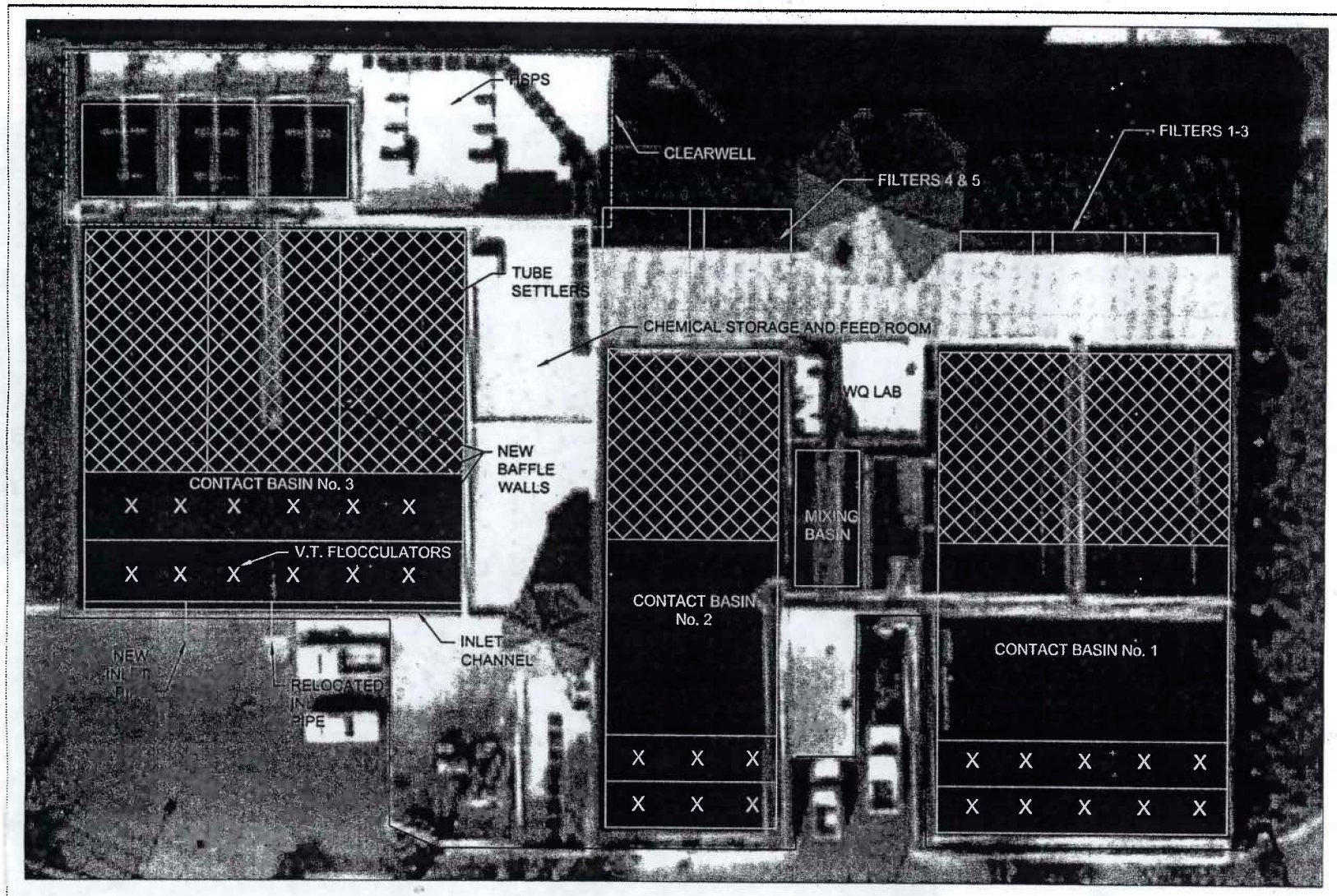


EXISTING FILTER DESIGN

RECOMMENDED FILTER MEDIA/UNDERDRAIN

MWH
MONTGOMERY WATSON HARZA

Figure 6–1
City Of Grants Pass
WTP Facility Plan
Recommended Filter Modifications



Basin 3 Modifications

- Remove walkway and center feed
- Remove launders
- Relocate inlet pipe
- Re-shape floor for sludge removal
- Continuous sludge removal system (Not Shown)

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Figure 6-2
City of Grants Pass
Water Treatment Plant Facility Plan
Suggested Basin Improvements for
30 MGD Expansion

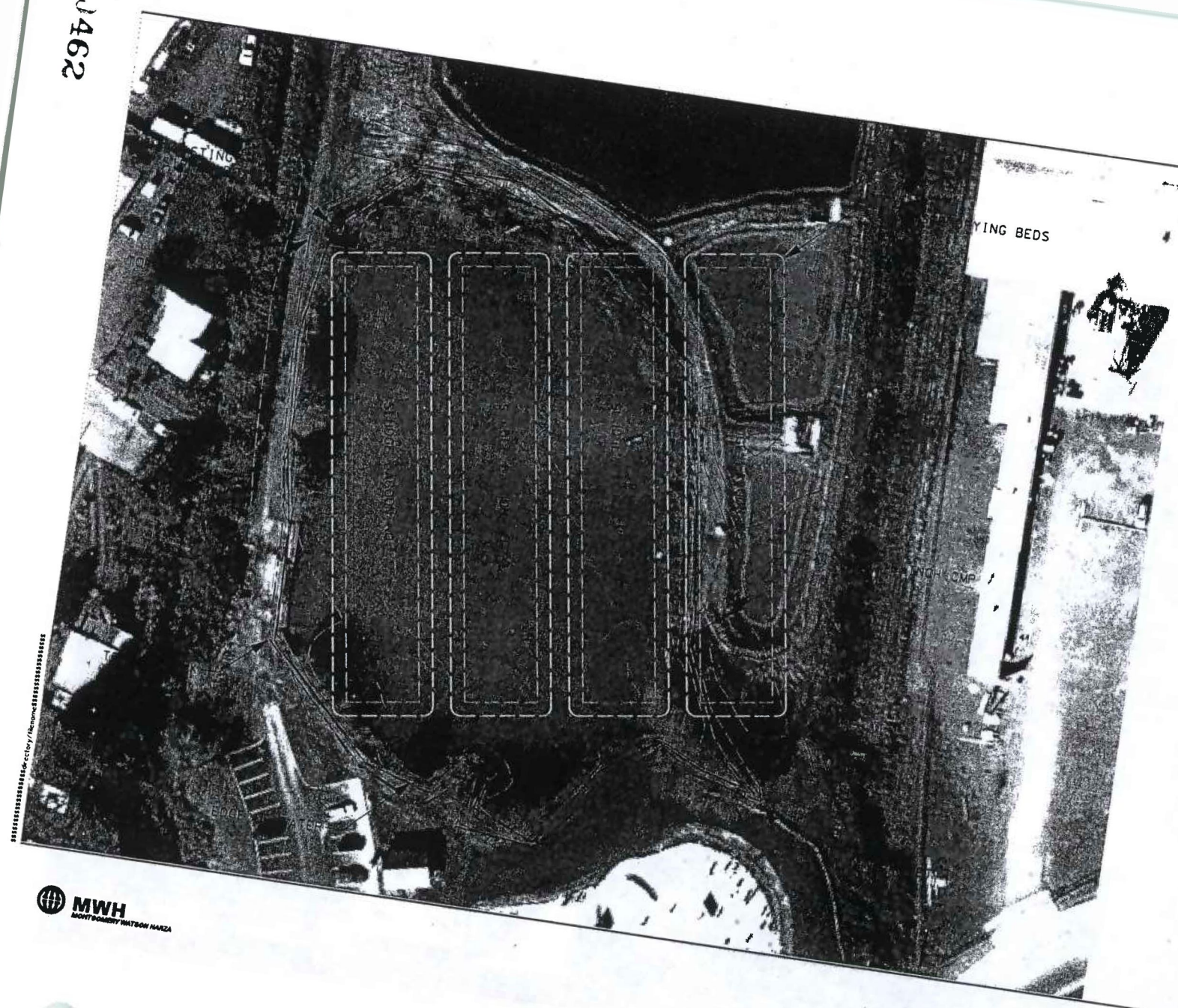
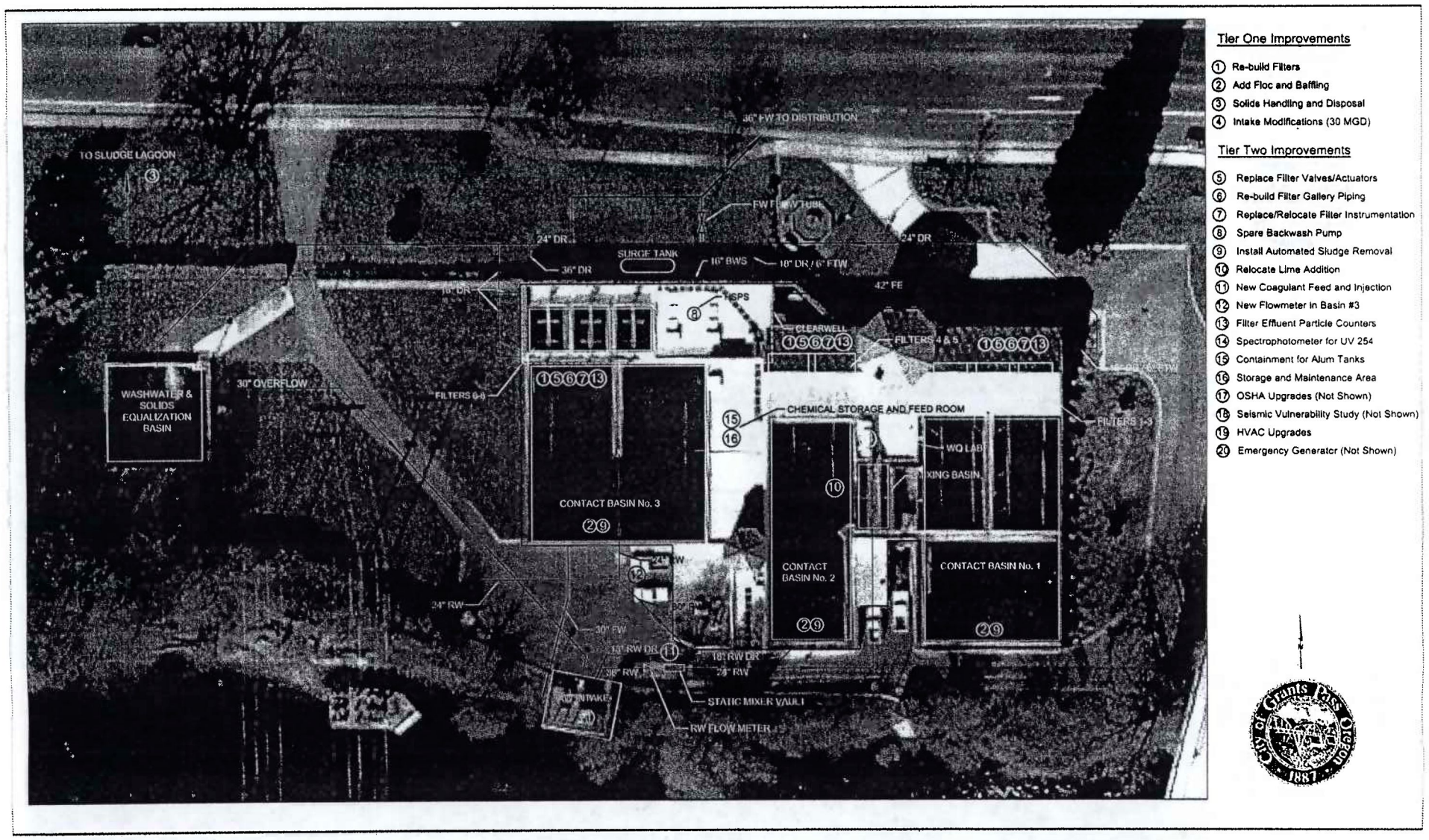


Figure 6-3
City Of Grants Pass
Water Treatment Facility Plan
Sludge Lagoon Improvements
(If Needed)



Tier One Improvements

1. Re-build Filters
2. Add Floc and Baffling
3. Solids Handling and Disposal
4. Intake Modifications (30 MGD)

Tier Two Improvements

5. Replace Filter Valves/Actuators
6. Re-build Filter Gallery Piping
7. Replace/Relocate Filter Instrumentation
8. Spare Backwash Pump
9. Install Automated Sludge Removal
10. Relocate Lime Addition
11. New Coagulant Feed and Injection
12. New Flowmeter in Basin #3
13. Filter Effluent Particle Counters
14. Spectrophotometer for UV 254
15. Containment for Alum Tanks
16. Storage and Maintenance Area
17. OSHA Upgrades (Not Shown)
18. Seismic Vulnerability Study (Not Shown)
19. HVAC Upgrades
20. Emergency Generator (Not Shown)

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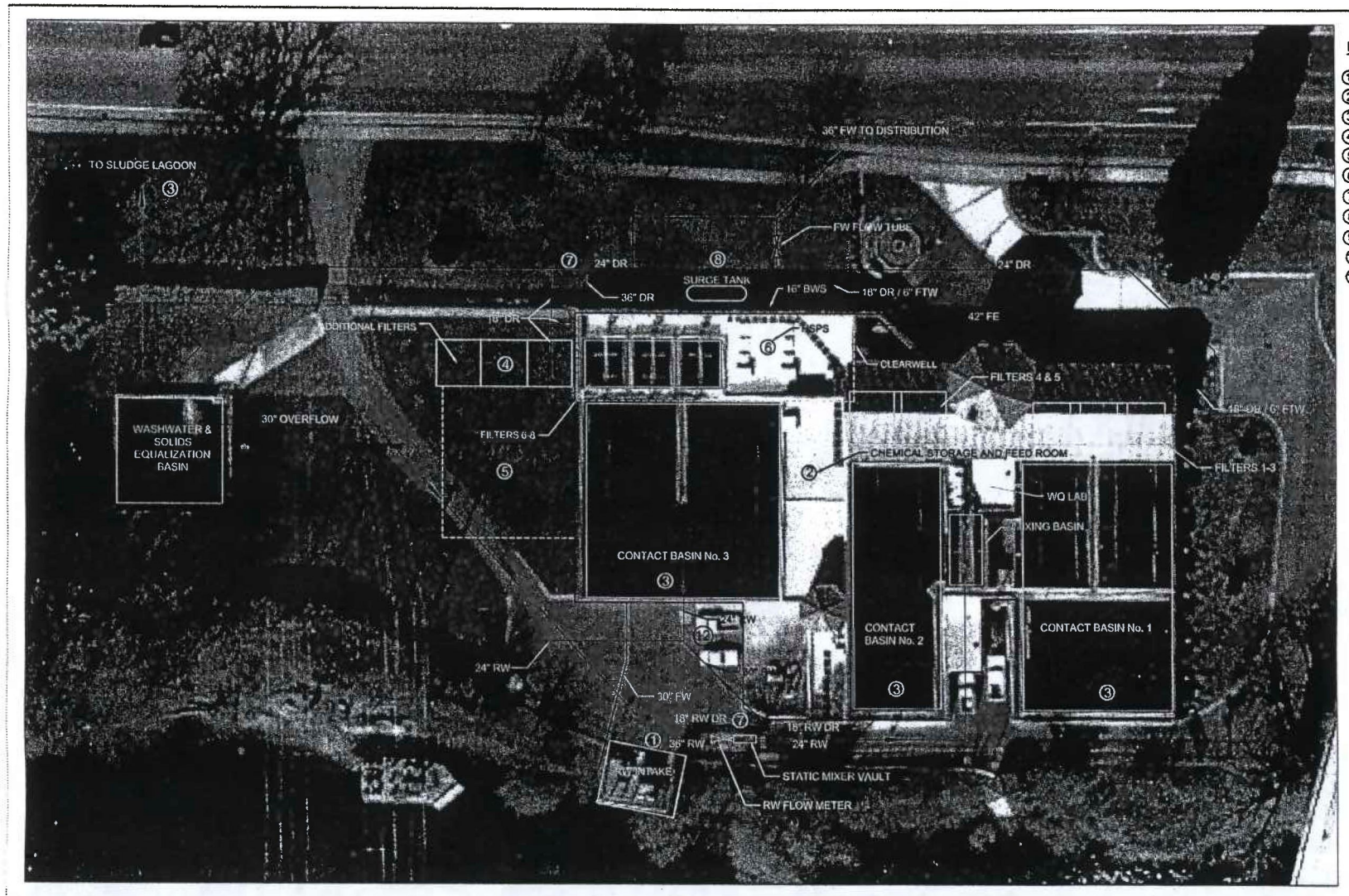
Figure 6-4
City of Grants Pass
Water Treatment Plant Facility Plan
Improvements to Maintain Capacity

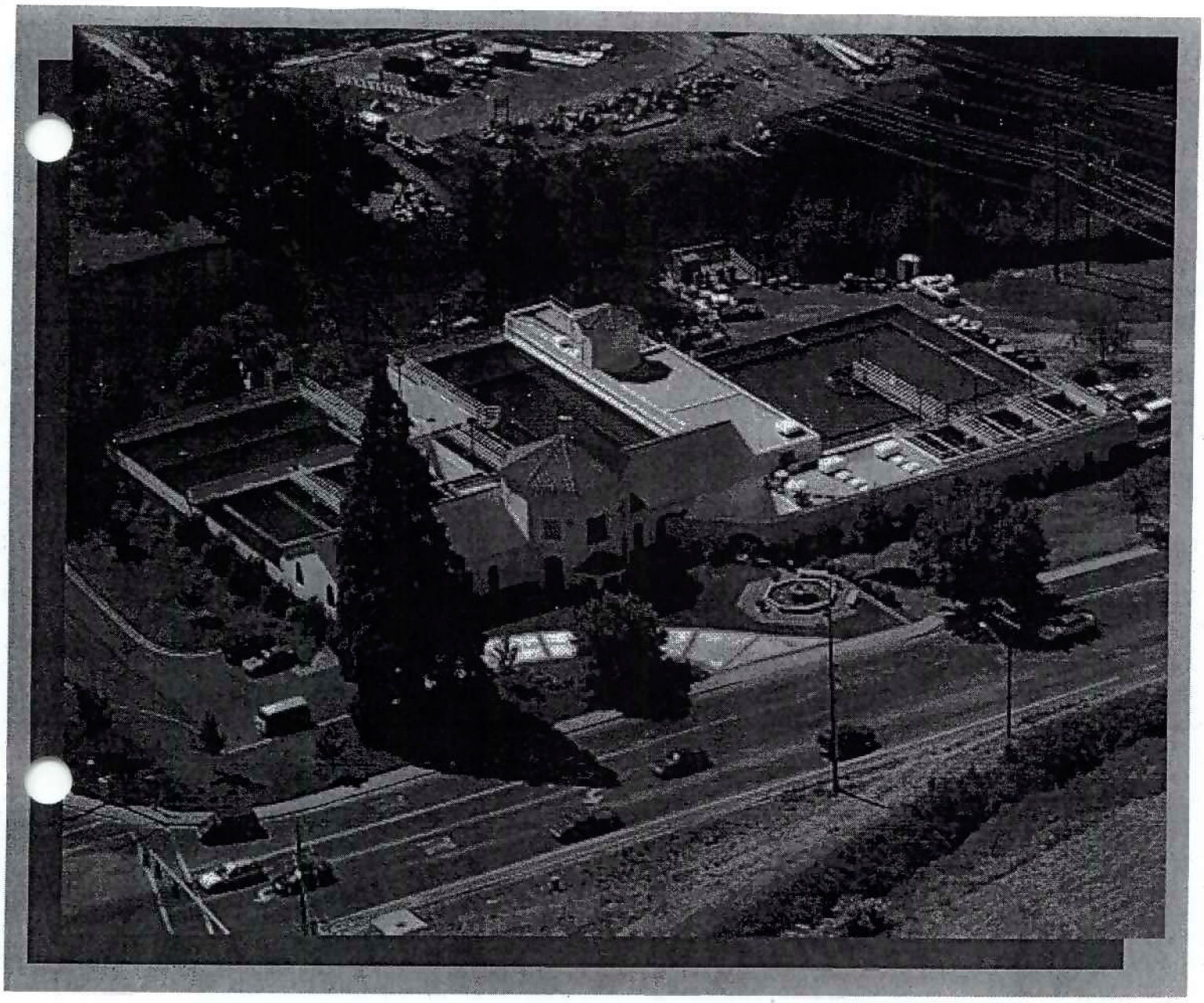


Plant Improvements (30 MGD)

1. New Raw Water Pumps (2)
2. Chemical System Improvements
3. Floc/Sed Basin Improvements
4. Three New Filters
5. Additional Clearwell Volume
6. Two New High Service Pumps
7. Yard Piping Improvements
8. Surge Control Improvements
9. Increase Site Electrical (Not Shown)
10. Electrical for Upgrade (Not Shown)
11. Instrumentation/Control for Upgrade (Not Shown)

MWH

Figure 6-5
City of Grants Pass
Water Treatment Plant Facility Plan
Improvements to Expand to 30 MGD



Recommendations and Implementation Plan

7 RECOMMENDATIONS AND IMPLEMENTATION PLAN

Based on the recommended improvements and facility planning presented in Section 6, and after further discussion with the City, the following summarizes the key elements for ensuring that the Grants Pass WTP continues to serve the City's needs for the next 20 years and beyond:

- Immediate (Tier-one) improvements to maintain capacity, improve and/or optimize performance, and continue to meet regulations;
- Longer-term (Tier-two) improvements to upgrade facilities and to ensure continued long-term performance; and
- Expand plant capacity to 30 mgd in the next 20 to 25 years, depending on growth and demands

These improvements were prioritized and scheduled to fit within the City's budgetary constraints. The City will also be implementing some capital improvement projects at the WTP, as recommended from the Vulnerability Assessment (VA), over the next few years. The VA improvements are not discussed herein.

7.1 IMMEDIATE (TIER-ONE) PLANT IMPROVEMENTS

The City's WTP is in need of immediate improvements in four areas to ensure that it can continue to reliably treat the 20 mgd rated capacity, to optimize performance and to continue to meet regulations as described in Section 6. The City should implement these projects within the next five years depending on priorities and to meet budgetary limitations. The project team discussed, evaluated and rated the improvements and developed the following prioritized list in order of implementation:

1. Solids Handling and Disposal Modifications
2. Intake Modifications
3. Filter Modifications
4. Basin Modifications

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IMPLEMENTATION PLAN

The solids handling and disposal improvements were rated the highest of the Tier-one projects because the City needs to remove solids from the lagoon as soon as possible to allow the plant to continue successful operations, and to avoid potential NPDES permit violations. The City has elected to implement Option 6 as described in **Section 6**.

The City intends to purchase a dredge and some "GeoTubes", and also make improvements around the existing lagoon to allow immediate and periodic removal of solids by City staff. Initially, solids from the lagoon will be dredged and pumped into at least two "Geo-Tubes" which will dewater the solids inside the tube via proper selection of fabric material and dewatering polymers. The tubes will be located at the perimeter of the lagoon and allowed to drain back to the lagoon. As liquid drains through the fabric, additional solids will be pumped into the tube until it is full. It is expected that the tubes will provide adequate dewatering over the summer period. The dewatered solids will then be removed from the tube and hauled to JO-GRO™ or to a landfill. Tubes will be re-filled periodically throughout the year and then allowed to dewater over each summer. It is initially estimated that the City will perform dredging operations 12 times per year.

Each tube will be approximately 9.5-feet diameter by 350-feet long (25,000 cf = 185,000 gallons), which means it can theoretically contain over 1.0 million pounds of 20% solids content sludge assuming a 70 lb/cf material density. One tube may be able to contain and dewater up to 25% of the total lagoon contents if the dewatering process performs well.

The City also intends to use Geo-Tubes to dewater the solids removed from the basins without discharging them to the lagoon as currently practiced. The solids removed from the basins will be diverted to the existing holding basin and mix tank previously used for powdered activated carbon (PAC). The solids will remain stirred, and then will be pumped into a Geo-Tube on the plant site while also feeding polymer. This approach, if successful, should limit solids accumulation in the lagoon and reduce its dredging frequency, since a large percentage of the lagoon solids come from cleaning the basins.

Purchasing the dredge also offers the City the ability to pump liquid solids into tanker trucks and haul the solids to an off-site location, for possible future storage and dewatering, as discussed in Section 6. This allows the City to further consider the possibility of discharging these solids at the WRP, or at the Redwood Pump Station Site, among other options. This approach significantly reduces the capital investment compared to the lagoons/drying beds (Option 1) and may be able to defer major capital investments for solids handling and disposal for many years if the process proves manageable and reliable.

The intake modifications were rated the second-highest priority due to the non-compliance with fish protection (screening) criteria and the inherent risk that this creates for the City. The sooner the City begins efforts to bring the intake into compliance, the sooner it can also assure itself of getting the intake approved for 30 mgd withdrawal rates to firm up its water rights on the Rogue River. This is the only Tier-one project that has a significant capacity expansion component since the intake will be sized for 30 mgd. It is estimated that one-third of the project cost should be allocated to capacity expansion.

The filter modifications were determined to be an important improvement since they represent the "heart" of the plant and this process requires upgrades within the next few years. Filter improvements were rated a higher priority than basin improvements and will likely lead to higher year-round improvements in plant production efficiencies, which should result in shorter plant operating periods to make the same amount of water compared to today's conditions.

The basin modifications were also determined to be an important improvement. If coagulation modifications are successful in lowering alum doses and improving overall plant performance, it may be possible to defer the implementation of formal flocculation. Also, the City should consider the recommendation for continuous sludge removal in the basins as part of Tier-two improvements.

Each of the Tier-one projects and the recommended timelines are presented below.

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7.1.1 Solids Handling and Disposal Improvements

Due to the urgent nature to remove solids from the lagoon, the City has accelerated this project to purchase the new dredge and Geo-Tubes, as well as make improvements at the lagoon site, in early 2004. The goal is to remove an adequate amount of solids from the lagoon (and into the GeoTubes) in Spring 2004 prior to the high demand season which usually begins in May. Solids from the basins will be dewatered with a separate Geo-Tube located at the plant. The City has allocated approximately $175,000 for this project.

7.1.2 Intake Modifications

The bulk of the construction work needs to occur during July and August to meet the in-water work period for the Rogue River. It may take 12 months or more to get the proposed intake modifications approved by various regulatory agencies considering their current backlogs. Therefore, the earliest likely construction period is summer 2006 assuming the City begins predesign and permitting efforts in mid-2004. A suggested schedule to complete the intake modifications is shown below.

- Begin predesign and permitting — July 2004
- Begin detailed design — January 2005
- Begin bid period — July 2005
- Issue construction Notice to Proceed — October 2005
- Complete shop drawing reviews — January 2006
- Delivery of Critical Equipment and Materials by — May/June 2006
- Construction complete — November 2006

7.1.3 Filter Upgrades

Due to funding constraints, the City can not begin design of this project until mid-2005. This means that construction can not begin until Fall 2006 because no filters can be out of service during the peak demand season. A suggested schedule to complete the filter modifications is shown below.

- Begin design — July 2005
- Begin bid period — November 2005
- Issue construction Notice to Proceed — January 2006
- Complete shop drawing reviews — April 2006
- Delivery of Critical Equipment and Materials by — September 2006
- Construction complete — April 2007

7.1.4 Basin Modifications

The City can complete this project in parallel with the filter upgrades. Designing and constructing both projects as one "package" will reduce total costs and minimize plant disruptions during construction to only one off-season. A suggested schedule to complete these improvements is shown below.

- Begin design — July 2005
- Begin bid period — November 2005
- Issue construction Notice to Proceed — January 2006
- Complete shop drawing reviews — April 2006
- Delivery of Critical Equipment and Materials by — September 2006
- Construction complete — April 2007

7.2 TIER-TWO PLANT IMPROVEMENTS

The Grants Pass WTP requires additional improvements for reliability, long-term efficiency, and to meet regulations including:

- Replacement of all pneumatic valve actuators with electric actuators, with new valves
- Re-build filter gallery piping for longevity and protection against damage
- Replace and re-locate filter effluent flowmeters
- Purchase a spare backwash pump
- Install continuous sludge removal systems in each basin
- Relocate lime addition point and modify inter-clearwell piping

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- Install new coagulant aid feed system
- Install new flowmeters for Basin #3 influent, raw water and finished water
- Provide containment around the alum tanks
- Provide a new storage and maintenance area
- Install filter effluent particle counters
- Purchase a new spectrophotometer
- Implement HVAC upgrades
- Perform a Seismic Vulnerability Study
- Install an emergency power generation system for 5 mgd

As presented in ***Table 6-7***, the estimated total project costs are $1.8 million for these improvements. Implementation of these improvements will follow Tier-one improvements and will depend on budgeting constraints and other issues. The project team discussed, evaluated and rated the Tier-two improvements and developed the following prioritized list in order of preferred implementation:

1. Replace filter gallery valves, actuators, piping and flowmeters
2. Upgrade/replace chemical feed systems and containment
3. Install continuous sludge removal systems in basins
4. Construct a new storage/maintenance building
5. Emergency generator

The City will implement the smaller cost items (such as a spare backwash pump, water quality analytical equipment, HVAC upgrades and the Seismic Study) as funds are available and perhaps via the plant operations budget.

If additional plant testing identifies that a coagulant aid or alternative coagulant system should be implemented, then this project should be accelerated and possibly integrated with one of the other Tier-one plant improvement projects (filter or basin improvements) to reduce sludge production and reduce operating costs. It may be possible to perform

this work as part of the operations budget. The City should also consider accelerating the lime relocation project if operating budget funds are available.

7.3 PLANT EXPANSION TO 30 MGD

The City will need to expand the plant's capacity in approximately 25 years (on-line by 2028) if demand growth is steady at 2.5% per year. The expansion would be required in 20 years (on-line by 2023) if demand growth is steady at 3% per year. A plant expansion to 30 mgd is estimated to have a project cost of $7.5 million, as presented in ***Table 6-8***. The size and scope of this project will require 3 to 4 years to implement. A preliminary schedule is indicated below, assuming the increased demand is required prior to the summer of 2028. When preliminary design begins, the City will have to decide whether to increase the plant capacity by 10 mgd or consider a smaller increment of expansion (minimum 5 mgd) which would defer some costs until later, but would be less efficient that expanding to 30 mgd at one time.

- Begin preliminary design — July 2025
- Begin bid period — September 2026
- Issue Construction Notice to Proceed — December 2026
- Construction complete — May 2028

7.4 SHORT-TERM SCHEDULE AND FINANCIAL PLANNING

Based on the preferred implementation schedule for Tier-one and Tier-two improvements, the following summarizes estimated expenditures for the next nine fiscal years. Each fiscal year begins July 1 and ends on June 30 of the following year. The costs shown below have not been adjusted for inflation.

Fiscal Year 2003/2004 (Current)

- Solids Handling Improvements = $175,000

Fiscal Year 2004/2005

- Intake design and permitting = $400,000

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Fiscal Year 2005/2006

- Intake construction = $500,000
- Filter upgrades (design and construction) = $200,000
- Basin improvements (design and construction) = $200,000

Fiscal Year 2006/2007

- Intake construction = $700,000
- Filter upgrades construction = $400,000
- Basin improvements construction = $400,000

Fiscal Year 2007/2008

- None

Fiscal Year 2008/2009

- Filter Gallery upgrades (design and construction) = $200,000

Fiscal Year 2009/2010

- Filter Gallery upgrades construction = $480,000
- Chemical System upgrades (design and construction) = $50,000

Fiscal Year 2010/2011

- Chemical System upgrades construction = $130,000
- Continuous sludge removal systems (design and construction) = $75,000
- New storage/maintenance building (design and construction) = $25,000

Fiscal Year 2011/2012

- Continuous sludge removal systems construction = $225,000
- New storage/maintenance building construction = $50,000

Fiscal Year 2012/2013

- Emergency generator for 5 mgd production = $300,000

The City will need to budget for these recommended improvements. Those projects which allow increased capacity, including the intake modifications, should be funded via SDC (System Development Charges) mechanisms. It may be possible to have the Corps of Engineers participate in the costs of the intake modifications project due to the eddy in front of the intake which was created during the riverbank stabilization project in 2001.

GRANTS PASS WATER RESTORATION PLANT

WASTEWATER FACILITIES PLAN UPDATE

Prepared for:

City of Grants Pass
101 Northwest A Street
Grants Pass, Oregon 97526

Prepared by:

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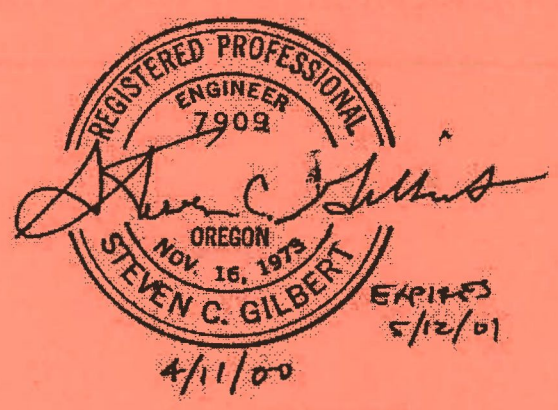
PMX #216-3416-002
June 2001

EXHIBIT 7
to UAPC Staff Report
Page 475

CERTIFICATE OF ENGINEER

The technical material and data contained in the revised sections of this document were prepared under the supervision and direction of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.



Prepared by: April Huff

Reviewed by: Steven C. Gilbert, P.E.

Checked by: Tom Nielsen, P.E.

Approved by: Doug Berschauer, P.E.

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PREFACE

This Grants Pass Facilities Plan Update is a revised edition of the January 1999 Grants Pass Facilities Plan (FP) prepared by Brown and Caldwell. The purpose of this revision came about in a Value Engineering (VE) Workshop held in June 1999. Flow and population projections were reevaluated and other treatment alternatives were conceptualized. The proposed liquid and solids treatment alternatives by Brown and Caldwell have remained as alternatives. However, because of the recalculation of the flow projections, the liquid stream alternatives were adjusted to treat the VE workshop calculated peak wet weather flow. The biosolids disposal and handling alternative was not affected.

The sections and chapters that have been revised are indicated in the footer with "Grants Pass Facility Plan – Update" and "Revised February 2000." Chapters 1, 9, 10, 11, and 12 were completely rewritten by Parametrix, Inc. In Chapters 2 and 4 where sections or sentences have been changed from the FP, this is indicated by a vertical bar in the margin. The remaining text, tables, and figures have been taken verbatim from the original Brown and Caldwell document.

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CHAPTER 1

EXECUTIVE SUMMARY

In January 1999, Brown and Caldwell completed a Facilities Plan (FP) for the Grants Pass Water Restoration Plant (WRP). A concern regarding the future flow and population projections prompted a Value Engineering (VE) Workshop in June 1999. During this VE Workshop with Parametrix, Inc. and the City, these projections were recalculated and other treatment alternatives for both the liquid and solids streams were conceptualized.

Two reports comprising several other alternatives were produced from the VE, one for the liquid stream and one for the solids stream. Based on feasibility and cost-effectiveness for the site, alternatives were selected from these reports and have been incorporated into this revised edition of the FP. During the VE Workshop, the projected population, flow, Biological Oxygen Demand (BOD), and Total Suspended Solids (TSS) loadings were recalculated. Chapters that contain these projections have been revised. Also, Chapters 9, 10, and 11 have been completely revised in accordance with the new alternatives. The alternatives that Brown and Caldwell chose have also been included; however, the liquid stream alternative has been adjusted to treat the VE projected peak flow of 37.5 mgd. Chapter 12 has been added to address a financial plan model.

BACKGROUND

Since 1935, the City of Grants Pass WRP has operated at its current site. The first plant expansion occurred in 1953, and in 1962 the plant was upgraded to a secondary treatment facility. The present activated sludge process was a part of a major plant renovation and expansion in 1974. In response to concerns of the Department of Environmental Quality (DEQ) regarding discharging directly into the Rogue River and effluent toxicity, more plant improvements occurred from June 1994 through 1996. These improvements included a fourth raw sewage pump, a temporary belt filter press (BFP) to improve wet weather solids disposal, two new rectangular primary clarifiers, a computer-based supervisory control and data acquisition system, and an Ultraviolet (UV) disinfection system. The most recent upgrade to the Grants Pass WRP was the addition of a second UV channel in 1999, which upgraded the disinfection capacity from 21.5 mgd to 43 mgd.

SERVICE AREA

Situated near the Rogue River, the Grants Pass WRP serves the City and the Harbeck-Fruitdale area. By the fall of 2000, the plant will be receiving sewage from the Redwood area. In addition, the North Valley area that includes the North Valley Industrial Park may be connected to the Grants Pass WRP sometime during the next 20 years, since the population and flow are low compared to the contribution from the City (less than 4 percent).

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Currently, a population of 25,500 is being served by the Grants Pass WRP. Estimated growth rates of 1.5 percent for Grants Pass and 1.6 percent for Harbeck-Fruitdale could increase the population to 34,960 by the year 2020. With the addition of Redwood, at a 3.10 percent population growth rate, and a 2 percent growth increase for North Valley, 11,500 additional people could be served by the year 2020.

To calculate the population equivalent for the community and industrial additions, 35 percent (VE Workshop, June 1999) of the total population for the four communities was used. By the year 2020, it is anticipated that the Grants Pass WRP would be serving 62,700 people.

COLLECTION SYSTEM

The existing collection system consists of approximately 110 miles of gravity sewers; one force main approximately 2,000 feet long; and three pumping stations.

Three hundred fifty acres of the downtown area are still serviced by vitrified clay pipe installed before 1927. The portion of the collection system constructed prior to 1964 has shown severe deterioration in both pipe materials and joint integrity. Since 1992, the City has reported 17 sinkholes due to pipe failure. Therefore, the City will be conducting a Sewer System Master Plan in 2001-02 to analyze the current collection system for future corrections of infiltration and inflow.

TREATMENT PLANT

Grants Pass WRP currently treats 4.5 million gallons per day (mgd) of average dry weather flow to a record peak storm flow of 26.5 mgd. The plant is comprised of numerous unit treatment processes for both liquid and solid streams, a control/laboratory building, and a maintenance shop.

The Grants Pass WRP has a 27-mgd peak hydraulic capacity for influent pumping, screening, and primary treatment; a 13-mgd hydraulic capacity for secondary treatment; and a 43-mgd capacity for UV disinfection. Flow exceeding the secondary treatment capacity receives only primary treatment and disinfection. This occurs only a few days a year during wet weather storm conditions.

For final disposal, the Grants Pass WRP land-applies Class B biosolids to farmland near the plant, weather permitting. When wet weather occurs, the biosolids are dewatered at the plant and hauled to the Merlin Landfill.

FUTURE TREATMENT REQUIREMENTS

To meet future requirements, both population and water quality standards as set forth in Chapter 340 of Oregon's Administrative Rules (OAR) must be considered.

Based on 2020 population estimates, future wastewater flow and load estimates were developed as shown in Table ES-1 and Table ES-2, respectively. Also shown in these tables are flow and load conditions that currently exist at the plant.

Table ES-1. Wastewater Flow Projections

	Year/Population					
	1999/42,900		2010/52,200		2020/62,700	
	Actual (mgd)	Actual (Gal/cap-day)*	Design (mgd)	Design (Gal/cap-day)*	Design (mgd)	Design (Gal/cap-day)*
Summer, Max. Month	7.4	173	8.4	160	9.4	150
Winter, Max. Month	11.0	256	12.2	233	13.5	216
Peak Day	22.0	514	24.6	471	27.5	438
PWWF	30.7	715	34.0	652	37.8	603
Summer Average	5.1	120	6.0	113	6.8	109
Winter Average	6.6	155	7.8	149	9.1	145
Annual Average	5.9	137	6.9	131	8.0	127

* This figure is arrived at by dividing the actual mgd by the population.

Table ES-2. Organic and Solids Loading (pounds per day)

	Year		
	1999	2010	2020
Average BOD_5	7,490	9,400	11,400
Maximum Monthly BOD_5	10,985	13,700	16,800
Average NH_3-N	327	390	450
Maximum Monthly NH_3-N	636	760	870
Average TSS	8,564	10,500	12,700
Maximum Monthly TSS	11,623	14,300	17,300

Because the Rogue River is used for salmon spawning, fish passage, and rearing of Coho, wastewater discharge requirements are strict.

To properly evaluate upgrading/expanding the Grants Pass WRP, an analysis of the facilities wastewater discharge permit limit is necessary. Table ES-3 presents the anticipated year 2020 BOD_5 and TSS treatment requirements as mandated by OAR 340-41-375. These values represent the dry weather season BOD_5 and TSS effluent concentrations that the plant should meet monthly.

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Table ES-3. Anticipated Year 2020 BOD_5 and TSS Treatment Requirements Based on OAR 340-41-375

	Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge, lbs/day		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based on Effluent Quality							
Summer, Dry Weather	8	10	15	-	670	1,000	1,300
Winter, Wet Weather	10	30	45	-	2,500	3,750	5,000

TREATMENT ALTERNATIVES

To select the best method for meeting Grants Pass treatment needs in the year 2020, three liquid stream alternatives and four solids stream alternatives were evaluated. The liquid stream treatment includes a conventional expansion alternative, a ballasted sedimentation alternative, and a Zenon process alternative. The biosolids disposal and handling proposes a Merlin Landfill co-compost facility, hauling dewatered biosolids to the Dry Creek Landfill, land applying Class B biosolids, and using an ATP component. The alternatives are summarized below.

Liquid Stream Alternatives

Alternative One – Conventional Expansion. Alternative One is the recommended improvement from the FP using a conventional expansion approach. By replicating the existing components, this option allows continuing ease of operation due to staff familiarity of the processes. This alternative has been modified to treat the peak wet weather flow calculated in the VE Workshop. A summary of the upgrades include:

- Additional mechanical bar screen with a capacity of 23.5 mgd.
- Removal of the four existing influent pumps and replacement with three 19 mgd pumps.
- Additional rectangular primary clarifier. Rehabilitate existing primary clarifiers.
- Two additional aeration basins. Rehabilitate existing aeration basins.
- Two additional 115-foot secondary clarifiers. Rehabilitate existing secondary clarifiers.

- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, and instrumentation and control system expansion.

Alternative Two – Ballasted Sedimentation. Alternative Two uses ballasted sedimentation to treat peak flows greater than 13.5 mgd. It is proposed to convert the existing gravity thickener into the ballasted sedimentation tank. The peak flows would be conveyed to this system and then recombined with the main treatment train to receive UV disinfection. Other upgrades to the Grants Pass WRP include:

- Install additional influent pumping capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.
- Odor containment at the influent pump station and mechanical bar screen area.
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary clarifiers would be rehabilitated.
- Addition of a bioselector in the aeration basin to provide filamentous bacteria control, which would improve the settling performance in the secondary clarifiers. The aeration basin would also be modified with fine bubble diffusers, dissolved oxygen control, and motorized gates.
- Two additional 90-foot secondary clarifiers. Rehabilitate the secondary clarifiers.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

Alternative Three – Zenon Process. Alternative Three incorporates the use of Zenon for secondary treatment. The wastewater can flow directly from the primary clarifiers, thereby eliminating the need for secondary clarifiers. Zenon is a microfiltration membrane system located in a suspended growth biological reactor. For this alternative, the membranes would be placed into the aeration basin to serve as a biological reactor. Other upgrades to the Grants Pass WRP for this alternative include:

- Additional influent pump to meet the projected firm capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.

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- Odor containment at the influent pump station and mechanical bar screen area.
- For peak overflows greater than 13.5 mgd, the existing gravity thickener would be converted into a ballasted sedimentation tank.
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary and secondary clarifiers rehabilitated for enhanced performance.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

Cost Estimate. A preliminary cost estimate for these alternatives is summarized in Table ES-4. Engineering, administration, and contingency are included in these costs.

Table ES-4. Comparison of Liquid Stream Alternatives

Alternative	Capital Cost (millions)
One – Conventional Expansion	$12.23
Two – Ballasted Sedimentation	$13.59
Three – Zenon Process	$23.96

Biosolids Disposal and Handling Alternatives

Four alternatives were evaluated for the solids treatment stream.

Alternative One – Merlin Landfill Co-compost Facility. Alternative One would co-compost digested primary and raw secondary biosolids. The existing digester would be rehabilitated and used only for digesting primary biosolids. A new component for dewatering digested primary biosolids to 15 percent solids would be installed in an existing building located on site. The biosolids would be hauled to the co-composting facility located at the Merlin Landfill. This facility would produce Class A biosolids, which would be available for public purchase.

Alternative Two – Dry Creek Landfill. Alternative Two hauls raw primary and secondary dewatered biosolids to the Dry Creek Landfill. These biosolids would be dewatered at the Grants Pass WRP to 15 percent solids using a belt filter press (BFP). The existing press and one new additional BFP would be installed in an existing building located on site. Extra hauling equipment would be required for handling the transport of all the biosolids to the landfill.

Alternative Three – Land Applying Class B Biosolids. Alternative Three is the recommended alternative from the FP. It is a continuation of the current biosolids management program of long hauling Class B biosolids to be land applied. However, because of an increase in future biosolids production, contracting with large landowners in Eastern Oregon to expand the land application area would be necessary. Biosolids would be dewatered to 15 percent, hauled to the site in large tractor/trailers, and applied with a manure spreader. During the winter, the biosolids would be stored near the application site. An additional gravity thickener, gravity belt, anaerobic digester, and belt filter press are necessary to meet the future solids production. The existing anaerobic digester would need to be rehabilitated. Class A biosolids can be produced by adding low cost aeration equipment in the future storage building to create a pilot-scale co-composting facility.

Alternative Four – Aerobic Thermophilic Pretreatment (ATP). Alternative Four is to produce Class A biosolids and distribute them to the public as fertilizer. To accommodate increasing loads in the future, an ATP would be installed instead of adding a new digester. To reduce plastics in the biosolids, a Muffin Monster would be added prior to the ATP. The existing digester would need to be rehabilitated to enhance performance, increase capacity, and repair deficiencies.

Cost Estimate. Preliminary cost estimates for the four alternatives are in Table ES-5. These values include engineering, administration, and contingency costs.

Table ES-5. Comparison of Biosolids Disposal and Handling Alternatives

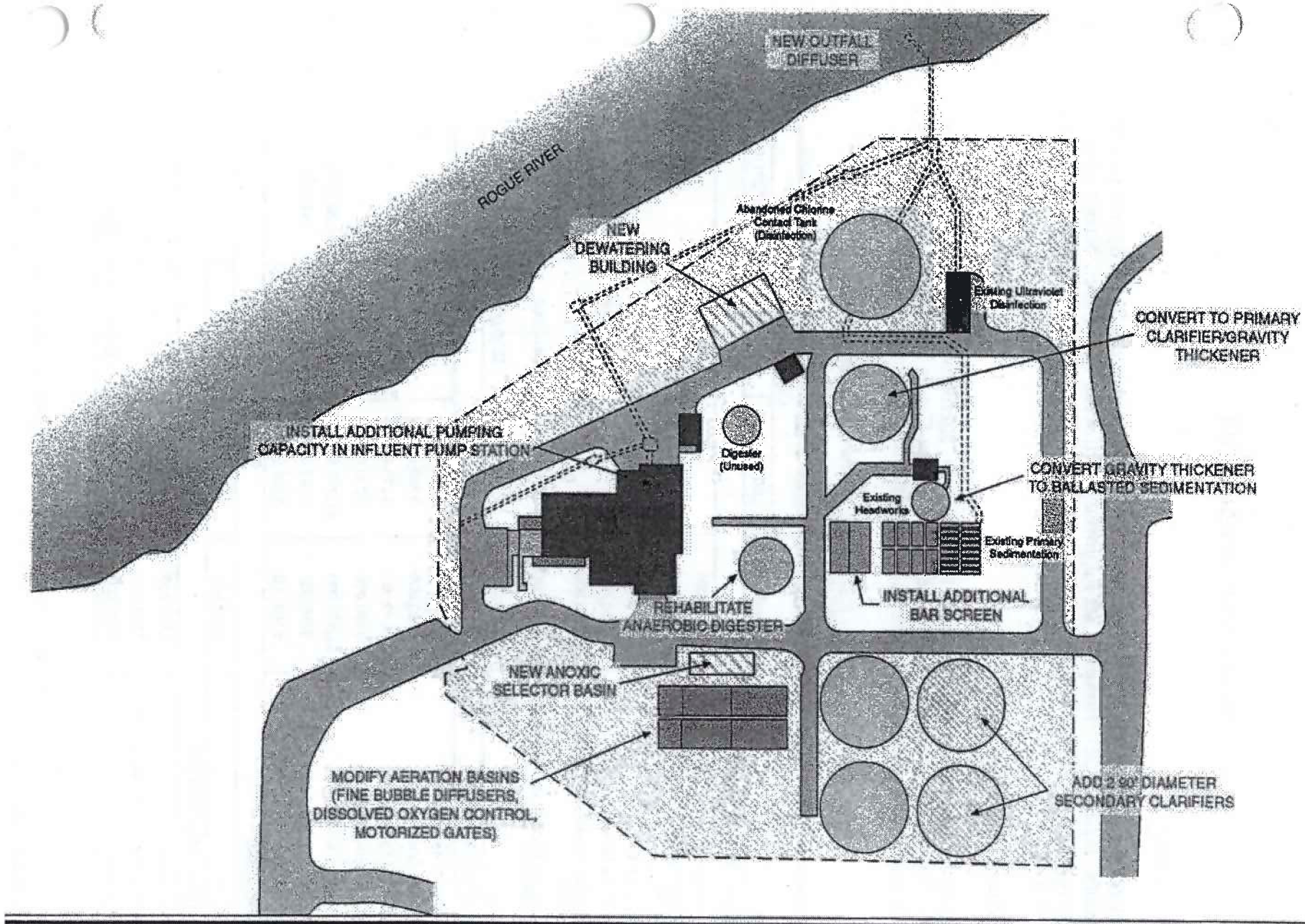
Alternative	Capital Cost (millions)
One – Merlin Landfill Co-compost Facility	$3.60
Two – Dry Creek Landfill	$1.40
Three – Land Apply Class B Biosolids	$11.00
Four – Aerobic Thermophilic Pretreatment (ATP)	$2.30

PREFERRED ALTERNATIVE SELECTION

To reduce the complexity of evaluating the alternatives for the liquid and solid stream treatments, two preliminary screening processes were conducted based on cost and feasibility. A list of advantages and disadvantages for each alternative was tabulated to assist in the decision making. These tables are located in Chapters 9 and 10. The result of this screening process was selection of one liquid and solids stream alternative.

Cost Estimate

The preferred alternative for the liquid and biosolids treatment streams and the collection system are located in Table ES-6. All costs are in the 1999-2000 dollar amounts costs and include engineering, administration, and contingency.



Parametrix, Inc. City of Grants Pass Facilities Plan/216-3416-002(02) 01/00 (S)

Figure ES-1
Preferred Alternative
Site Plan

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PREFERRED ALTERNATIVE

Collection System

Improvements to the collection system will further be investigated in the Sewer System Master Plan scheduled for 2001-02. However, set improvements include Pine Street, Second Street, and Western Avenue. Costs for these specific locations and the Master Plan have also been included in Table ES-6.

Liquid Stream Treatment

The ballasted sedimentation alternative is the preferred alternative for the liquid stream treatment. Figure ES-1 presents the proposed layout for the Grants Pass WRP. Below is the anticipated time line of when the components or upgrades would occur at the Facility.

Table ES-6. City of Grants Pass - Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001-2004	2005-2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping	$560,000		$560,000	
Raw Sewage Pipeline to Ballasted Sedimentation	$240,000		$240,000	
Screening Odor Control	$390,000	$390,000		
Mechanical Bar Screen No. 2	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000		$3,510,000	
Modify Existing Primary to Combination Clarifier/Thickener	$610,000		$610,000	
Yard Piping	$270,000		$270,000	
SECONDARY TREATMENT				
Aeration Basin Fine Bubble	$530,000	$530,000		
Aeration Basin Selector	$260,000	$260,000		
Blowers and DO Control	$870,000	$870,000		
Rehabilitate Existing Clarifiers	$770,000	$770,000		
New Secondary Clarifiers	$2,400,000	$1,200,000		$1,200,000
Yard Piping	$770,000	$400,000		$370,000
Motorized Gates	$340,000	$340,000		
FINAL TREATMENT				
Outfall Diffuser	$540,000	$540,000		
OTHER PLANT IMPROVEMENTS				
Lab Improvements	$100,000	$100,000		
Operation Building Repairs and Office	$130,000	$130,000		
SCADA System Expansion	$670,000	$250,000	$250,000	$170,000

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Table ES-6. City of Grants Pass – Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001-2004	2005-2006	2010-2011
Equipment Improvements from Audit Results	$250,000	$100,000	$75,000	$75,000
Plant Landscaping	$100,000	$40,000	$30,000	$30,000
Public Education Program	$50,000	$20,000	$20,000	$10,000
SOLIDS THICKENING AND DIGESTION				
Rehabilitate Existing Digester	$740,000	$740,000		
Dewatering Centrifuge	$1,000,000	$1,000,000		
SOLIDS HANDLING OFF-SITE				
Co-composting Facility	$1,850,000	$1,850,000		
COLLECTION SYSTEM				
Pine Street	$1,050,000	$1,050,000		
2nd Street	$700,000	$700,000		
Western Avenue	$580,000	$580,000		
Master Plan	$170,000	$170,000		
TOTALS	$19,680,000	$12,030,000	$5,795,000	$1,855,000

* 1999-2000 dollars.

Year 2000

- Install odor containment and control at the influent pump station and mechanical screening areas of the plant.
- Add an anoxic selector basin to the aeration basin.
- Modify the existing aeration basin, convert the existing aeration system to fine bubble diffusers, and add dissolved oxygen control and motorized gates. This would require new or modified aeration blowers.
- Rehabilitate the existing secondary clarifiers to correct short circuiting and flow distribution problems.
- Add a third secondary clarifier.
- Begin laboratory upgrades and improvements.
- Begin operations building repairs and modifications.
- Begin instrumentation and control system expansion.
- Conduct a plant equipment audit.

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- Continue to install plant landscaping.
- Develop and institute a public education program.

Year 2005

- Install additional influent pumping capacity.
- Add a second mechanical bar screen.
- Convert the existing circular primary clarifier to a combination primary clarifier/gravity thickener.
- Modify the existing gravity thickener to a ballasted sedimentation tank to treat peak flow.

Year 2010

- Add a fourth secondary clarifier.

Biosolids Handling and Disposal

For the biosolids handling and disposal, the Merlin Landfill Co-compost Facility (Alternative One) is the preferred alternative. This alternative would consist of the existing digester, a new dewatering device, and a co-composting facility.

Under this solid waste handling system, rehabilitated digesters would be used to treat only primary clarifier solids. The secondary clarifier solids would be combined with the digested primary solids, dewatered in a new dewatering component, and trucked to the new co-composting facility located at the Merlin Landfill. The existing belt dewatering system would remain on site for standby or emergency use.

This plan goes into effect immediately.

FINANCING PLAN

Various funding alternatives exist for the City of Grants Pass to implement the Capital Improvement Plan (CIP). The CIP includes three major elements: Treatment Plant (liquid treatment), Biosolids, and Collection System. To fund these elements, the City would be using revenue bonds from monthly user charges and system development charges, State Revolving Funds (SRF), and grants. It is projected that the City of Grants Pass will apply for SRF loans in the 2001–2002 fiscal years. Therefore, the financial situation of the City's wastewater utility is excellent. Given the ability to secure loans from the SRF program and the sound fiscal management of the utility, the City will be able to fully fund the CIP as outlined in this Facility Plan.

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CHAPTER 2

STUDY AREA CHARACTERISTICS

Development of sound, long-range sewerage plans for the Grants Pass area requires consideration of both natural and socioeconomic environmental characteristics. The natural environment, including topography, geology, soils, climate, and water resources, is the primary determinant of development in the study area. Other factors, such as economic activity, population growth, and land use, interact with the natural resources of an area and may profoundly affect the overall environment as development continues. This section defines the study area and discusses the characteristics of both its physical environment and its economic environment.

SEWERAGE STUDY AREA

The area covered by this study is consistent with the sewer service area described in the May 1983 *Sewage Collection System Master Plan.*[1] The study area follows the urban growth boundary (UGB) defined in the *Grants Pass Comprehensive Plan for Community Development*[2] and includes the City of Grants Pass and the unincorporated Harbeck-Fruitdale area south of the city. The Harbeck-Fruitdale area is a special service district that contracts with the City of Grants Pass for sewerage services.

In addition, the sewerage study area has been expanded to include the Redwood and North Valley areas. Portions of the unincorporated Redwood area lie outside of the UGB. This area, bordered by the Rogue River to the north and Allen Creek to the east, is served by a separate sewage collection and treatment system historically operated by the county. The North Valley, also known as Merlin, lies outside the UGB to the north of the city and has largely been dependent on on-site sewage treatment systems. In the near future, these two districts may begin contracting with the City of Grants Pass for their sewerage services. Both the sewerage study area and the UGB are illustrated on Figure 2-1.

PHYSICAL ENVIRONMENT

The physical environment includes the topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items as they relate to the sewerage planning program. This information has been updated from Brown and Caldwell's 1985 *Facilities Plan, Financial Plan, and Rate Study*[3] when possible.

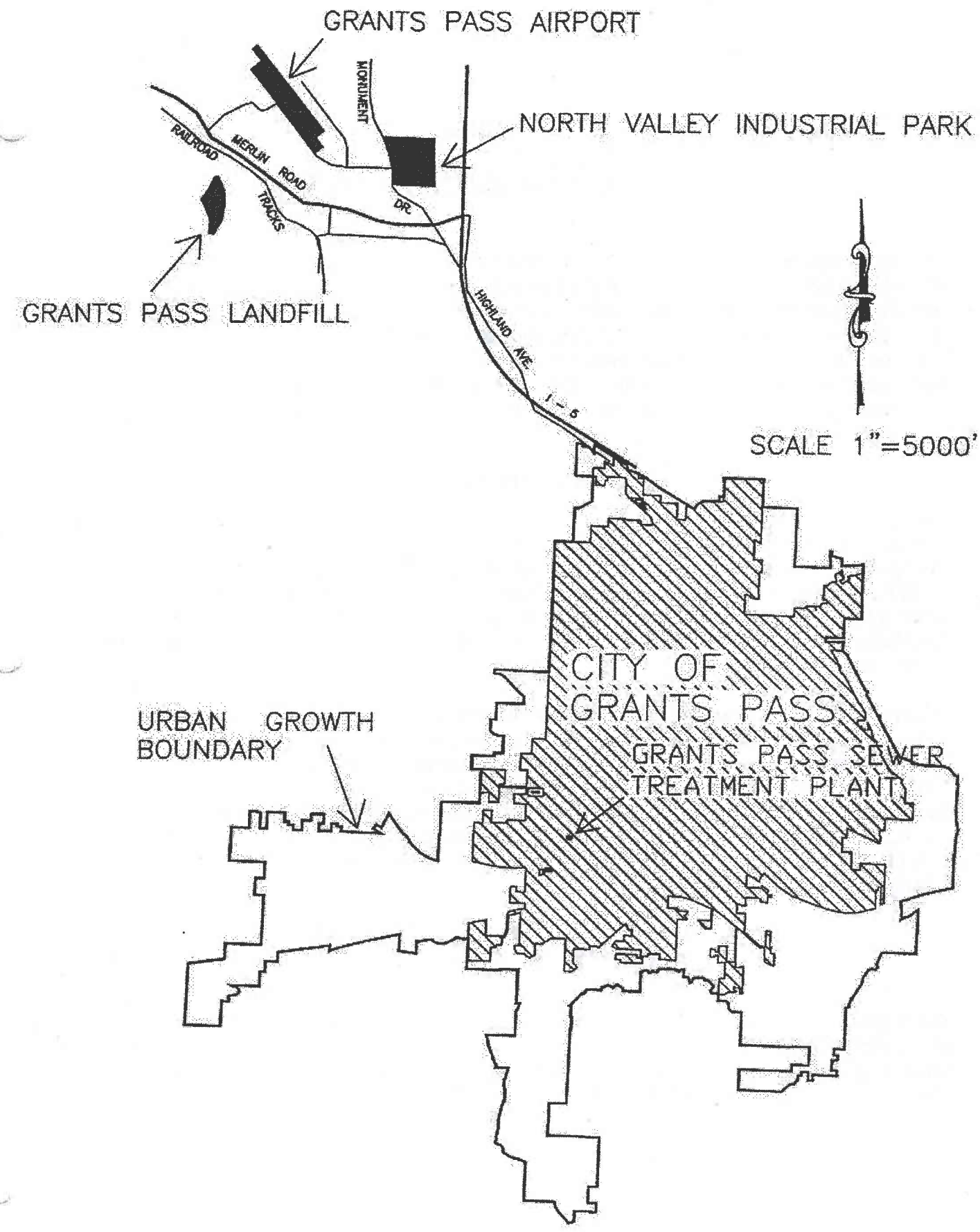

Figure 2-1. Study Area

Topography, Geology, and Soils

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

Topography. Grants Pass lies in the Rogue River Valley in the Klamath Mountain Range of Oregon. The Siskiyou Mountains, part of the Klamaths, lie to the south and west of Grants Pass. To the northeast a spur connects the Klamaths to the Cascade Range.

Away from the valley floor, the terrain quickly grows steep. The lava and metavolcanic rock composing Beacon Hill (elevation 2,117 feet) and Baldy Mountain (elevation 2,740 feet) to the northeast and southeast of the city does not weather easily. Its ruggedness has limited development in these areas. The softer granite of Dollar Mountain to the northwest and various hills to the south and southwest of the city shows greater weathering. Their rounded ridges and gentle slopes have generated alluvium, encouraging development in these areas.

The Rogue River Valley begins at the base of the surrounding hills and exists as a well-defined stream terrace some 10 to 15 feet above the bed of the Rogue River. The valley slopes toward the river at an average gradient of 1 to 2 percent and is composed of relatively flat-lying alluvium. Elevations on the low-lying valley floor range from 880 to 1,100 feet above sea level. The Rogue River traverses the valley in a general east-west direction on an average slope of about 6 feet per mile.

Geology. The Klamath Mountains are composed largely of Paleozoic and Mesozoic metamorphic rocks derived from sedimentary and volcanic formations. Intrusions of granitic and ultrabasic rocks are common.[4] The mixed assemblage is probably responsible for the distinctive mineralogy of the region. The presence of gold, copper, and mercury led to the region's history of mining. The Almeda mine, for instance, was located a few miles to the northwest of Grants Pass.

The study area contains several major geologic units. The large deposits of alluvium which constitute the valley floor date from the Pleistocene epoch. At that time, uplifting of the coastal areas of the Klamath Mountains reduced the sediment-carrying capacity of the river, thus forming the valley floor.[5] The alluvium reaches thicknesses of 100 to 150 feet in places near the river.[6]

Diorites and granites dating from the Upper Jurassic and Lower Cretaceous occur as irregular masses throughout the study area. Dollar Mountain and the Cathedral Hills Park area are two places where these granitic intrusive rocks are prominent. Many of these rocks are quite weathered.

Ultramafic intrusive rocks are less common. Small outcroppings occur northeast of the city. These serpentines, peridotites, and greenstones were formed during the Upper Jurassic epoch.

Northeast and southeast of Grants Pass lie the greenstones of the Applegate Group. These metavolcanic rocks form Baldy Mountain and are present along both forks of Jones Creek. They date from the Upper Triassic.

Other members of the Applegate Group are gneisses and schists found along Fruitdale Creek and forming Beacon Hill. These occur mainly as contact metamorphic zones along intrusive granites. Along with the greenstones, they are relatively resistant to weathering.

Soils. Weathering of the different geologic units has given the soils of this area a wide range of characteristics.

The soils that underlie the developable portions of the Rogue River Valley are of the greatest importance to the sewerage study. A survey conducted by the U.S. Department of Agriculture[7] identified the soil types found in this area for agricultural purposes. A brief summary of the study with generalized engineering interpretations is presented here.

The most important soil types in the valley are Newberg fine sandy loam, Barron coarse sandy loam, and Clawson sandy loam. Newberg fine sandy loam is the principal soil type in the floodplain and terraced areas of the valley. It occupies a strip along the Rogue River that is generally about a mile in width; however, it narrows to about 2,500 feet at Grants Pass. The soil is well-drained and presents no major problems for sewage collection and treatment.

Barron coarse sandy loam occupies extensive portions of the Rogue River Valley and underlies most of Grants Pass west of Gilbert Creek. The soil generally occurs upslope from Columbia fine sandy loam and extends as valley fill material into most of the minor tributary valleys. This soil has a slightly higher clay content than the Newberg loam.

Clawson sandy loam underlies a major portion of Grants Pass east of Gilbert Creek. Typically, this soil consists of about 1 foot of smooth-textured silt loam overlying a compact silty loam or clay loam subsoil. At a depth of about 30 inches, the subsoil assumes an extremely gritty texture, reflecting the presence of coarse granitic material. The subsoil terminates at shallow depths in coarse granitic rock. The soil is flat-lying and poorly drained, and because of the impervious nature of the shallow bedrock, it is waterlogged during the winter and spring months. In some areas, the water table stands at less than 3 feet below ground level well into the summer. The high groundwater conditions that accompany this soil type can be a problem when sewer pipes lying in the soil have cracks or leaks. Groundwater infiltrates into the cracks and leaks, significantly increasing the flow of liquid to the sewage treatment plant.

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Climate

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of sewerage facilities. Rainfall is especially significant, because it can cause large flow increases in sewage collection systems. For example, stormwater runoff may directly enter the sewers at manholes or through illicitly connected roof drains. Accumulated rainfall may raise groundwater levels in poorly drained areas, allowing entry of water into the sewer system through leaks or defective joints.

Other climatic factors can also affect sewerage planning. Biological treatment processes depend on air temperature to a large extent. Temperature, cloud cover, and the rate of evaporation are important factors to be considered in design of sludge lagoons.

General Climatic Conditions. Grants Pass is considered to have a mild climate, although temperatures below freezing and above 100 degrees F occur for short periods annually. The climate is influenced by air movement from the Pacific Ocean, located about 60 miles west of Grants Pass. However, the intervening coastal mountains modify the effect of the marine air masses, causing this portion of the Rogue River Valley to receive less annual rainfall and to have fewer cloudy and rainy days than most other portions of Western Oregon.

Precipitation. Nearly 75 percent of annual rainfall in Grants Pass occurs during the months of November through March (Figure 2-2). Most of the annual 32.3 inches of precipitation is in the form of rain, although about 4 to 5 inches of snow falls each year. Seasonal snowfalls have rarely exceeded 10 inches, and the snow usually melts almost as it falls. Daily, monthly, and annual precipitation for Grants Pass is summarized in Table 2-1. Precipitation during the last 2 years has been above the 60-year average (1937-1996), though it was below the average for the prior 10 years (Figure 2-3).

Temperature. Temperatures in Grants Pass usually remain moderate through the winter. Subfreezing temperatures may persist long enough to freeze water in aboveground facilities; however, they rarely last long enough to cause freezing in buried facilities.

Summers are warm and dry. Temperatures exceed 100 degrees F an average of 6 days a year. Nighttime summer temperatures are generally cool, averaging about 51 degrees F during July, the warmest month.

Grants Pass has an average growing season of 158 days. The growing season is defined as the period between the last 32 degree F temperature reading in the spring and the first 32 degree F temperature reading in the fall. Table 2-1 summarizes statistical temperature information for Grants Pass.

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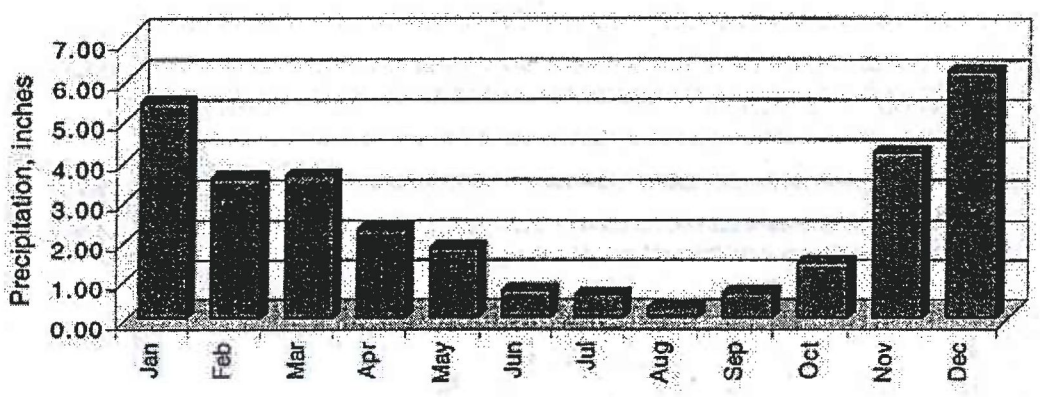

Figure 2-2. Monthly Average Precipitation

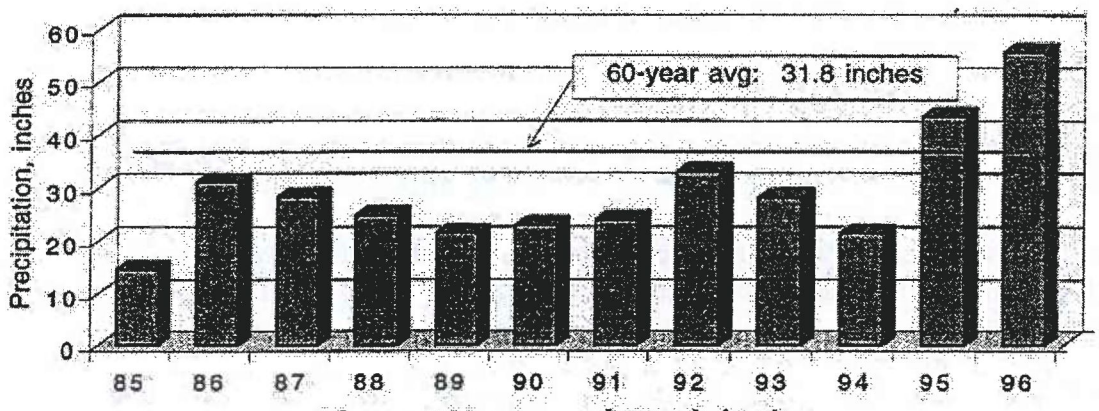

Figure 2-3. Annual Precipitation

Table 2-1. Climatic Summary for Grants Pass

Month	Temperature, °F					Precipitation, inches	
	Mean, maximum	Mean, minimum	Mean	Highest recorded[a]	Lowest recorded[a]	Mean	Greatest daily
January	45.1	31.1	39.6	69	2	5.4	3.4
February	50.4	38.6	44.1	76	5	4.0	4.3
March	56.8	44.4	48.2	84	18	3.4	2.3
April	59.1	46.8	53.1	94	25	1.9	2.0
May	65.1	52.6	59.2	102	27	1.4	1.6
June	70.8	59.4	65.1	108	33	0.7	1.5
July	75.5	63.4	71.0	110	39	0.3	1.5
August	76.5	65.6	70.2	110	36	0.3	0.8
September	69.2	59.4	64.6	108	28	0.8	2.9
October	62.2	50.1	55.0	98	20	2.3	5.3
November	50.1	37.9	45.0	74	12	4.5	3.2
December	46.3	34.1	40.2	67	-1	5.9	4.1
Annual	57.1	52.3	54.6	110	-1	30.8	5.3

Source: Records of the Oregon Climatological Service, 1928-1996.
[a] 1934-1990

Other Climatic Factors. Sunshine is usually abundant during the spring, summer, and fall, but the area is generally cloudy during the winter months. Early morning fog occurs frequently during November, December, and January. Fog is less common in October and February and rarely occurs during the rest of the year. Wind speed and direction are not routinely measured at Grants Pass. The prevailing wind direction, however, is from the west, approximately parallel to the axis of the Rogue River Valley.

Evaporation data are also not available for the study area. Because evaporation is an important climatological factor, evaporation data collected at the Medford Experimental Station was used. We believe these data approximate evaporative conditions in the study area because Medford is less than 30 miles away, has a similar mean annual temperature, and has an elevation only 250 feet higher. Table 2-2 presents a summary of Class A pan evaporation at Medford. The evaporation values applicable to large bodies of water such as lakes or ponds can be obtained by multiplying the measured values by a pan coefficient of 0.75. Applying this coefficient to

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the measured figure of 44 inches yields an average annual evaporation of 33 inches. Net evaporation, which is defined as the annual evaporation minus the annual rainfall, is very close to zero.

Table 2-2. Average Annual Evaporation

Month	Mean, inches	Percent of total
January	0.55	1.3
February	1.09	2.5
March	2.27	5.2
April	3.87	8.8
May	5.52	12.6
June	6.77	15.4
July	8.81	20.0
August	7.35	16.7
September	4.53	10.3
October	2.04	4.6
November	0.68	1.5
December	0.49	1.1
Annual	43.97	100.0

Note: Class A pan evaporation measured at Medford Experimental Station.

Source: U.S. Weather Bureau "Climatic Summary of the United States, Supplemental for 1951 through 1960, Oregon."

Water Resources

The principal water resources in the study area are surface water from the Rogue River and its tributaries and groundwater from the alluvium covering the river valley. The water resource most important for this study, the Rogue River, drains 2,460 square miles above Grants Pass before traversing the study area. The Rogue River is used for the city potable water supply, for irrigation, and for recreational use.

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A number of individual wells rely on the area's groundwater. The alluvium is the major aquifer with yields of 40 gallons per minute (gpm) being common. The volcanic formations usually yield less water, but in a few highly fractured areas wells have yielded 60 gpm. Many of the high yields are not sustainable, as the aquifers are small and substantial drawdown occurs.

Both the quantity and quality of water resources are important to sewage planning. A receiving stream must have adequate volume and assimilative capacity to accept discharge of treated sewage effluent. Also, the quality of the water must be examined to ensure that it remains safe for other uses. This section examines water quantities and water quality issues concerning the Rogue River and discusses the Grants Pass Irrigation District as it relates to sewerage planning.

Water Quantity. One use of the Rogue River is to receive treated effluent from wastewater generated in Grants Pass and other communities in the Rogue and Bear Creek valleys. The river must have the capacity to assimilate wastes without adverse effects on the quality of the river. The ability of a river to dilute and assimilate wastewater is highly dependent on its stream flow characteristics. Stream flow characteristics are described by measurements taken of stream discharge over a period of several years. The U.S. Geological Survey has collected stream flow data for a number of years.[5] The most recent 30 years of these data are summarized in Table 2-3.

The table shows that stream flows in the Rogue River can fluctuate widely from year to year. The largest discharge on record, 152,000 cubic feet per second (cfs), occurred during the December 1964 flood. The lowest recorded discharge was a minimum day of 606 cfs during 1968. Reservoirs have since been constructed in the Rogue River basin to provide storage of high wet weather flows for release during dry weather periods. With flow augmentation, this low flow has not occurred again, although as recently as 1995 a minimum daily flow of 744 cfs was recorded. The minimum monthly mean flow has also increased (Figure 2-4). The Rogue River Management Plan lists the minimum regulated river flow at Grants Pass as 1,200 cfs.

It is necessary to identify flood-prone areas, because any site considered for sewage treatment works must be evaluated for the risk of flooding. Section 5.3 of the *Grants Pass Comprehensive Plan for Community Development* describes flood history in the area, identifies flood-prone areas, evaluates the degree of hazard from flooding, and describes appropriate safeguards for the community. The following information is summarized from that document.

A flood insurance study was conducted for Grants Pass in 1979, and a similar study was performed for Josephine County in 1980. These studies defined the area that would be affected by a 100-year flood. Flood magnitudes are defined by the chance of occurrence of flood of a specific size: a 100-year flood has a 1 percent chance of occurring in a given year, and a 1-year flood has a 100 percent chance of occurring. The area affected by the 100-year flood is divided into the floodway and the floodway fringe. These areas are illustrated for the Grants Pass area on Figure 2-5.

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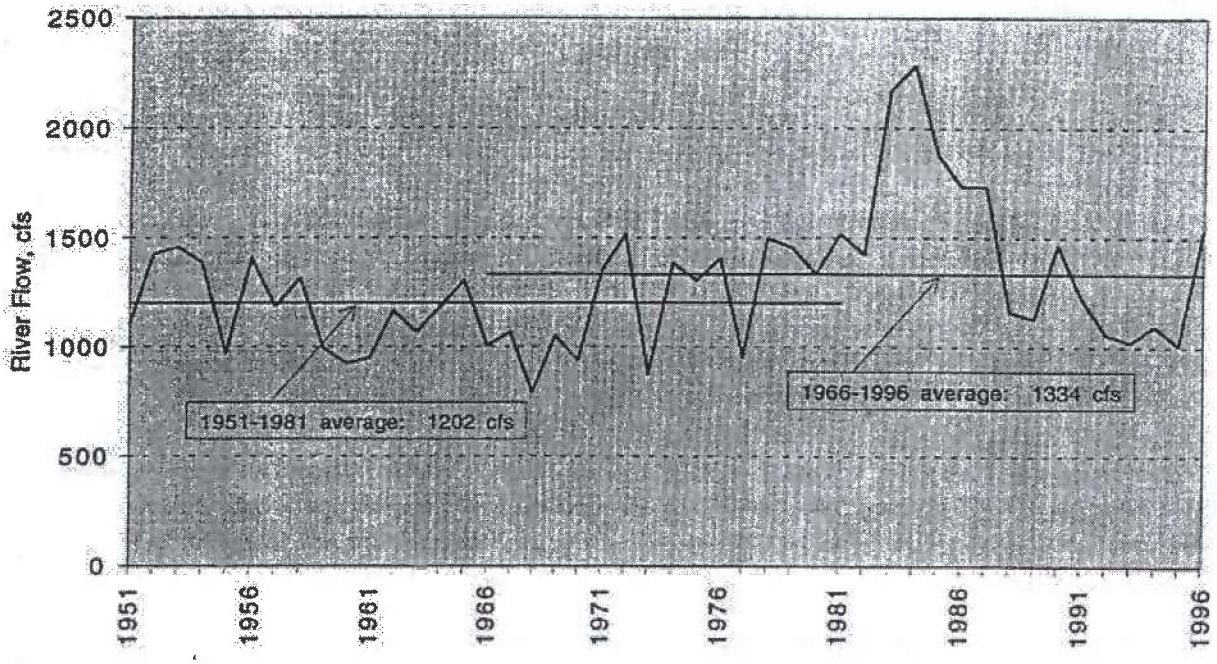

Figure 2-4. Rogue River Minumum Monthly Mean Flow

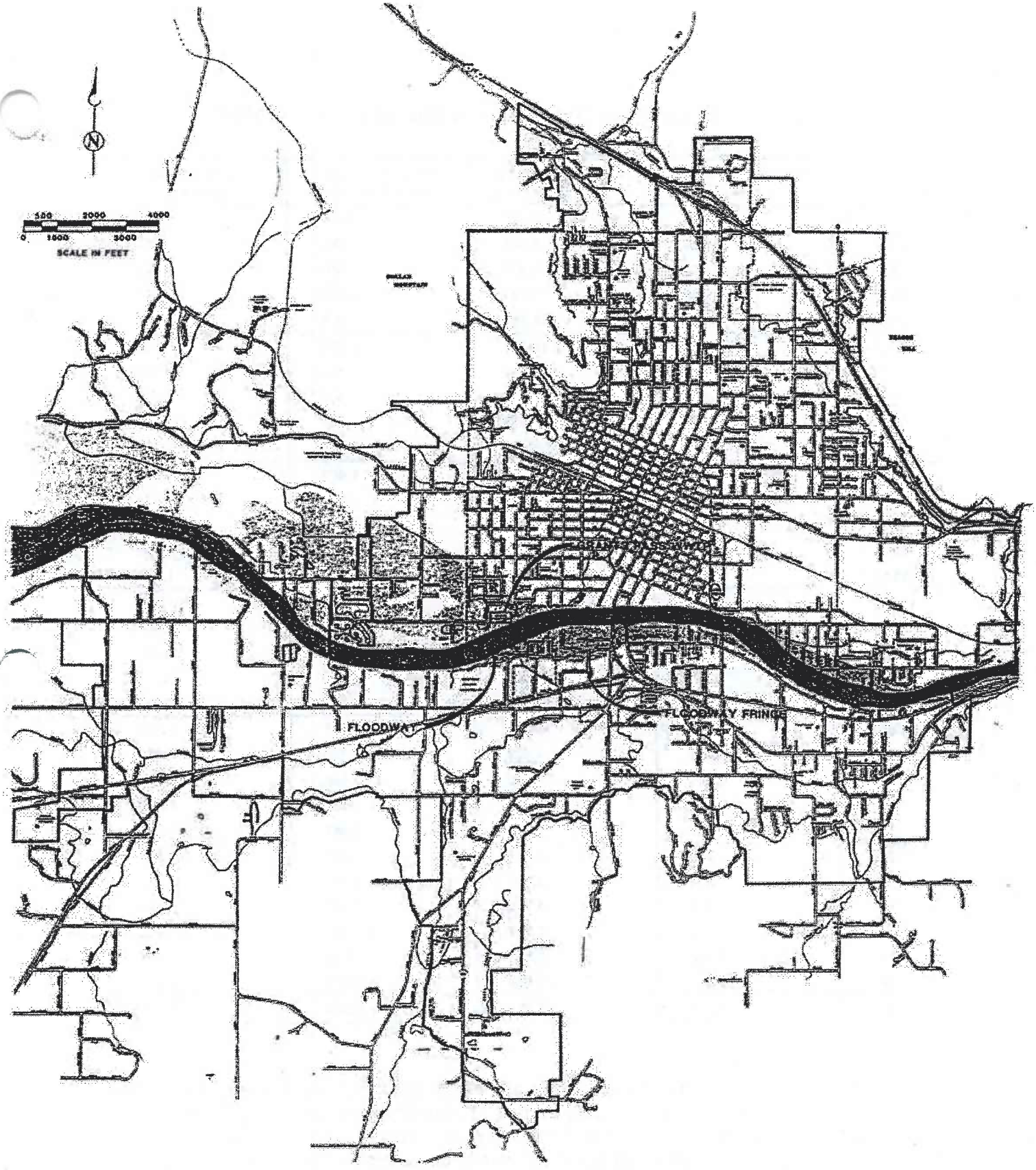

Figure 2-5. Rogue River Floodway

Table 2-3. Rogue River Stream Flow at Grants Pass (1966-1996)

Water Year	Yearly mean flow, cfs	Maximum monthly mean flow, cfs	Minimum monthly mean flow, cfs	Minimum daily flow, cfs
1966	2,901	7,795	1,004	796
1967	3,321	6,782	1,067	1,010
1968	2,015	6,479	791	606
1969	3,256	7,125	1,047	721
1970	3,428	13,360	939	900
1971	5,305	12,800	1,352	1,020
1972	5,208	10,410	1,514	1,370
1973	2,256	4,202	878	836
1974	6,269	15,170	1,384	1,120
1975	4,076	9,421	1,305	1,200
1976	4,440	6,449	1,405	1,200
1977	1,267	1,538	954	710
1978	3,233	7,369	1,497	1,380
1979	2,585	4,862	1,455	1,150
1980	3,037	7,754	1,333	1,070
1981	1,927	2,015	1,515	1,110
1982	5,047	14,030	1,423	1,190
1983	4,984	10,960	2,171	1,990
1984	5,276	12,550	2,282	1,570
1985	3,322	7,669	1,873	1,550
1986	3,288	9,543	1,731	1,470
1987	2,610	4,429	1,730	1,270
1988	1,894	3,340	1,160	906
1989	3,356	7,496	1,128	1,010
1990	2,059	2,795	1,467	1,220
1991	2,266	4,119	1,227	959
1992	1,538	2,406	1,059	941
1993	3,306	5,916	1,020	876
1994	1,573	2,549	1,094	808
1995	3,162	5,239	1,008	744
1996	4,874	10,630	1,530	1,240
Average	3,325	7,297	1,334	1,095

The boundary of a 100-year flood can change as land in the floodplain is developed. New structures divert the water, raising the level of the 100-year flood and increasing the amount of land that will be inundated by a flood of that magnitude. This is called encroachment. The National Flood Insurance Program has developed standards for development within floodplains. To assist cities in floodplain management, the program divides the area that would be inundated

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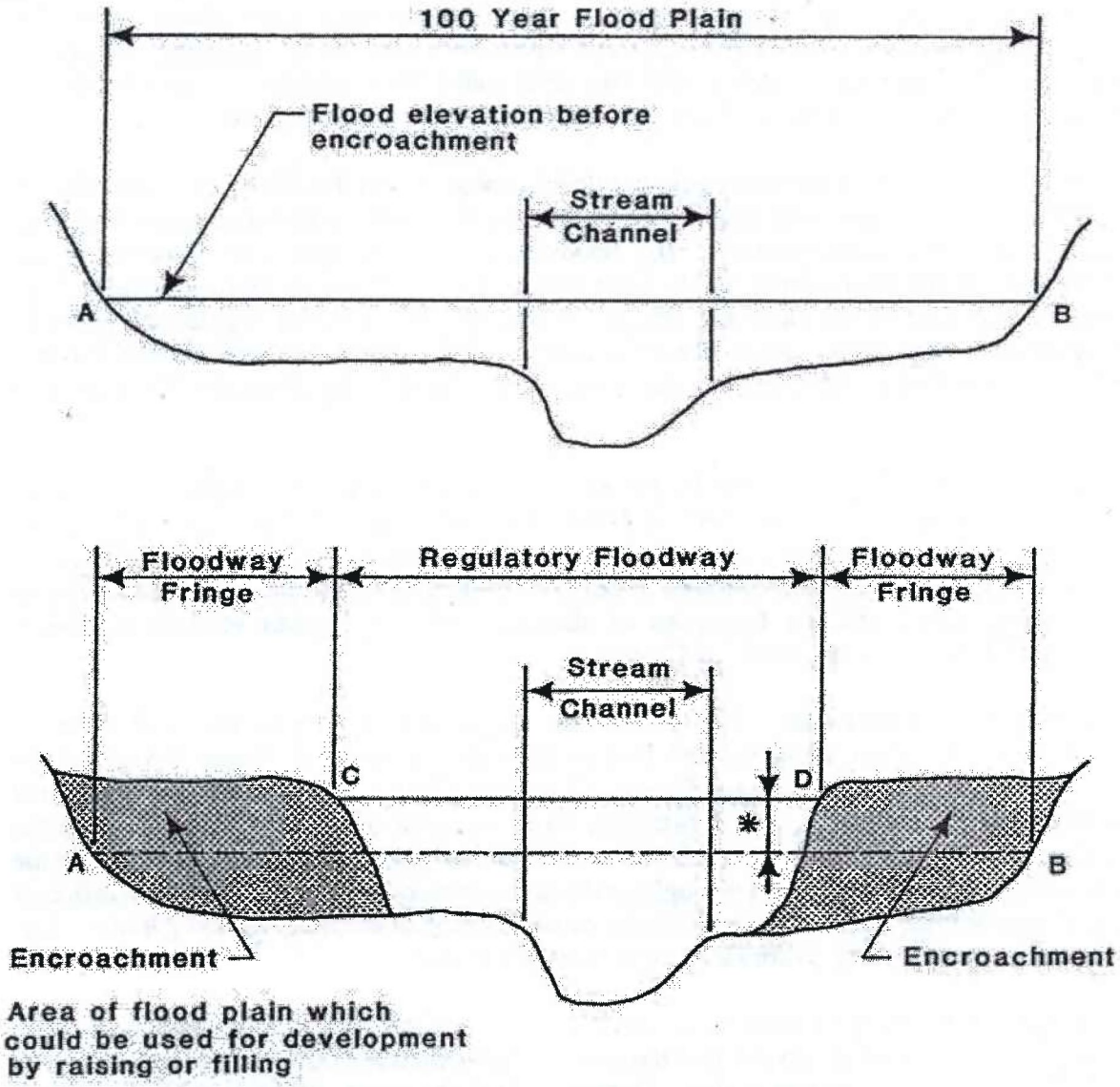

Line A–B is the flood elevation before encroachment
Line C–D is the flood elevation after encroachment

* Max. rise of 1.0 foot (N.F.I.P. regulations)

Figure 2-6. Flood Plain Schematic

by a 100-year flood into two segments: the floodway and the floodway fringe. Figure 2-6 illustrates the relationships between the floodway and the floodway fringe. The floodway includes the actual stream channel and any adjacent areas that must be kept free of encroachment (development) to keep from raising the level of the 100-year flood more than 1 foot. The floodway fringe includes the area between the defined floodway and the boundary of the 100-year flood. The floodway fringe is the area that could be completely obstructed without increasing the surface water elevation of the 100-year flood more than 1 foot.

The City has adopted an ordinance regulating development within the 100-year floodplain. As described in the Comprehensive Plan,[2] land within the floodway is not considered buildable. Regulations do allow development in the floodway if the developer can demonstrate that floodwater would not be diverted by the new structure in such a way as to adversely affect adjacent development in the floodway fringe. However, the stringent regulations generally preclude development within urban floodway areas. Development in the floodway fringe is permitted, but the first livable floor must be constructed at least 1 foot above the 100-year flood elevation.

Water Quality. Water quality in the Rogue River basin is protected by Oregon law. Oregon Administrative Rules, Division 41, Section 340-41-365, sets standards for water quality in the Rogue basin. The rules cover dissolved oxygen concentrations, temperature increases, pH values, coliform counts, creation of tastes or odors or toxic conditions that harm aquatic life or affect drinking water, and the formation of sludges. The regulations relating to sewage treatment and disposal are discussed in Chapter 6.

Grants Pass Irrigation District. The Grants Pass Irrigation District was formed in the early 1920s to supply irrigation water to land located between the town of Rogue River and the confluence of the Applegate and Rogue Rivers. Currently, about 7,700 acres of agricultural and residential lands are irrigated. The district has water rights to divert up to 150 cfs from the Rogue River during irrigation season. A series of canals constructed by the district carries the water from the Savage Rapids Dam throughout the area covered by the district. Many residents in the sewerage service area use water from the canals to irrigate landscaping and gardens. The canals are also used to carry stormwater away from these lands.

The irrigation season typically lasts from about April 15 to October 1. Examination of flows received at the treatment plant shows that the influent flows increase during those months, even though little rain falls during that period. It appears that one source of infiltration and inflow in the Grants Pass sewage treatment plant is irrigation water that has seeped into the ground and infiltrated the sewer system. Although there are currently discussions about removal of the Savage Rapids Dam, this report assumes that the Grants Pass Irrigation District will continue to function as it does now. Even if the dam were removed, pumps would likely be installed to deliver water to the irrigation district's canals.

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SOCIOECONOMIC ENVIRONMENT

The socioeconomic environment of an area can profoundly affect sewage facility planning. Residential, commercial, and industrial development patterns are determining factors in the location, design, and cost of trunk sewers, while land use and recreation patterns can influence site selection for sewage disposal. Historic and existing land use patterns can be used to make projections of future development patterns. These projections are then used to develop design criteria for sewage planning.

Many factors make up the socioeconomic environment of a community. This section examines those which are most important to sewerage planning: population and land use.

Population

Consideration of population trends is crucial to long-term sewerage planning. In order to size new facilities and expansions, historical population trends must be examined to predict future population growth.

The Grants Pass area has experienced steady population growth since the 1920s. This increase has been in line with the national population trend of people moving to the west and southwest from the northeastern and central states and to rural areas from urban areas. The population in Josephine County rose during the 1970s (5.11 percent annually), but slowed dramatically during the 1980s (0.6 percent annually). The City of Grants Pass grew more rapidly than the county during the last decade (1.5 percent annually).

Current Population. Population values for the area within the Grants Pass city limits are well documented. The 1990 census lists the population of Grants Pass at 17,424. Current population within the city limits is estimated at 20,526.

The population within the area currently served by sewers consists primarily of the population within the city limits and the population within the Harbeck-Fruitdale area. The City keeps records of the number of sewer connections for billing purposes. The sewer connections are separated into a number of categories, including residential hookups within the city limits and the number of hookups within the Harbeck-Fruitdale area. The population of Harbeck-Fruitdale can be estimated from the number of sewer hookups. The Josephine County Planning Department estimates that there are 2.3 people per residential sewer connection in Harbeck-Fruitdale. Therefore, it is assumed that 80 percent of the hookups within Harbeck-Fruitdale are residential, and the current population can be approximated at 4,200. Adding the Harbeck-Fruitdale population (4,200) to the city population of 20,526 gave an existing sewerage service area population of 24,700.

The future sewerage service area will include the existing service area, the Redwood area, and the North Valley area. The 1997 Redwood population is 4,758 and the number of North Valley residents is 1,200. Adding these values to the sewer service area population of 24,700 yields a population of 30,684. It is also necessary to take into account the commercial and industrial contribution as a form of a population equivalent. It is assumed that this value is 35 percent of the total residential population, which equals 10,739. Therefore, the total sewer service area is 41,423.

Population Projections. Based on the City's Comprehensive Plan, new population projections for the future sewerage service area have been developed. It is estimated that the Redwood area will have growth rates of 3.1 percent per year that will produce an estimated population of 9,600 by year 2020. The North Valley area is projected to grow at a 2 percent per year rate for a year 2020 population of 1,892.

The future sewerage service area population can therefore be approximated as the sum of the city limits, Harbeck-Fruitdale, Redwood, and North Valley populations, and the commercial/ industrial equivalent. Year 2020 service area population is estimated as 62,712. A summary of the population projections is shown in Table 2-4 and on Figure 2-7.

Table 2-4. Population Growth by Area

Area	1997 Population	2020 Population	Average growth rate, percent/year
City Limits	20,526	28,908	1.5
Harbeck-Fruitdale	4,200	6,053	1.6
Redwood	4,758	9,600	3.1
North Valley	1,200	1,892	2.0
Commercial/Industrial Equivalent	10,739	16,259	1.8
Future Service Area	41,423	62,712	1.8

The City and County have already established that pumping of Redwood's wastewater to Grants Pass WRP will begin in the fall of 2000. However, plans to provide sewer service to the North Valley area are less certain. Since the population and flow from North Valley are low compared to the contribution from the City (less than 4 percent), it was assumed that this flow would also be directed to the Grants Pass WRP within the planning period.

Land Use

The capacity of wastewater treatment facilities is highly dependent not only on the service area population, but also on the uses of the land. Land use is divided into three main categories: residential, commercial, and industrial. Each of these categories is subdivided based on density, type of use, and location.

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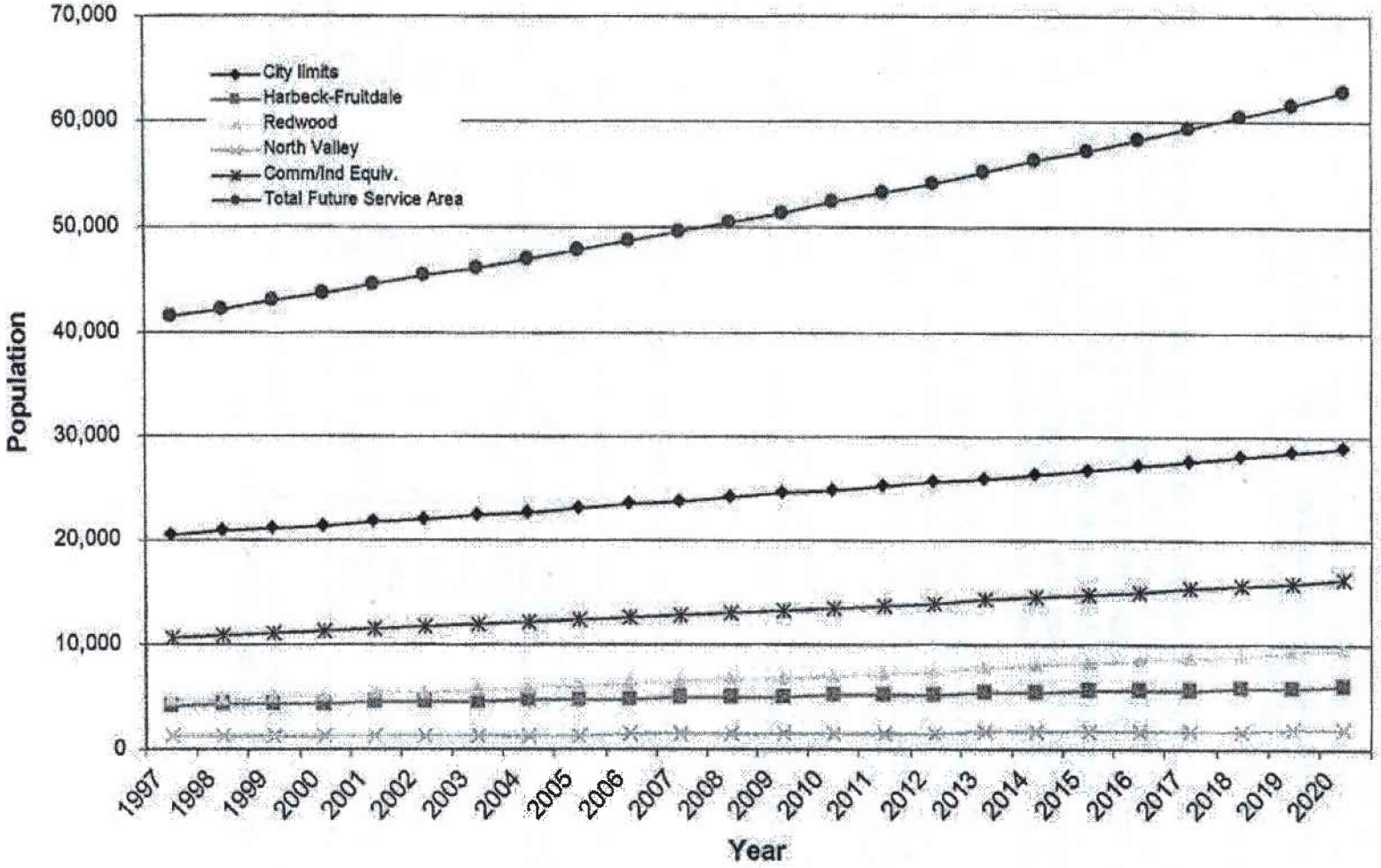

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Figure 2-7
Historic and Projected Grants Pass Population

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The City planning department keeps records on the amount of land in each zoning category within the city limits. The City assigns each land parcel a designation of improved, unimproved, or exempt. We assumed that only improved land is currently served by the wastewater treatment plant. The planning department estimates that there are approximately 2,912 acres of improved land within the city limits. The total amount of improved, unimproved, and exempt land within the city limits is 3,986 acres.

Because Harbeck-Fruitdale is part of the sewer service area, land use in these areas must also be considered. Land use records for Harbeck-Fruitdale were unavailable, so land use within these areas was estimated. We assumed that the Harbeck-Fruitdale area would be primarily residential with minor commercial and industrial land use. We also assumed that the Harbeck-Fruitdale area is developed to 40 percent of build-out capacity. Total land area within Harbeck-Fruitdale was determined from city maps and is estimated as 1,900 acres.

Future land use for each category was estimated as the sum of existing improved, unimproved, and exempt land for that category. While it is unlikely that all land within the service area will be developed by 2013, annexations of additional land into the service area should offset any remaining undeveloped land.

The *Sewerage Collection System Master Plan* contained land use estimates for 1980 and 2000. Our land use estimates are summarized and compared with the *Sewerage Collection System Master Plan* estimates in Table 2-5.

Table 2-5. Service Area Land Use Summary

Land use category	Area, acres			
	1980[a]	1990	2000[a]	2013
Residential	1,816	2,746	3,755	4,414
Commercial	362	591	743	993
Industrial	464	334	473	477
Public and semi-public	493	[b]	776	200[b]
Total	3,135	3,671	5,747	5,885

[a] Obtained from Sewerage Collection System Master Plan, James M. Montgomery, 1983.
[b] Public and semi-public land area included in other values.

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CHAPTER 3

EXISTING WASTEWATER TREATMENT PLANT

An understanding of the condition and performance of the existing wastewater treatment plant is essential in order to determine which unit processes can be incorporated into future facilities. In this chapter, each unit process is evaluated from an operational standpoint, the capacities are examined, performance records are analyzed, and deficiencies are discussed.

The City of Grants Pass has operated a wastewater treatment plant at the current plant site since 1935. The original plant consisted of a bar screen, a grit chamber, two rectangular sedimentation basins, an anaerobic digester, and a control building. In 1953 a second digester and a third sedimentation basin were added to the plant. The plant was upgraded to a secondary treatment facility in 1962. The modifications included an influent pumping station, a circular primary sedimentation basin, a trickling filter, and conversion of the existing rectangular sedimentation basins to secondary clarifiers. The next major plant improvement occurred in 1974. Plant capacity was increased and the secondary process was converted from a trickling filter to activated sludge. New treatment units included an influent pumping station, comminutors, aeration basins, two circular secondary clarifiers, a gravity sludge thickener, an anaerobic digester with a control building, a plant control building, and a water analysis laboratory. The trickling filter was converted to a chlorine contact basin and the existing influent pumping station was retained as a tank drain pumping station. The existing control building, rectangular secondary clarifiers, and the original digester were abandoned.

Several plant improvements were made during 1994 and 1995. These included installation of a gravity belt thickener (GBT) for waste activated sludge. This improved the performance of the existing gravity thickener by feeding it only primary sludge. A belt filter press (BFP) was also installed to dewater digested sludge. This allowed the plant to haul dewatered sludge for landfill disposal during wet weather when access for agricultural application was not possible. The BFP is a temporary installation until a long-term wet-weather solids handling program is implemented.

A more significant upgrade was completed in 1996. These improvements, listed below, were in response to the Department of Environmental Quality's (DEQ's) requirement to eliminate plant bypassing during high wet weather flows and to eliminate effluent toxicity associated with the chlorine residual.

- A fourth raw sewage pump.
- New headworks with mechanically cleaned bar screen.
- Two new, rectangular primary sedimentation basins.
- Effluent flow meter.
- Ultraviolet disinfection system.

The following sections discuss the design and performance of the current plant facilities.

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PLANT DESIGN

The overall plant layout is presented as Figure 3-1. A plant schematic and hydraulic profile are illustrated in Figures 3-2 and 3-3. The plant design data is summarized in Table 3-1.

The plant currently has firm raw sewage pumping capacity, screening, and primary treatment capacity for a peak flow of 27 million gallons per day (mgd). The secondary treatment process has a peak hydraulic capacity of about 13 mgd. All primary effluent flow in excess of 13 mgd is conveyed directly to final disinfection. The primary effluent flow to disinfection is controlled by a motorized valve modulated to control the maximum level in the secondary clarifier effluent troughs. Because of hydraulic limitations of the recently installed UV disinfection equipment, peak flow through disinfection is limited to about 23 mgd.

Raw wastewater flows to the plant site through three influent sewers, combines in the influent junction structure, and flows into the influent pumping station wet well. The plant is equipped with a raw sewage bypass weir which will allow emergency flows to pass directly to the outfall. Emergency flows correspond to storm conditions that cause sewer flows to exceed the plant's hydraulic capacity to treat the once-in-5-year peak flow of 27 mgd. The influent pumping station houses four raw wastewater pumps, three with a capacity of 9 mgd and one which can pump up to 12 mgd. Each pump is equipped with a variable-speed drive unit. The wastewater passes through a mechanical bar screen which has a peak capacity of 23.5 mgd. A bypass channel with a manually-raked bar screen serves as a back up to the mechanical bar screen.

After screening, the raw sewage flow is split between the two new 108-foot by 21.5-foot rectangular primary clarifiers and the 70-foot-diameter circular primary. The majority of the time, when wastewater flows are low, only the rectangular primaries are in service. When flow exceeds the 16.4-mgd capacity of the rectangular units, a motorized gate opens and automatically places the circular primary into service. Sludge from all primaries is pumped continuously to degritting cyclones and then directed to a 30-foot-diameter gravity thickener. Thickener overflow is returned to the tank drain pumping station and pumped to the headworks. The thickened primary sludge is pumped to the anaerobic digester.

Primary effluent flows into a 112,500-cubic-foot aeration basin for secondary treatment. The aeration basin is separated into six bays by baffle walls. The configuration of the return sludge piping, feed channel, and aeration bays allows for great flexibility in process operation. The system can be operated in plug flow, step feed, or contact stabilization mode. Activated sludge is returned to the aeration basin by two variable-speed pumps, each having a capacity of 3.9 mgd. Three 2,500-cubic-feet-per-minute centrifugal blowers provide aeration air to coarse bubble diffusers.

Mixed liquor flows from the aeration basin into two 75-foot-diameter, 12-foot-deep secondary clarifiers. Each clarifier is equipped with a suction-sweep sludge withdrawal mechanism and an 18-foot-diameter inlet skirt.

Secondary effluent is then directed to the ultraviolet light (UV) disinfection system. The UV system consists of two lamp banks mounted in a single channel. Each bank contains 48 medium pressure lamps. A portion of the disinfected effluent is reclaimed for plant utility water and site irrigation. Hypochlorite is fed to the reclaimed water system to control slime growth in the piping system. Disinfected effluent is then discharged through a 42-inch-diameter shoreline outfall to the Rogue River.

Waste activated sludge (WAS) is pumped from the return activated sludge (RAS) sump to a 1.5-meter GBT. Thickener filtrate is returned to the headworks. The thickened WAS is pumped to a single 59,000-cubic-foot anaerobic digester. The digester is equipped with a floating cover and gas mixing system. Digester gas is burned in the plant boiler for sludge heating and space heating. Digested sludge is transferred from the anaerobic digester to a 12,500-cubic foot holding tank (known as the No. 2 digester). It is then loaded into a 3,400-gallon truck and hauled for agricultural land application. Because land application of sludge is prohibited when there is standing water or groundwater within 4 feet of the surface, a belt filter press is available to dewater sludge for landfill disposal. This prevents solids from building up in the plant during wet weather. The belt filter press was used a total of 27 days in 1996. The 1.2-meter press is a temporary, trailer-mounted installation to be used only when needed until a long-term solids handling system is implemented.

A 900-kilowatt diesel-fueled gas turbine-generator provides standby power. The generator has adequate capacity to satisfy the entire plant's electrical requirements.

The 1996 plant improvements included a computer-based supervisory control and data acquisition system (SCADA). Currently, this system only controls the new facilities constructed in 1996. The long-term plan is to phase out the old hard-wired control panels and move all control to the SCADA system as plant improvements are constructed.

PLANT OPERATION

The treatment plant operates under the authority of a National Pollutant Discharge Elimination System permit issued by the Oregon DEQ. The existing permit expired in September 1997, and the plant currently operates under a provisional extension of that permit while a new permit is considered. The permit discharge requirements are summarized in Table 3-2.

Daily plant effluent suspended solids (SS) concentrations for January 1995 through July 1997 are shown in Figure 3-4. The high effluent SS values are usually caused by either solids washout at high flows or the bulking of filamentous solids. Figure 3-5 indicates that high effluent biochemical oxygen demand (BOD) concentrations also occur on occasion. The high BOD values often coincide with high SS concentrations, indicating that much of the BOD is associated with the solids.

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Table 3-2. Current Discharge Permit Requirements

Parameter	Average effluent concentration, mg/L		Mass discharge, lb/day		
	Monthly	Weekly	Monthly average	Weekly average	Daily maximum
May 1 - October 31					
BOD	20	30	670[c]	1,000[c]	1,300[c]
TSS	20	30	670[c]	1,000[c]	1,300[c]
Fecal coliform	200[a]	400[a]	[b]	[b]	[b]
November 1 - April 30					
BOD	30	45	1,600[d]	2,400[d]	3,200[d]
TSS	30	45	1,600[d]	2,400[d]	3,200[d]
Fecal coliform	200[a]	400[a]	[b]	[b]	[b]

Other parameters:
1. pH 6.0 - 9.0
2. BOD and TSS removal efficiency shall not be less than 85 percent on a monthly average basis.
3. The mixing zone shall not exceed the portion of the Rogue River from 10 feet upstream to 300 feet downstream of the point of discharge. The zone of immediate dilution shall not exceed 10 percent of the defined mixing zone.
4. Total chlorine residual concentrations shall not exceed 0.048 mg/L as a daily maximum.

[a] Number per 100 mL.
[b] Not applicable.
[c] Based on an ADWF of 4 mgd.
[d] Based on an AWWF of 6.4 mgd.

Historically, raw wastewater bypasses due to hydraulic overloading occurred as many as 12 times in a year. Upgrades to the headworks and primary sedimentation units brought on-line in August 1996 have increased the plant's hydraulic capacity substantially, so that the treatment plant can now provide primary sedimentation for up to 27 mgd. Since the upgrade, raw wastewater has been bypassed only during two very high-flow events during the area-wide flood events of December 1996 and January 1997.

The secondary treatment system has a lower hydraulic capacity of 13 mgd, so on a more frequent basis the primary effluent control valve opens to allow primary effluent to bypass the secondary treatment system. This occurred for 3 days in November 1996, 16 days in December 1996, and 4 days in January 1997. Bypassed primary effluent is combined with the secondary effluent and disinfected before the plant effluent water quality sampling point, so its contribution is included in records of the plant effluent quality.

LABORATORY AND PERSONNEL FACILITIES

The condition of the working environment plays a vital role in successful wastewater treatment plant operation. The primary areas of concern are the operator facilities, laboratory, maintenance shop, and office space.

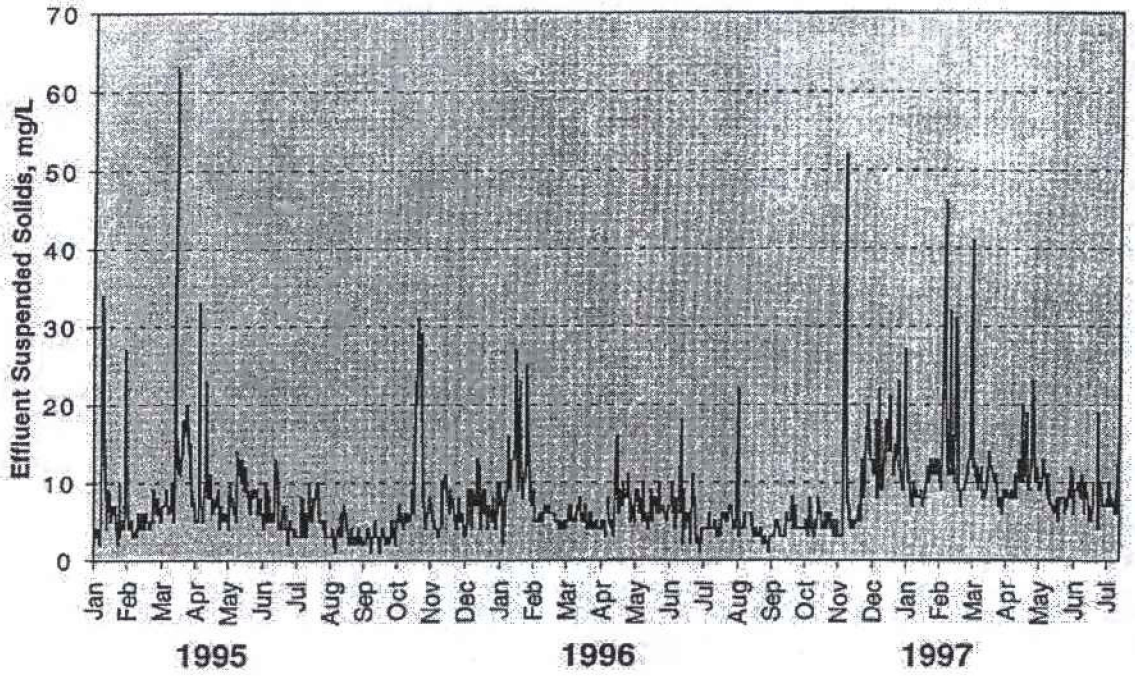

Figure 3-4. Grants Pass Plant Effluent Suspended Solids

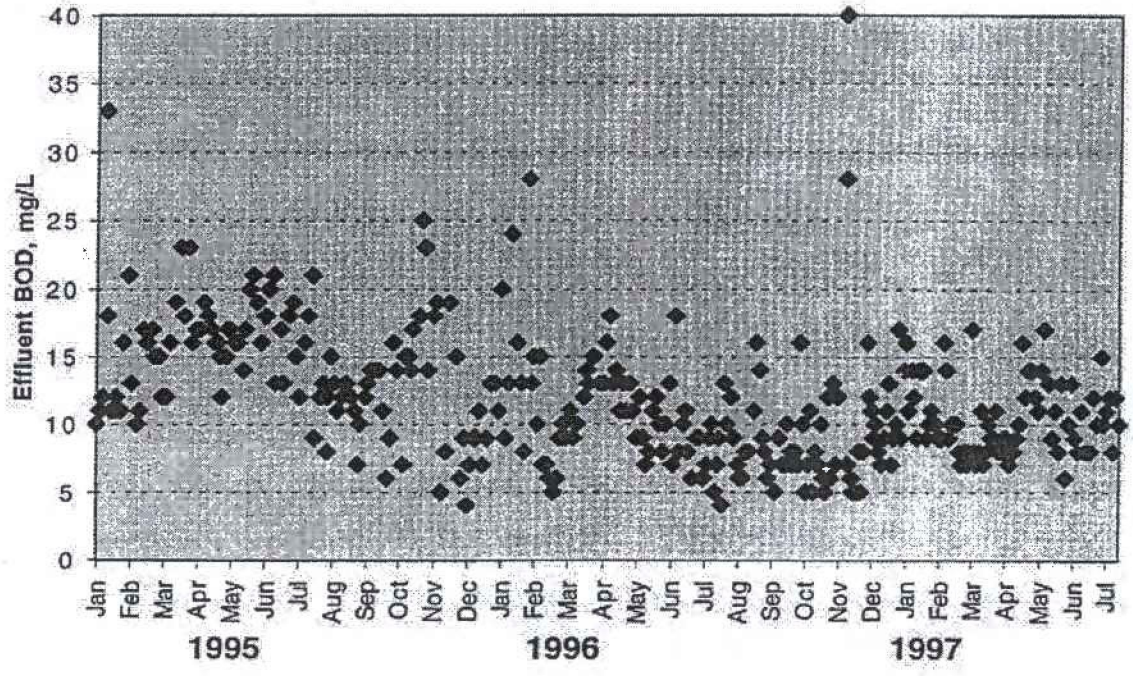

Figure 3-5. Grants Pass Plant Effluent BOD

Operator Facilities

The operations building includes a men's washroom with locker area and a women's washroom. The 1975 operation and maintenance manual recommended a staff of 11 for the plant. The men's locker room is sized to accommodate a staff of 11 if consideration is given for two lockers per employee. One locker would be used for clean clothes and a second locker would be used for soiled clothes and boots. The facilities are sufficient for existing staff and up to three temporary male employees. The existing facilities comply with the requirements of Oregon Administrative Rule 437-112 which outline regulations for water and sanitation.

The women's washroom consists of two lockers, a rest area, a toilet, and sink. Private shower facilities are available.

A day room is provided for breaks, lunches, and group meetings. The day room can accommodate staff of about 12 persons.

Laboratory

The laboratory consists of an analytical area, a sample room, and a storage area. The area east of the laboratory is used for administrative work.

Future laboratory needs depend upon the numbers and types of analyses that will be performed. For routine analyses of biochemical oxygen demand, suspended solids, nutrients, and other process tests, the existing laboratory is adequate. If the city chooses to perform more sophisticated analyses such as atomic absorption spectroscopy or gas chromatography, additional analytical work space will be required. The administrative area east of the laboratory could easily be converted to an analytical work area.

A significant limitation in the existing laboratory is fume hood space. The fume hood is used for exhausting fumes from the laboratory to the outside. Various analyses, including chemical oxygen demand and total Kjeldahl nitrogen are performed under the fume hood. The fume hood work area should be increased for safety and convenience reasons. The laboratory counter tops have also deteriorated and need to be replaced.

Maintenance Shop

Two areas of the plant are used for maintenance of instrumentation and equipment. Major equipment maintenance is performed in a large garage/workshop on the south side of the operations building. A small instrument work room is located behind the east wall of the MCC room.

Office Space

The original design included two offices: the director's office and the lab office. These offices have been combined into one large work area. The control room is also used as a work area for the operations staff.

Office space expansion is necessary; the control room could be subdivided, or part of the porch area east of the control room could be enclosed. The control room is approximately 450 square feet, while the porch is about 400 square feet. Removal of the chlorination equipment has also provided space suitable for conversion to office space.

INDUSTRIAL PRETREATMENT PROGRAM

In 1996, the City of Grants Pass adopted an industrial pretreatment program to meet the Federal requirement outlined in 40 CFR 403, and to protect the Grants Pass Water Restoration Plant (WRP) from harmful industrial discharges. The new Sewer Use Ordinance was adopted as Chapter 8.50 of the Grants Pass Municipal Code. Approved by the Oregon Department of Environmental Quality in 1996, the ordinance is included as Appendix F of this Facilities Plan.

Under the new ordinance, significant industrial sewer users (SIUs) are required to meet the national categorical pretreatment standards (40 CFR Chapter 1, Subchapter N, Parts 405-471) specific to each user's industry. Certain discharges (e.g., hazardous materials, foaming agents) which could cause damage to the WRP or interfere with WRP operation are prohibited. The ordinance clearly outlines procedures for permit application, permit issuance, reporting requirements for permittees, compliance monitoring, public information, and enforcement.

The four SIUs receiving permits under the new plan are described in Table 3-3. One electroplating and one metalfinishing facility are strictly regulated in heavy metals and pH discharge, while one industrial discharger has been identified as a potential source of oils, grease, and glues.

Table 3-3. The City of Grants Pass Significant Industrial Users

Business	Industrial category	Estimated flow, gpm	Potential pollutants
Wayne Printed Circuits	Electroplating	506 gpd	Metals-pH
Swissmetric	Metalfinishing	3,343 gpd	Metals-pH
U.S. Forest Industries	None	117,978 gpd	Oils, grease, and glues
Clear Water CO-OP LCC	None	17,060 gpd	BOD, TSS, FOG, pH

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Clear Water Co-op, a licensed septage hauler, is a potential source of fats, oils, greases, BOD, total suspended solids, and extreme pH. This new private company began accepting septage from many local sources in August 1997. Their on-site treatment process dewaters septage sludge. The company hauls the sludge to the landfill, and discharges the liquid filtrate to the WRP sewage collection system. During the first month of operation, it added approximately 13.7 pounds per day (ppd) of BOD to the WRP. This load may increase significantly in the future as the company's operations grow. To protect the treatment plant from extreme load events ("slugs") which can overwhelm and severely damage its biological treatment system, this new, high-strength waste source should be carefully monitored to observe its effects (if any) on the plant. If slug loads do occur, the city's industrial discharge ordinance would require dischargers to provide equalization to blend slugs with wastewater flow. Likewise, if the BOD load leads to adverse effects on the WRP's biological treatment system pretreatment may be considered.

UNIT PROCESS PERFORMANCE AND CONDITION

Plant operating reports for August 1996 through July 1997 were examined to determine the present loading and operating performance for individual treatment unit processes. Loading rates for the primary sedimentation basins, aeration basin, secondary clarifiers, gravity thickeners, and No. 1 anaerobic digester are summarized in Table 3-4. This section discusses the capacity, expansion capability, performance limitations, and operating and maintenance problems of each unit process.

Influent Pumping Station

The capacity of the influent pumping station is 27 mgd, assuming one pump is used for standby. Pumping station capacity can be expanded by replacing one or more existing pumps with pumps of greater capacity.

The new influent pump runs well without clogging. When the older influent pumps are called into service at higher flows, they often clog with rags lodging between the impeller and the wear ring. Plant maintenance staff must unbolt the pump backing plate and raise the impeller to provide necessary clearance to remove the rags.

The mechanical bar screen and screenings compactor have been operating satisfactorily since August 1996.

Wastewater composition is monitored with a flow proportional composite sampler located just downstream of the influent pumps. The sampler was installed in 1994.

Table 3-4. Treatment Process Loading and Performance

Unit Process	Design Value	Current Value
Primary sedimentation		
Hydraulic loading, gallons per day per square foot[a]		
Average dry weather (Aug-Sep 1996 and Jun-Jul 1997)	--	990
Average wet weather (Oct 1996-May 1997)	--	730
Peak wet weather[b]	2750[a]/3500[b]	3,180
BOD removal, percent		
Average	30	41
During maximum flow month[c]	15[c]	36
SS removal, percent (average)		
Average	60	54
During maximum flow month[c]	30[c]	53
Aeration basins		
Organic loading, lb BOD per day per 1,000 cubic feet[d]	49.5	32
Mean cell residence time, days		
Average	5	4.7
Observed range of monthly average[e]	--	2.9-7.9
Sludge yield, pounds SS produced per pound BOD removed	0.7	0.47
Mixed liquor suspended solids, average, mg/L	2,500	1,500
Secondary clarification		
Hydraulic loading, gallons per day per square foot		
Average	--	645
Peak wet weather[f]	1,470	1,585
Return sludge concentration, average, mg/L	5,000	5,130
Return sludge flow, average, mgd	--	2.3
Gravity thickening of primary sludge		
Solids loading, ppd per square foot	6	4.9
Thickened sludge concentration, percent	5	4.5
Thickening of secondary sludge (WAS)		
Solids loading, (lb/hr, if run 7 hr/d)	700	530
Thickened sludge concentration, percent	5	5.7
Anaerobic digestion		
Volatile solids loading, ppd per cubic foot	--	0.095
Volatile solids loading, ppd	--	5,605
Detention time, days	20	28
Volatile solids reduction, percent	55	52
Sludge disposal, dry ton/yr	--	606

This table calculated from 1 year's data, August 1996 through July 1997.

[a] October 1996 through May 1997, both rectangular basins and the circular basin are in use. During the rest of the year, only the rectangular basins were used.

[b] Limited to 27 mgd. Circular/rectangular primary loading.

[c] December 1996. Design performance at peak day flow.

[d] Aeration basin modes ignored—calculated assuming plug flow.

[e] During 12-month study period August 1996 through July 1997.

[f] Limited to 14 mgd.

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Primary Sedimentation Basin

The rectangular primary sedimentation basins have an average hydraulic loading rate of 990 gallons per day per square foot (gpd/sq ft) during the dry weather months (June through September). This is well below the maximum capacity (peak wet weather) loading rate of 3,500 gpd/sq ft. During wet weather (October through May), when flows are high and the circular primary sedimentation basin is in service, the hydraulic loading rate on the three basins averages 730 gpd/sq ft, also well below the circular basin's maximum capacity (peak wet weather) loading rate of 2,750 gpd/sq ft. BOD and SS removal in the primary sedimentation basins average 41 percent and 54 percent respectively. Because the overflow from the gravity thickeners is not included in determining the applied load, the actual BOD and SS removals are somewhat greater than the values shown above. Examination of the plant records reveals that even during December 1996, the maximum flow month of the study period, the basins removed an impressive 36 percent of BOD and 53 percent of SS flowing to them.

The primary sludge is pumped continuously through a cyclone degritting system. This equipment was also installed as part of the 1996 plant improvements. Grit separation efficiency has improved and quantities of grit collected have increased significantly. After degritting, the primary sludge is directed to the gravity thickener.

Secondary System Bypass

A motorized valve was added in 1996 to limit flow through the secondary treatment system. The two circular secondary clarifiers were designed for a flow of up to 13 mgd, and solids washout has historically occurred when flows have increased above that level. An automatic level sensor now monitors the depth of flow in the secondary clarifier effluent launder. During high flow events, when the level reaches a maximum desired launder depth, the primary effluent control valve automatically opens. This diverts primary effluent directly to disinfection. When flows decrease the launder level drops and the control valve automatically closes, forcing all primary effluent to the secondary treatment system. This approach assures that the secondary treatment system operates at its maximum hydraulic capacity before primary effluent is diverted to disinfection.

Aeration Basin

The aeration basin receives an average BOD load of about 3,590 ppd. The secondary process has been operated with an average 4.7-day mean cell residence time and an observed sludge yield of 0.47 pounds SS produced per pound BOD removed. These values are about average for a typical activated sludge process. The average mixed liquor suspended solids concentration of 1,500 milligrams per liter (mg/L) is a bit low, but within the range of expected values.

Basin Operational Flexibility. The aeration basin is divided into six bays, and feed channels run along three perimeter walls. The design of the aeration basin and return sludge piping allows for great flexibility in the operation of the secondary process. The basin can be operated in any of the most common activated sludge modes: plug flow, step feed, contact stabilization, and reduced-aeration plug flow. Selection of the proper operational mode can help prevent nitrification, growth of filamentous bacteria, and solids washout.

Plug flow mode is illustrated in Figure 3-6. Primary effluent (PE) and RAS are mixed prior to entering the feed channel. The mixture of PE and RAS, or mixed liquor, flows through the six bays and to the secondary clarifiers. The aeration basin is typically operated in plug flow mode during dry weather.

Step feed operation is shown in Figure 3-7. RAS flows into Bay 1, while PE is discharged into the feed channels. The feed channels distribute PE into all six bays. Step feed operation increases solids storage capacity, thereby producing longer sludge ages than plug flow mode. The process may be varied by discharging PE into different combinations of bays.

Contact stabilization mode is shown in Figure 3-8. RAS flows into Bay 1, and PE is discharged into Bay 3. Bays 1 and 2 contain RAS, while Bays 3A, 3B, 4, and 5 contain mixed liquor. Sludge age can be adjusted by discharging PE into different bays. Sludge age is maximized by discharging PE into Bay 5. Aeration basin operation can be shifted to contact stabilization mode during high flow events to provide solids storage and help prevent solids washout.

Reduced-aeration plug flow mode is shown in Figure 3-9. PE flows into Bay 1. RAS is combined with PE in Bay 3A and the mixed liquor is aerated in Bays 3B, 4, and 5. Reduced-aeration plug flow decreases the solids inventory, sludge age, and aeration basin treatment capacity. Little treatment occurs in Bays 1 and 2 because PE does not contain a sufficient population of microorganisms to consume large quantities of organic material.

Operational Problems. In previous years, the secondary treatment process was plagued with filamentous sludge and foam forming excessive scum layers on the secondary clarifiers. Recent upgrades of the handling facilities have provided the plant staff better control of the solids inventory leading to improved secondary process performance and effluent quality. The shorter sludge age helps to reduce nitrification and prevent growth of filamentous bacteria.

Secondary Clarifiers

The secondary clarifiers have an average hydraulic overflow rate of 645 gpd/sq ft. The overflow rate at the design flow of 13 mgd is 1,470 gpd/sq ft. Secondary clarification capacity can be increased by adding new clarifiers.

Both of the clarifiers currently leak in the centerwell seal. This leakage allows mixed liquor to dilute the more concentrated RAS. This, in turn, dilutes the waste activated sludge concentration and places additional hydraulic load on the gravity belt thickener. The RAS solids concentration averages about 5,130 mg/L. Return sludge flow averages 2.3 mgd.

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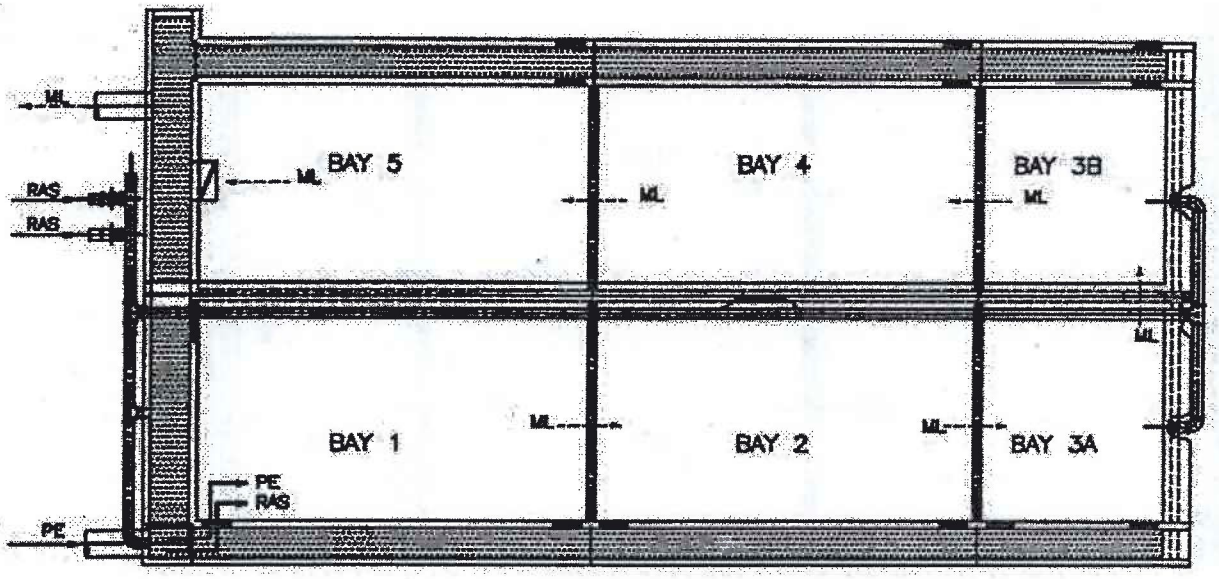


Figure 3-6. Plug Flow mode

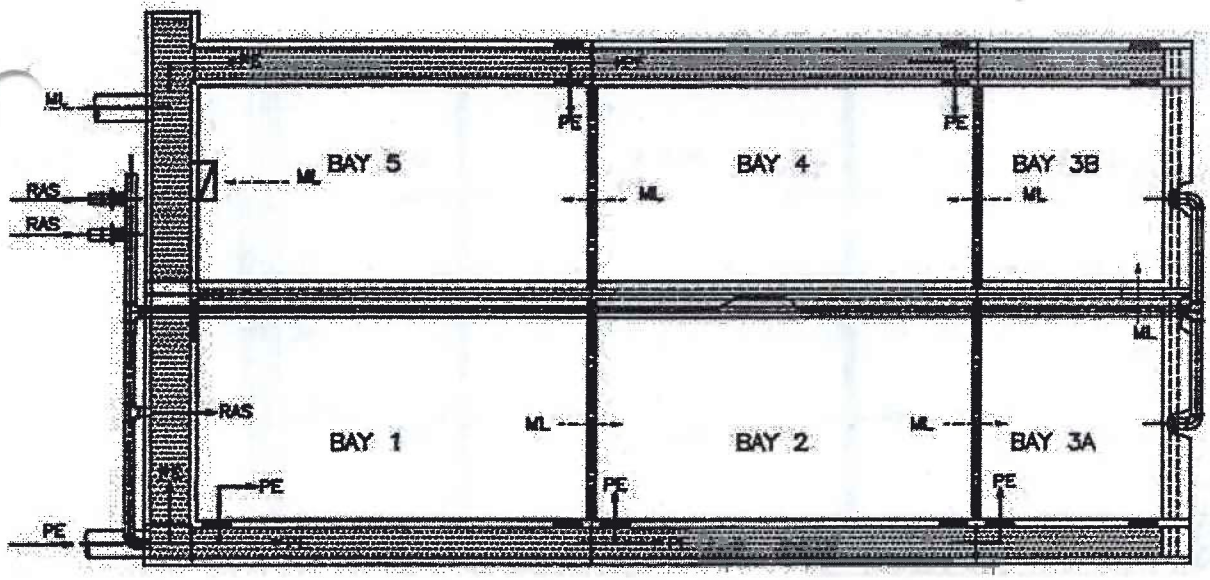
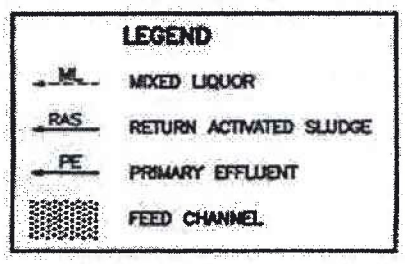


Figure 3-7. Step Feed Mode



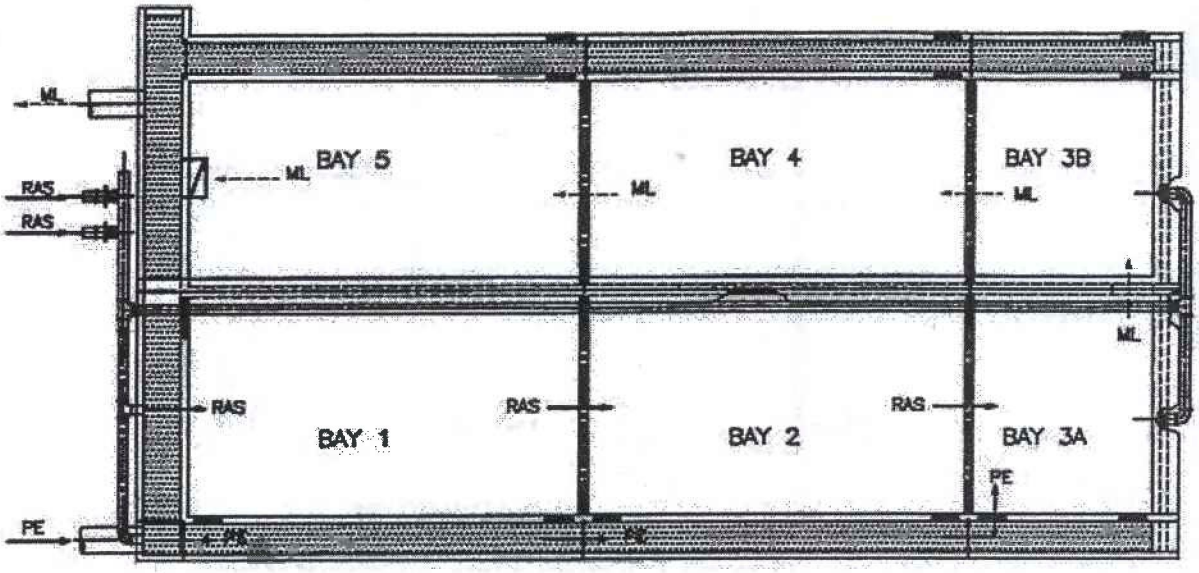


Figure 3-8. Contact Stabilization Mode

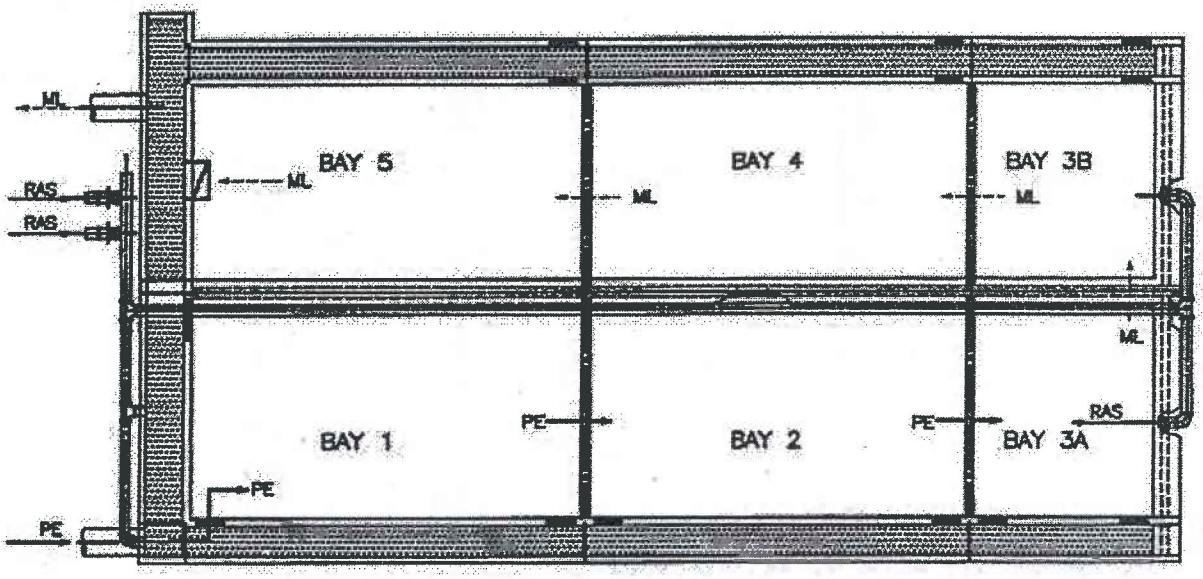
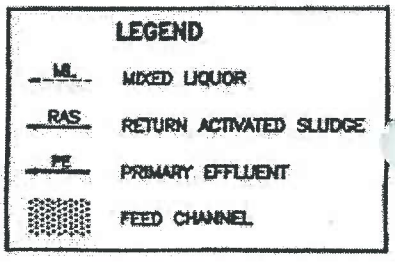


Figure 3-9. Reduced-Aeration Plug Flow Mode



Disinfection

The chlorine based disinfection system was replaced with a UV system that started operation in August 1996. All of the chlorination equipment has been removed from the plant. The UV system uses medium pressure lamps that are equipped with an automatic cleaning system. This was one of the first installations of this type of UV equipment in the northwest. Numerous electrical and control problems were encountered during the first year of operation. The equipment manufacturer has replaced the failed components and the plant's disinfection requirement has been continuously met.

The UV system was designed for a peak hydraulic capacity of 27 mgd. However, headloss measurements indicated that the equipment could only pass about 20 mgd. Recently, the manufacturer modified the equipment to reduce headloss and increase peak hydraulic capacity. Analysis of recent headloss measurements indicate a peak hydraulic capacity of 23 mgd.

Instrumentation

The 1996 plant improvements included a computerized supervisory control and data acquisition system, SCADA. It is intended that this system be expanded as future plant improvements are made and, eventually, the old control system will be replaced.

Gravity Thickener

The gravity thickener receives approximately 3,450 ppd of primary sludge. The current load of 4.9 ppd per square foot is 42 percent of the design solids loading capacity. The thickener typically produces sludge with a concentration of 4.5 percent, which compares well to textbook values. During the day when the plant is staffed, thickened sludge is pumped from the gravity thickener to the No. 1 anaerobic digester.

Gravity Belt Thickener

The gravity belt thickener receives approximately 3,700 ppd of secondary sludge. The current load of 530 pounds per hour is 76 percent of the design solids loading capacity. The thickener typically produces sludge with a concentration of 5.7 percent, which compares well to textbook values. Polymers are added to the sludge to enhance thickener performance. During the day when the plant is staffed, thickened sludge is pumped from the gravity belt thickener to the No. 1 anaerobic digester.

Anaerobic Digesters

The No. 1 digester volatile suspended solids loading rate is approximately 0.095 ppd per cubic foot. The average detention time, based on the volume of sludge pumped to the digester, is

28 days. The No. 1 digester is currently operating at 84 percent of the design solids loading capacity. The addition of the gravity belt thickener for WAS has increased the overall solids concentration within the digester and made more efficient use of the available volume. The solids concentration inside the digester currently averages 2.2 percent. Digester performance is within typical ranges with volatile solids destruction averaging 50 percent.

The digester needs cleaning, thorough inspection, and rehabilitation. It has not been cleaned since it was placed into service in 1975. Significant rag and grit accumulation has very likely reduced the useful volume. Inspection of the cover has shown corrosion around the gas dome and rainwater is leaking thought the cover roofing material. Significant corrosion was observed when the gas mixing compressor was replaced in 1995. The digester will need to be taken out of service for cleaning. At that time the cover can be inspected, sand blasted and recoated, roofing replaced, and corroded gas piping replaced.

Digester 2 is primarily used for sludge storage. It had a significant accumulation of rags and grit when it was last cleaned in 1993.

Belt Filter Press

During rainy periods when land application of biosolids is not feasible, storage of solids within the treatment process created operational problems leading to potential plant upsets. To prevent this situation, the city purchased a used trailer-mounted 1.2-meter BFP. This equipment allows digested biosolids to be dewatered and hauled to the landfill. Although the BFP is a temporary installation, the plant now has the ability to handle biosolids during all weather conditions. This additional operational flexibility has translated to smoother and more reliable plant operations, improved performance, and more consistent effluent quality.

OUTFALL MIXING ZONE ANALYSIS

Mixing zone tests were performed at the WRP outfall in the Rogue River on July 17, 1991. The tests were accomplished by injecting a tracer dye into the effluent at the treatment plant and tracking the dyed effluent plume in the river with special instrumentation. The Rogue River flow measured at the Grants Pass gauging station was approximately 2,300 cubic feet per second (cfs) during the test. The average effluent flow from the WRP during the test was 5.2 mgd (8 cfs).

The effluent plume hugged the right bank after exiting the outfall and remained within 12 feet of the shoreline (6 percent of the stream width) for more than 400 feet downstream. The edge of the immediate mixing zone was approximately 10 feet from the outfall with a minimum effluent dilution of approximately 1.7:1. Within 300 feet, which the DEQ has defined as the mixing zone for the Grants Pass outfall, the minimum dilution was 10:1. An outfall diffuser would improve near field mixing.

Appendix A contains the detailed mixing zone study report.

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EFFLUENT BIOASSAY ANALYSIS

Three bioassay test series were performed on WRP effluent in 1996. The October 1996 test was a resample from the August 1996 test. Each series consisted of three effluent samples. Northwestern Aquatic Sciences ran bioassays on the three samples using both fathead minnows and daphnia exposed to various effluent concentrations, in accordance with the bioassay plan approved by the DEQ.

February 1996. Sixty percent of the daphnia died when exposed to 100 percent effluent for 48 hours. This constitutes failure of the acute toxicity test. A 7-day exposure chronic toxicity test showed that while a concentration of 50 percent effluent allowed daphnia survival, their reproduction was significantly reduced. Normal reproduction was observed at concentrations at or below 25 percent effluent. Using standard mathematical techniques, the laboratory calculated that an exposure concentration of 7.5 to 41 percent would theoretically allow normal daphnia survival and reproduction.

August 1996. All fathead minnows survived when exposed to 100 percent effluent during a 48-hour acute toxicity test and a 7-day chronic toxicity test.

The laboratory sent a portion of the August 21 effluent sample to a CH2M Hill laboratory for a 96-hour chronic exposure test with green algae. The algae showed no reduction in growth, even when grown in full-strength effluent.

October 1996. Retest for daphnia only; fathead minnows were not tested. The WRP effluent passed a 48-hour acute toxicity test (daphnia exposure to 100 percent effluent concentration). A 7-day exposure chronic toxicity test showed that while a concentration of 100 percent effluent allowed daphnia survival, their reproduction was significantly reduced at all concentrations except the 6.25 percent effluent concentration sample. Using standard mathematical techniques, the laboratory calculated that an exposure concentration of 2.3 to 8.6 percent would theoretically allow normal daphnia survival and reproduction. The report on the October tests cautions that the survival success data just barely satisfied the statistical analysis, and low reproductive success was noted in all effluent concentrations.

Full results of the bioassays are included as Appendix B.

The bioassay results raise concerns that some aquatic organisms may be adversely affected near the point of discharge, depending on the effluent concentration of exposure. The likely culprit is thought to be ammonia in the WRP effluent, which ranged from 9 to 25 mg/L in the 59 samples taken during the period of January 1996 through July 1997.

We used an Environmental Protection Agency (EPA) mixing zone data spreadsheet program to model ammonia concentrations in the Rogue River near the WRP outfall.[1] According to the

[1] Background ammonia concentration is taken to be 0.06 mg/L, reported as the average annual Rogue River ammonia concentration measured at Gold Hill.

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model, when a 1.7:1 dilution factor is applied (measured in the mixing zone analysis at the Zone of Immediate Dilution) to the available data, ammonia levels are well below acute toxicity thresholds throughout all months of the year, reaching a maximum of 48.7 percent of the threshold for the month of November. However, chronic toxicity is a tougher standard. The model predicts that in July the ammonia concentration at the edge of the mixing zone (measured in the mixing zone analysis at a dilution of 10:1) will slightly exceed the chronic toxicity threshold, reaching a concentration 102.5 percent of threshold level.

The ammonia toxicity model therefore predicts that this effluent should not cause acute toxicity in the Rogue River at any time during the year, but may exert a chronic toxic effect during the month of July. However, the existing data, consisting of a few grab samples during some months, is not representative of the actual plant effluent. Concerns raised by the bioassay and modeling results would best be addressed by the implementation of a formal effluent ammonia monitoring program. The WRP has recently begun twice per week ammonia samples on a year-round schedule, which will improve the data. A more detailed mixing zone analysis should be conducted. This would include outfall dilution modeling and an effluent sampling plan to fully characterize ammonia and temperature variations.

SOLIDS MANAGEMENT

Land application of wastewater treatment biosolids (sludge) has been successfully practiced by the City of Grants Pass for many years. A number of land application sites are currently approved and in use. The focus of the land application program has been the beneficial use of biosolids as a partial or complete substitute for commercial fertilizers on agricultural lands.

Biosolids Characteristics

The fertilizer value and metals content are two critical parameters that control sludge application rates and feasibility. The sludge solids quantity and concentration influence the application methods and site area requirements.

Quantity. The City of Grants Pass Water Restoration Plant currently generates between 14,000 and 22,000 gallons of sludge per day. The No. 2 digester, with a capacity of 133,000 gallons, is used for sludge storage. The sludge is produced, stored, and transported in a liquid form. The average solids concentration is approximately 2.2 percent. From August 1996 through July 1997, the plant produced a total of 606 dry tons of biosolids.

Quality. Digested sludge quality is monitored for several distinct purposes.

1. Nutrient levels are measured to determine appropriate application rates to fertilize agricultural lands.
2. Metal concentrations are monitored regularly to ensure that "clean sludge" criteria are met.

3. If Class A or Class B certification is desired, pathogen reduction and vector attraction reduction criteria must be met in addition to the metals criteria.

Nutrient test results from five samples collected from the No. 2 digester are presented in Table 3-5. These test results indicate the total nitrogen concentration averages 9.4 percent. Ammonia nitrogen is roughly 5.4 percent, nitrate nitrogen is negligible, and the remaining 4 percent is organic nitrogen. With thin (less than 1/2 inch) surface applications of liquid sludge, approximately half of the ammonia nitrogen will be lost to the atmosphere through volatilization. Approximately 20 percent of the organic nitrogen can be expected to be made available through microbial transformation during the course of the first year following application. This results in roughly 70 pounds of projected available nitrogen per dry ton of sludge. Three to 5 percent of the organic nitrogen (2.4 to 4 pounds per dry ton) will continue to become available during years 2 through 5.

Table 3-5. Grants Pass Sludge Quality Nutrient Concentrations, Percent

Nutrient	Sample date					
	9/11/96	11/18/96	2/5/97	4/16/97	6/17/97	Average
NH_3 Nitrogen	6.5	5.86	3.8	5	5.8	5.392
NO_3 Nitrogen	0.009	0.002	0.0033	0.0022	0.001	0.0035
TKN	12	11.6	6.7	7.7	8.8	9.36
Total P	9.4	4.8	9.5	3.4	3.7	6.16
Potassium	0.882	1.01	0.924	1.1	0.889	0.961

Nitrogen is but one of the essential plant nutrients available in this material. One dry ton of sludge will also provide roughly 123 pounds of phosphorus and 19 pounds of potassium. The soils to which this sludge is applied will also benefit from the addition of over 1,000 pounds of volatile solids per dry ton, which is transformed into soil organic matter by microorganisms.

This material does contain trace amounts of regulated metals. Metals test results from five samples collected from the No. 2 digester are presented in Table 3-6 with the EPA 503.13 limits for clean sludge. The comparison shows that the Grants Pass digested sludge easily meets all clean sludge metals criteria.

Table 3-7 compares these concentrations with the current cumulative limits established by the EPA in 40 CFR Part 237 to calculate the land application limits for this sludge. Copper is the metal that would limit the total cumulative loading to 505 dry tons per acre (dT/Ac). This loading corresponds to a site life of 297 years based on the current annual sludge application of 1.7 dT/Ac.

Class A certification is not currently being sought by the Grants Pass facility.

Table 3-6. Grants Pass Sludge Quality Metals Concentrations, mg/kg dry solids

Compound	EPA 503.13 Regulations Table 3[a]	Sample Date					
		9/11/96	11/18/96	2/5/97	4/16/97	6/17/97	Average
Arsenic	41	ND @ 0.5	ND @ 1	ND @ 1	ND @ 1	ND @ 1	
Cadmium	39	3.9	3.5	1.7	3.2	2.5	3
Chromium	1,200	26	22	33	26	20	25
Copper	1,500	475	485	379	386	480	441
Lead	300	175	107	95	93	124	119
Mercury	17	5.43	2.9	2.6	3.96	4.35	4
Molybdenum	75[b]	ND @ 10	ND @ 10	ND @ 10	ND @ 10	ND @ 10	
Nickel	420	17.4	20	39	21	21	24
Selenium	36	ND @ 1	ND @ 2	ND @ 2	ND @ 2	ND @ 2	
Zinc	2,800	919	894	684	674	868	808

[a] "Clean sludge" limit.
[b] Ceiling concentration permitted in EPA 503.13 Table 1 regulation for land application.

Table 3-7. Grants Pass Cumulative Land Application Sludge Loading Limits

Parameter metal	Sludge concentration, mg/kg	Cumulative level, lb/Ac	Sludge application to reach limit, dT/Ac	Site life, years
Lead	119	1,782	7,489	4,405
Zinc	808	891	551	324
Copper	441	445.5	505	297
Nickel	24	178.1	3,711	2,183
Cadmium	3	4.5	750	441

Land Application

The current sludge application methods and site characteristics are summarized in the following section.

Application Methods. The City of Grants Pass currently has a 3,400-gallon tanker truck and a 3,500-gallon tanker truck, each equipped with a spreader bar for direct application of liquid sludge in the field. This is suitable under relatively dry soil conditions. During wet weather, the truck can cause ruts in the field that interfer with farming. When site soils are too wet to safely apply sludge from the truck, an irrigation gun and 3,500 feet of pipe are used. This equipment extends the time frame for application. However, regulations prohibit sludge application if the depth to permanent groundwater is less than 4 feet and the depth to temporary groundwater is less than 1 foot.

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The No. 2 digester currently provides sludge storage equivalent to 4 to 6 days of sludge production. To keep pace with sludge production, an average of seven truck loads of sludge must be land applied each day (based on a 5-day work week). The ability to haul seven truck loads a day depends on the haul distance to the site and the application method. Spreading directly from the truck is faster than using the irrigation gun. Experience has shown that the practical maximum haul distance is approximately 15 miles one way. Haul distances to the sludge application sites currently range from 4 to 15 miles.

Site Management. Eighteen sites totaling 841 acres are approved to receive Grants Pass WRP sludge (Table 3-8). Average site size is 47 acres. Annual sludge loading rates are based on the available nitrogen content of the sludge and the nitrogen requirement of the crop grown on the application site. Pasture land is the most common application site. The average loading rate during August 1996 through July 1997 was 1.65 dT/Ac.

Current Land Application Constraints. The sludge land application program requires careful management to ensure that the solids disposal requirements are satisfied. Available sludge storage, sludge solids concentration, and application site haul distances all influence the success of the overall solids program.

Table 3-8. Land Application Sites in Use by the City of Grants Pass[a]

Site	Distance, miles[b]	Acres
Riverbanks	14	187
Hoover	4	22
Cohen	9	30
AVR	12	42
Aguiar	13	110
Pennington	13	50
Jackson	12	7
Klose	10	9
Buddenhager	10	50
Young	13	26
Jackson	12	12
Hill	11	37
Sorensen	11	45
Walking J	11	70
Heisner	11	45
Hart	10	60
Benton	14	12
Hyde	12	10

[a] Approved sites are subject to change. The sites listed are those currently used when this report was prepared.
[b] One-way haul distance.

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The plant's current limited storage capacity means that sludge must be hauled almost daily to ensure that solids accumulation will not adversely affect plant performance. Daily land application is extremely difficult to maintain during the winter. Wet weather severely limits application site suitability and the ability to apply sludge. The 2.2 percent average sludge solids concentration means that 97.8 percent of each haul trip is devoted to water. Combining this with long haul distances severely reduces the efficiency of the overall operation. For example, if the solids concentration were increased to 4 percent, only four trips rather than seven would be needed to handle the daily sludge production. Similarly, a higher solids concentration would also make better use of the plant's available solids storage capacity. In the future when it becomes necessary to travel longer distances to find a sufficient number of application sites, the solids hauling efficiency will become even more critical.

A 1991 plant audit by Brown and Caldwell revealed that the 3,400-gallon sludge hauling truck is near the end of its useful life. Addition of a new, second truck has helped the situation, but the old truck requires frequent repair. With seven sludge hauls per day, two reliable sludge trucks are a definite necessity. The importance of sludge hauling to the treatment process suggests that the city should consider replacing the old sludge truck.

Recognizing these constraints, the current solids management program is successfully meeting the plant's sludge handling needs. Sufficient application sites within reasonable hauling distances are being used. Sludge has been applied at the proper loading rates. The low sludge metals content presents no problems with long-term sludge application.

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DESIGN DATA

DESCRIPTION	
Population Served (1980)	20,500
Estimated population (1995)	22,000
Design Flows	
ADWF, mgd	
NPDES Permit (expires 1997)	4
Secondary treatment design (a)	3.9
AWWF, mgd	6.4
NPDES Permit (expires 1997)	8.4
Peak Month Flow, mgd	9.5
Peak Week Flow, mgd	16.5
Peak Day Flow, mdg	23.5
PWWF, mgd	27
Septage	Excluded
Design Loads	
Average BOD, ppd	6,700
Peak Month BOD, ppd	9,400
Average SS, ppd	7,100
Peak Month SS, ppd	9,900
Raw Sewage Pump	
Existing, Number - HP	3-75
Type	VS centrifugal
Capacity each, mgd	9
New, number - HP	1-125
Type	VS centrifugal
Capacity, mgd	12
Station firm capacity, mgd	27
Screens	
Mechanically Raked	
Number	1
Bar Spacing, inches	0.5
Width, feet	5
Capacity, mgd	23.5
Manually Raked	
Number	1
Bar Spacing, inches	0.75
Width, feet	5
Capacity, mgd	23.5
Screenings Compactore	
Number	1
Type	Screw
Diameter, inches	10
Speed, rpm	5
Capacity, ct/hr	10
Primary Sedimentation Basins	
Existing	
Number	1
Diameter, feet	70
Sidewater depth, feet	9
Overflow rate at PWWF, gpd/sf	2750
Capacity PWWF, mgd	10.8
New	
Number	2
Length, feet	108
Width, feet	21.5
Sidewater depth, feet	11
Overflow rate at PWWF, gpd/sf	3500
Overflow rate at 3.9 mgd, gpd/sf	840
Overflow rate at 9.5 mgd, gpd/sf	2050
Capacity each PWWF, mgd	8.2
Total primary treatment capacity, mgd	27
Primary treatment efficiency	
SS removal, %	
ADWF 3.9 mgd	60
Peak day 23.5 mgd	30
BOD removal, %	
ADWF 3.9 mgd	30
Peak day 23.5 mgd	15

DESCRIPTION	VALUE
Primary sludge and grit pumps	
Circular primary	
Type	Recessed impeller
Number	1
Hp	10
Capacity, each, gpm	210
Rectangular primary	Recessed impeller
Type	3
Number	10
Hp	220
Capacity, each, gpm	
Primary scum pumps	
Circular primary	
Type	Progressive cavity
Number	1
Hp	5
Capacity, each, gpm	25
Rectangular primary	Progressive cavity
Type	1
Number	7.5
Hp	50
Capacity, each, gpm	
Grit cyclone	
Number	2
Capacity, each, gpm	220
Underflow, each, gpm	18
Grit washer	1
Number	15
Screw diameter, inches	18
Slurry flow, gpm	2.6
Overflow rate, gpm/sf	
Activated sludge aeration tanks	2
Number	3
Baye/2 tanks	22.5
Volume/bay, 1000 cf	112.5
Total volume, 1000 cf	Coarse bubble
Diffuser type	
Conventional treatment (ADWF) (b)	48.5
Loading, ppd BOD/1000 cf	32
Loading, ppd BOD/100 lb MLSS	
(Based on 2500 mg/l MLSS)	
Contact stabilization (PWWF) (b)	
(Assuming solid 5000 mg/l in sludge	
reaeration tank and RAS rate 0.5 x ADWF)	
1 bay contact tank	
Loading, ppd BOD/100 lb MLSS	24.5
Detention time in contact tank, min.	16.5
2 bay contact tank	
Loading, ppd BOD/100 lb MLSS	31
Detention time in contact tank, min.	32.5
RAS pumps	
Number	2
Type	VS propeller
Hp	15
Capacity, each, mgd	3.9
WAS pump	1
Number	VS Self prime
Type	7.5
Hp	150 - 250
Capacity, range, gpm	
WAS flow meter	Magnetic
Type	4
Diameter, inches	300
Capacity, each, gpm	

DESCRIPTION	VALUE
Aeration blower	
Number	3
Type	Centrifugal
Hp	125
Capacity each, 1000 cfm	2.5
Combined capacity, 1 standby, 1000 cfm	7.5
Maximum channel air (Jcfm/ft), 1000 cfm	1.45
Air supplied/ADWF gal treated, 1000 cfm	2.77
Hp installed/1000 lbs (ADWF) BOD removed	101
Secondary sedimentation basins	
Number	2
Diameter, ft	75
Depth, average, ft	12.3
Overflow rate at 3.9 mgd, gpd/sf	440
Overflow rate at 8.5 mgd, gpd/sf	1075
Overflow rate at Peak flow, 13 mgd, gpd/sf	1470
Capacity, each peak flow, mgd	6.5
Capacity, total peak flow, mgd	13
Chlorine contact basin (out of service)	
Number	1
Volume, 1000 cf	83
Diameter, ft	100
Depth, ft	8
UV Disinfection system	
Type	Medium pressure
Number of channels	1
Length, ft	36
Width, ft	6
Depth, ft	10
Hydraulic capacity, mgd	27
Lamp modules per channel, number	2
Lamps per module, number	48
Total number of lamps	96
Power consumption, peak, kw	289
UV dose, microwatt-sec/sq cm	28,400
Dimensions, approximate	
Length, ft	8
Width, ft	6
Height, ft	4
Exposure at 27 mgd, sec.	4.6
Effluent flow meter	
Type	ultrasonic
Diameter, inches	48
Range, mgd	0 - 30
Outfall	1
Number	36 and 42
Diameter, inches	Open end shoreline
Type	235
Length, ft	0.5
Submergence, min, ft	10:1
Dilution at 300 ft, plant flow 5.2 mgd and river flow 2600 cfs (c)	
Effluent quality BOD & SS mg/l	
May-Oct	<20
Nov-April	<30

DESCRIPTION	VALUE
Gravity thickener for primary sludge	
Number	1
Diameter, ft	30
Depth, ft	10.75
Solids loading average, lbs/day/sq ft	6
Hydraulic loading, gpd/sq ft	467
Expected solids concentration, percent	5
Thickened sludge/scum pumps, number	
Number	2
Type	Progressive cavity
Hp	7.5
Capacity, gpm	50
Thickened sludge flow meter	
Type	Magnetic
Diameter, inches	3
Capacity, gpm	100
Belt thickener for WAS	
Number	1
Belt width, meters	1.5
Solids loading, lbs/hr	700
Sludge flow at 0.7% solids, gpm	200
Capture, percent	90
Thickened sludge pump	
Number	1
Type	VS progressive vcoity
Hp	5
Capacity range, gpm	25-50
Thickened WAS flow meter	
Type	Magnetic
Diameter, inches	3
Capacity, gpm	75
Filtrate pump	
Number	1
Type	VS centrifugal
Hp	5
Capacity, gpm	300
Polymer system	
Neat polymer feed, gph	0.4 - 8
Polymer solution, gph	120 - 1200
Polymer feed 15% solution, lbs/hr	0.05 - 10.5
Polymer dose, lbs/ton solids	0.1 - 21
Anaerobic digester	
Number	2
Cover type	Floating
Diameter, ft	50
Depth, ft	30
Volume, 1000 cf	58
Average sludge loading, lbs/day	8000
Thickened sludge, percent	6
Detention time, days	20
Average gas production, 1000 cf/day	38
Circulating sludge pumps	2
Number	Recessed impellor
Type	20 & 7.5
Hp	600 & 250
Capacity, gpm	

DESCRIPTION	VALUE
Gas circulating compressor	
Number	1
Type	Rotary lobe
Hp	15
Capacity, scfm	250
Mixing power, Hp/1000 cf	0.25
Sludge holding tank (former secondary digester)	
Number	1
Diameter, ft	35
Depth, ft	13
Volume, 1000 cf	12.5
Sludge loading pump	
Type	VS progressive cavity
Hp	15
Capacity, gpm	410
Sludge grinder	
Type	In-line
Hp	5
Diameter, inches	8
Capacity, gpm	800
Reclaimed water pumps	3
Number	Vertical turbine
Type	2-15, 1-30
Hp	2-250, 1-500
Capacity, each, gpm	
Hypochlorite (3W chlorination)	500
Storage, gal at 12.5%	0.05 - 4
Feed pump, gph	96
Max dose, lbCl/day	
Plant drain pumps	2
Number	Centrifugal
Type	25
Hp	2100
Capacity, each, gpm	
Standby generator	1
Number	Gas turbine
Type	900
Capacity, kw	
EPA reliability class, current/required	I/II
Influent/effluent samplers	Flow proportional
Type	Plant flow meter
Sample pacing	
Service and instrument air compressors	Reciprocating
Type	10
Hp	100
Pressure, psi	40
Capacity, each, cfm	

(a) No change from 1972 design.
(b) 1972 design BOD loading 8,000 ppd and PWWF 13 mgd.
(c) Mixing zone study conducted July 1991.

BROWN AND CALDWELL

SUBMITTED: _____ (PROJECT MANAGER) DATE: _____
APPROVED: _____ (BROWN AND CALDWELL) DATE: _____
APPROVED: _____ DATE: _____

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2"-SCALE ACCORDINGLY)

FILE 1216
DRAWN RFJ/GTM
DESIGNED JCD
CHECKED K.H.
CHECKED _____

RECORD DRAWING
This record drawing was prepared using information reported to Brown and Caldwell and contains only the standard and customary level of detail. The information was not independently field verified. There is no ongoing program to update the drawings to reflect changes subsequent to the date indicated. Therefore, this drawing cannot be relied upon as an exact representation of actual conditions.
By _____ Date _____

ZONE	REV.	DESCRIPTION	BY	DATE	APP.
	1	Add data per DEQ request	JCD	1/93	
	2	AS CONSTRUCTED	NDS	10/93	

REVISIONS

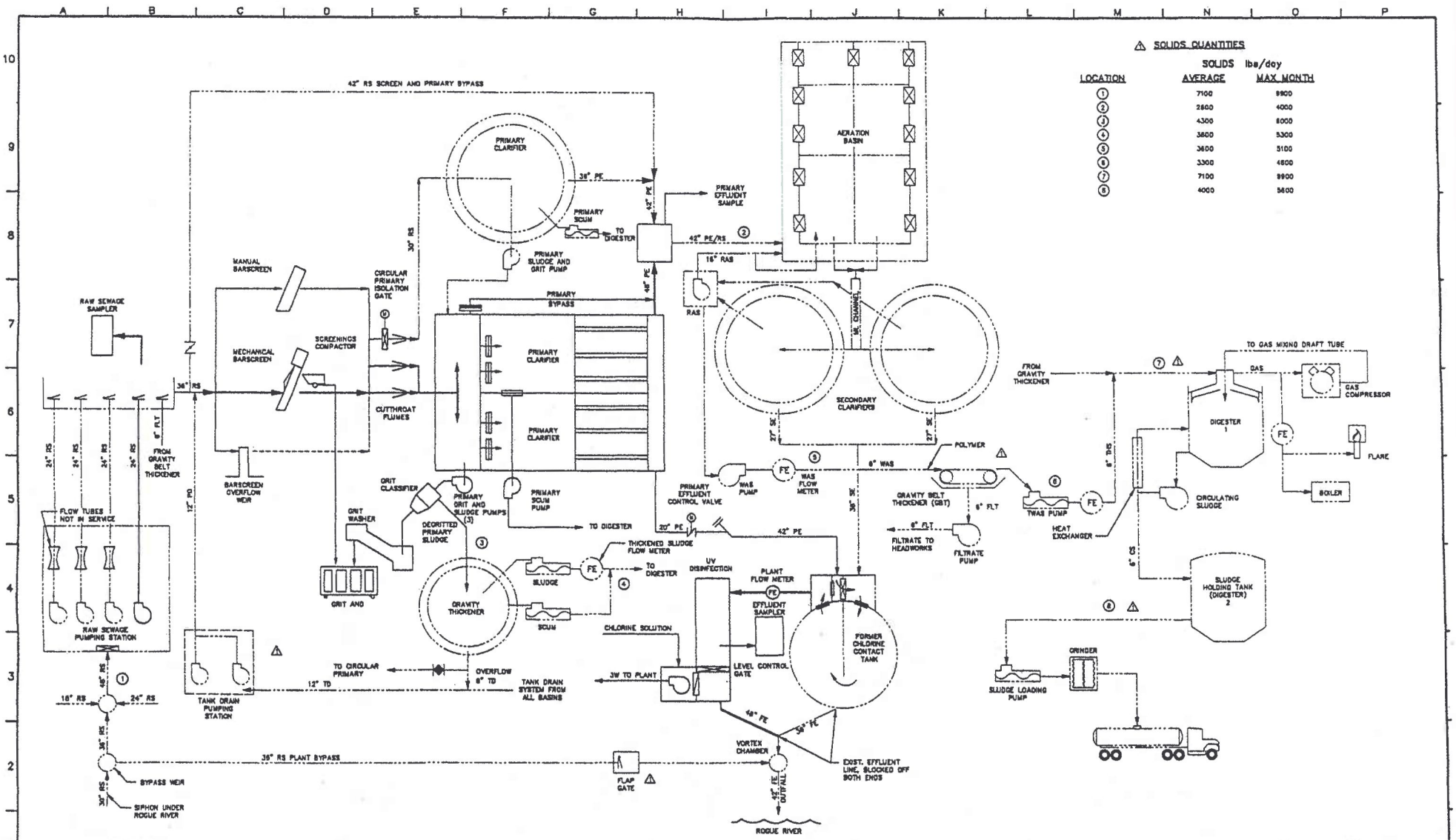
CITY OF GRANTS PASS

WATER RESTORATION PLANT
HEADWORKS, PRIMARY AND DISINFECTION

TABLE 3-1
DESIGN DATA

SCALE: NO SCALE
DRAWING NUMBER: G7
SHEET NUMBER: 8A

00539



BROWN AND CALDWELL

SUBMITTED: _____ DATE: _____
APPROVED: _____ DATE: _____
APPROVED: _____ DATE: _____

LINE IS 2 INCHES AT FULL SIZE (IF NOT 2"-SCALE ACCORDINGLY)

FILE 12L8
DRAWN REJ/GTM
DESIGNED JCD
CHECKED K.H
CHECKED

RECORD DRAWING
This record drawing was prepared using information reported to Brown and Caldwell and contains only the standard and customary level of detail. The information was not independently field verified. There is no ongoing program to update the drawings to reflect changes subsequent to the date indicated. Therefore, this drawing cannot be relied upon as an exact representation of actual conditions.
By _____ Date _____

ZONE	REV.	DESCRIPTION	BY	DATE	APP.
	1	SOLIDS HANDLING	JCD	1/95	
	2	AS CONSTRUCTED	MDS	10/96	

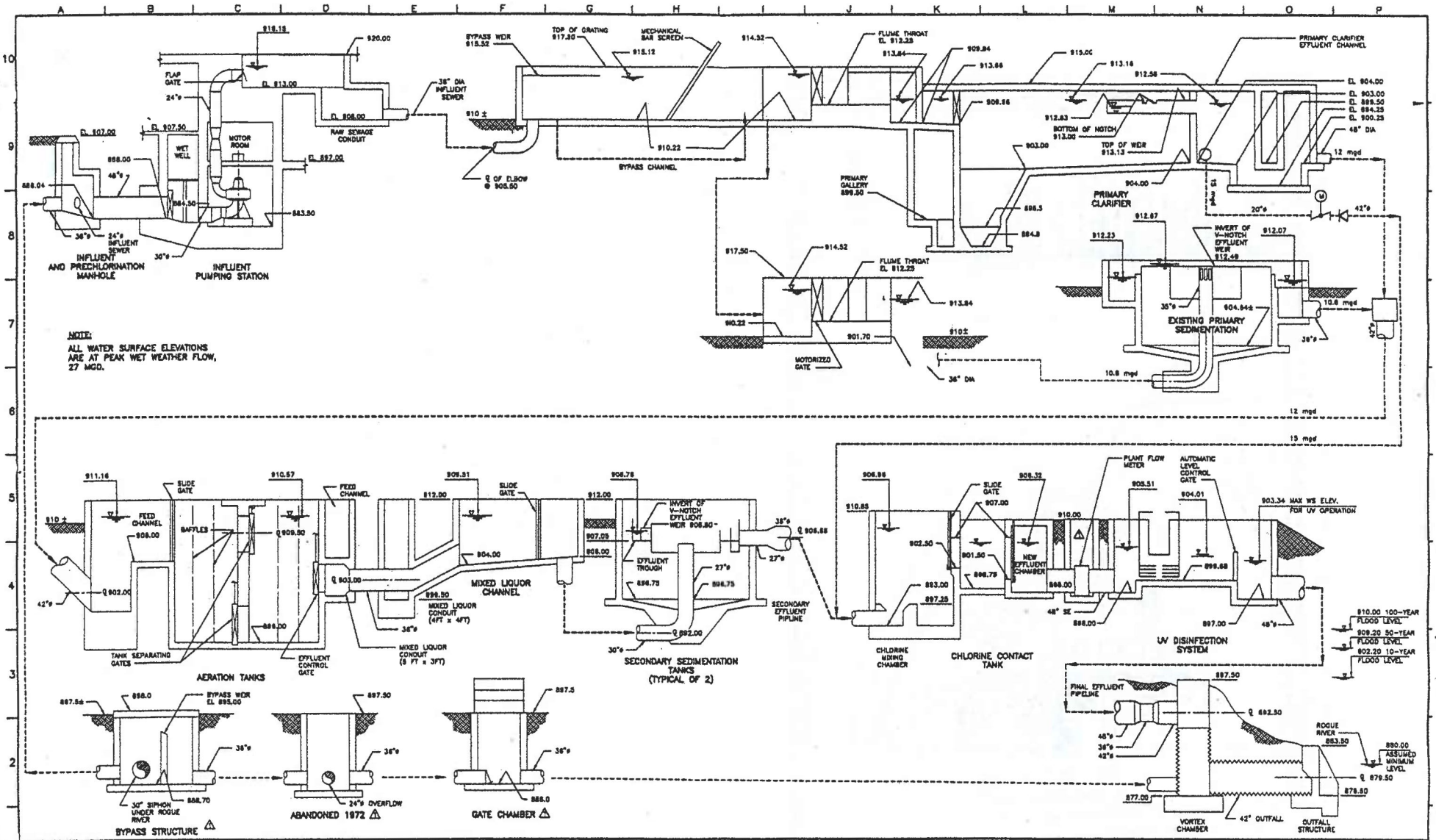
REVISIONS

CITY OF GRANTS PASS

WATER RESTORATION PLANT
HEADWORKS, PRIMARY AND DISINFECTION

FIGURE 3-2
PROCESS SCHEMATIC

SCALE: NO SCALE
DRAWING NUMBER: G6
SHEET NUMBER: 6



BROWN AND CALDWELL	CITY OF GRANTS PASS	FIGURE 3-3 HYDRAULIC PROFILE
	WATER RESTORATION PLANT HEADWORKS, PRIMARY AND DISINFECTION	SCALE: NO SCALE; DRAWING NUMBER: G5; SHEET NUMBER: 5

RECORD DRAWING: This record drawing was prepared using information reported to Brown and Caldwell and contains only the standard and customary level of detail. The information was not independently field verified. There is no ongoing program to update the drawings to reflect changes subsequent to the date indicated. Therefore, this drawing cannot be relied upon as an exact representation of actual conditions.

ZONE	REV.	DESCRIPTION	BY	DATE	APP.
	2	BYPASS STRUCTURE AND GATE CHAMBER AS CONSTRUCTED	JCD MDS	1/95 10/96	

REVISIONS

SUBMITTED: _____ DATE: _____
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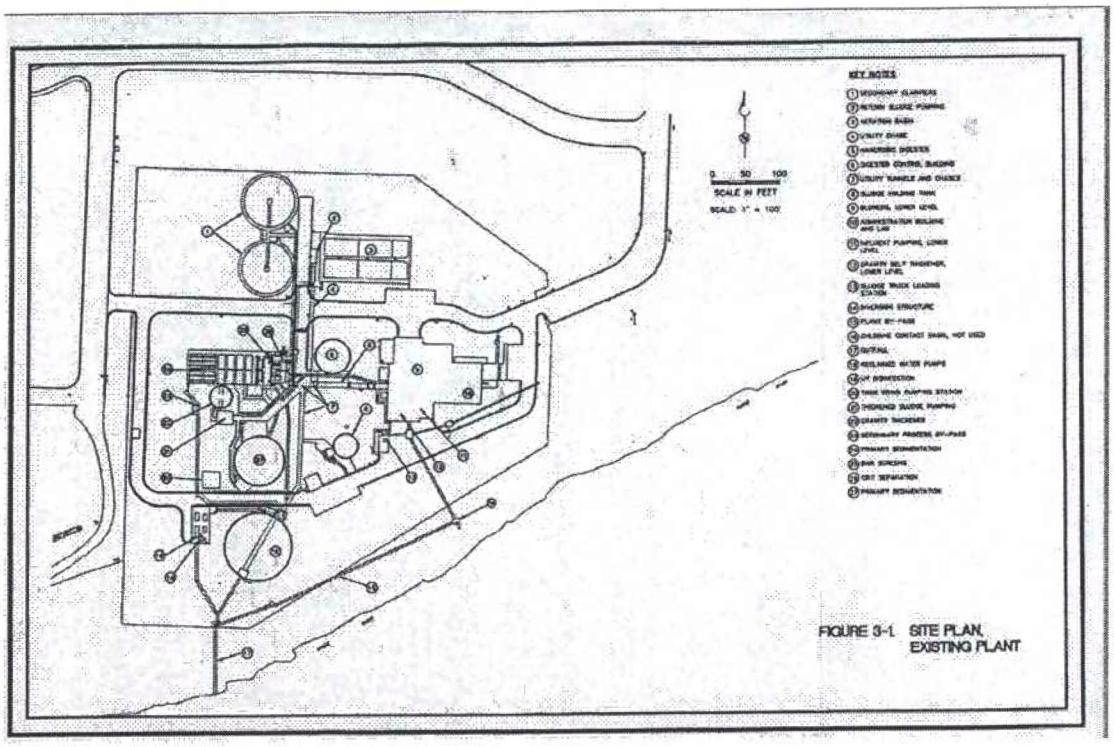

FIGURE 3-1 SITE PLAN, EXISTING PLANT

CHAPTER 4

WASTEWATER CHARACTERISTICS

The design of wastewater treatment and disposal facilities is dependent mainly upon estimates of hydraulic and organic loading. Differences in physical and socioeconomic environments cause wastewater characteristics to vary among communities. Therefore, wastewater characteristics, including volume, strength, and composition, were determined by reviewing historical operating records for the Grants Pass Water Restoration Plant (WRP).

DEFINITION OF TERMS

As a preface to the review of wastewater characteristics, definitions of the terms used in this study are presented below:

Wastewater. The total fluid flow in a sewerage system. Wastewater may include sanitary sewage, industrial wastes, commercial wastes, and infiltration and inflow (I/I).

Sanitary Sewage. Waterborne waste principally derived from the sanitary conveniences of residences and institutions.

Industrial Wastes. Waterborne wastes produced as the result of manufacturing or processing operations.

Commercial Wastes. Waterborne wastes derived from business establishments.

Infiltration. Water that enters the sewerage system from the surrounding soil. Common points of entry include broken pipe and defective joints in pipe and manhole walls. Although generally limited to sewers laid below the normal groundwater level, infiltration can also occur when rain or irrigation water soaks into the ground adjacent to residences and enters shallow house sewer laterals with defective pipes or joints.

Base Infiltration. Water that enters the sewerage system from the surrounding soil during periods of low groundwater.

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

Biochemical Oxygen Demand (BOD). A measure of wastewater strength in terms of the quantity of oxygen required for biological oxidation of the organic matter contained in the wastewater. The BOD loading imposed on a treatment plant influences both the type and degree of treatment that must be provided to produce the required effluent quality. All references to BOD in this report are 5-day BOD tests incubated at 20° C.

Suspended Solids (SS). A measure of the quantity of suspended material contained in the wastewater. The quantity of SS removed during treatment influences the sizing of sludge handling and disposal processes as well as the effectiveness of disinfection with chlorine.

CURRENT FLOWS AND LOADS

Developing accurate estimates of current plant flows and loads is a critical step in the facilities planning process. The current flows and loads serve as the basis for estimating future flows and loads; these flow and load projections are in turn used in the sizing of new wastewater treatment and conveyance facilities. For this evaluation, we analyzed the Grants Pass WRP records for January 1991 through December 1996.

Wastewater Flows

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

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

Historical daily plant flows are depicted in Figure 4-1.

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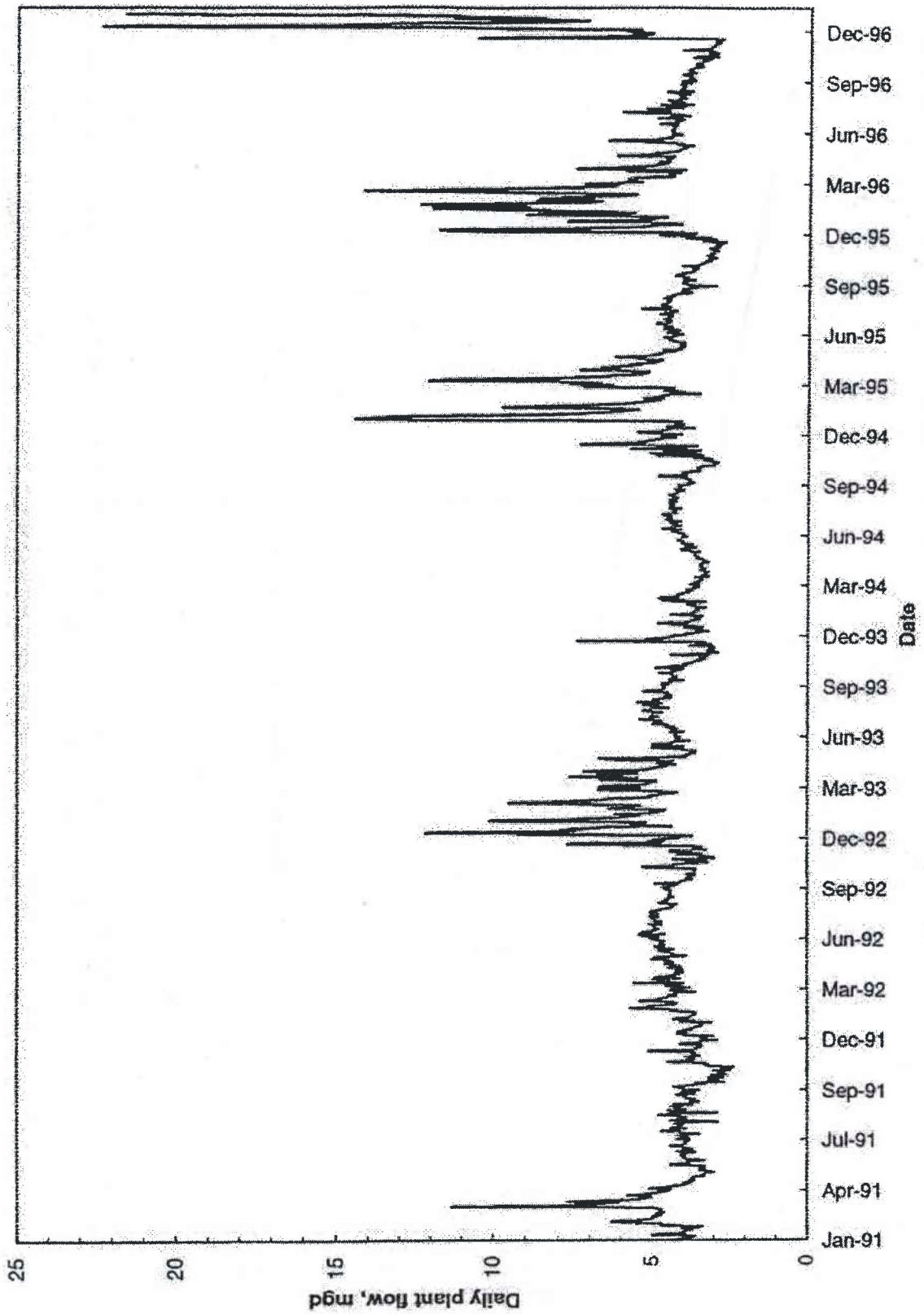

Figure 4-1. Daily Plant Flows

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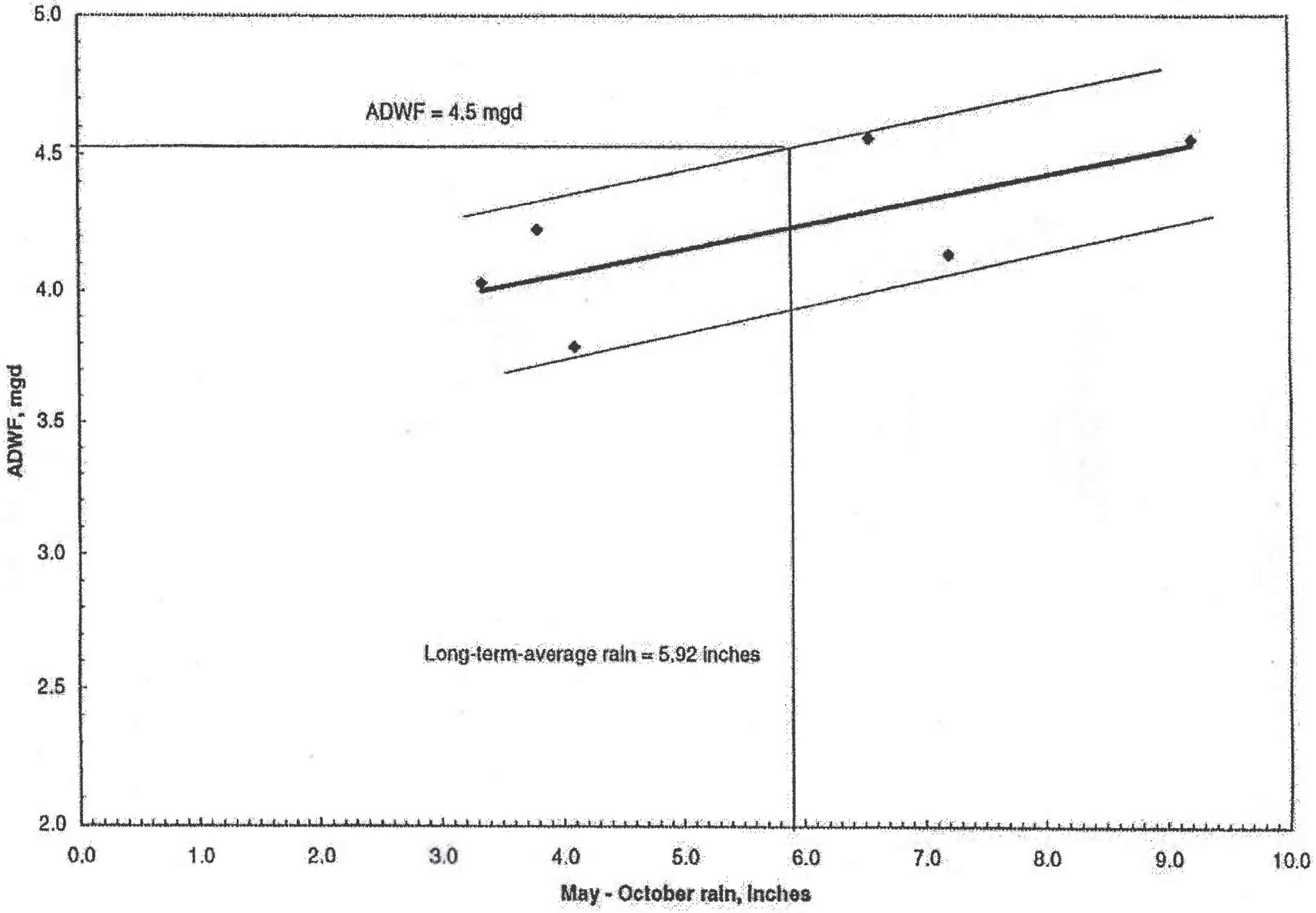

Figure 4-2. Average Dry Weather Flow

Flow Records. In evaluating wastewater flow records, it is necessary to first identify any limitations in flow measurement or pumping capacity. In addition, any unique or unusual conditions which could affect historical flow records should be ascertained. Two factors must be considered when analyzing Grants Pass' peak flow records:

- Southern Oregon experienced drought conditions from the mid-1980s through the mid-1990s. Only in the past 2 years have annual rainfall quantities exceeded historical averages.

- Prior to the construction of new primary sedimentation basins in 1996, flows which exceeded the capacity of the plant were bypassed to the river. The bypass quantity was estimated.

Rainfall Records. Peak wastewater flows are heavily influenced by rainfall. Therefore, the techniques suggested by DEQ for calculating plant flows require consideration of statistical recurrences of rainfall quantities. Table 4-1 is a statistical summary for rainfall in the Grants Pass area.

Table 4-1. Statistical Rainfall Summary for Grants Pass, 1951 Through 1980

Month	Mean rainfall, inches	Greatest monthly rainfall, inches	Greatest daily rainfall, inches	1-in-5 Year recurrence rainfall, inches	1-in-10 Year recurrence rainfall, inches
Jan	6.46	14.20	3.35	9.17	11.36
Feb	4.21	10.61	4.30	6.19	7.90
Mar	3.46	8.14	2.23	5.17	6.70
Apr	1.73	5.32	1.41	2.58	3.34
May	1.34	4.73	1.43	2.12	2.93
June	.55	2.09	.91	0.91	1.34
July	.21	1.02	.81	0.38	0.63
Aug	.46	3.33	.75	0.79	1.32
Sept	.89	4.19	2.86	1.46	2.10
Oct	2.47	8.86	1.98	3.92	5.46
Nov	4.59	15.15	3.20	6.95	9.14
Dec	5.94	16.06	4.07	8.63	10.90
Average	32.31				
Maximum	6.46	16.06	4.30	9.17	11.36

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Monthly Flows. Monthly average flows for January 1991 through December 1996 are presented in Table 4-2. The annual average flow for this period was 4.7 million gallons per day (mgd).

Table 4-2. Monthly Flows

Month	Plant flow, mgd	Month	Plant flow, mgd
Jan-91	4.07	Jan-94	3.82
Feb-91	5.03	Feb-94	3.95
Mar-91	6.14	Mar-94	3.67
Apr-91	3.97	Apr-94	3.36
May-91	3.61	May-94	3.73
Jun-91	3.95	Jun-94	4.18
Jul-91	4.11	Jul-94	4.41
Aug-91	4.05	Aug-94	4.36
Sep-91	3.82	Sep-94	4.05
Oct-91	3.20	Oct-94	3.43
Nov-91	3.39	Nov-94	4.63
Dec-91	3.64	Dec-94	4.65
Jan-92	3.72	Jan-95	8.06
Feb-92	4.27	Feb-95	5.45
Mar-92	4.41	Mar-95	6.97
Apr-92	4.37	Apr-95	5.66
May-92	4.43	May-95	4.50
Jun-92	4.79	Jun-95	4.48
Jul-92	4.96	Jul-95	4.57
Aug-92	4.73	Aug-95	4.37
Sep-92	4.45	Sep-95	3.96
Oct-92	3.97	Oct-95	3.48
Nov-92	3.79	Nov-95	3.09
Dec-92	5.41	Dec-95	6.00
Jan-93	6.69	Jan-96	8.16
Feb-93	6.29	Feb-96	8.36
Mar-93	5.43	Mar-96	5.44
Apr-93	5.81	Apr-96	5.07
May-93	4.47	May-96	4.49
Jun-93	4.33	Jun-96	4.40
Jul-93	4.79	Jul-96	4.49
Aug-93	4.92	Aug-96	4.14
Sep-93	4.61	Sep-96	3.87
Oct-93	4.24	Oct-96	3.42
Nov-93	3.32	Nov-96	4.50
Dec-93	4.11	Dec-96	11.89

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For most municipalities, the ADWF is relatively unaffected by rainfall. Nevertheless, we compared ADWF with May through October rainfall for the past 6 years. As shown on Figure 4-2 for the Grants Pass WRP, ADWF generally increases as rainfall increases. However, it is not a well-defined, linear relationship. Figure 4-2 shows that the flow-rainfall relationship is more of a band than a line. This suggests that factors other than rainfall, such as population growth, contribute to changes in ADWF. Using the upper boundary of the band and the long-term-average May through October rainfall of 5.92 inches, the current ADWF is estimated at 4.5 mgd.

The DEQ supports using the best-fit line for estimating wastewater flows. Figure 4-2 shows the best fit line corresponding to an ADWF of 4.2 mgd. In an effort not to underestimate this key flow parameter, we selected to use for planning purposes the higher, more conservative value of 4.5 mgd.

The AWWF is greatly influenced by rainfall. Figure 4-3 plots average flow and total rainfall for November through April for the past 6 years. As with ADWF, the relationship between flow and rainfall is more of a band than a straight line. Using the upper boundary of the band, the long-term-average November through April rainfall of 26.39 inches corresponds to a November through April flow of 5.7 mgd. This compares well to the average November through April plant flow for the past 2 years of 5.76 mgd. The AWWF corresponding to the best-fit line on Figure 4-3 is about 5.0 mgd. This value is significantly lower than the value observed for an average rainfall of about 25 inches. Therefore, we selected to use the more conservative approach, described by the upper band line, at 5.7 mgd.

To calculate maximum month flows, DEQ recommends plotting monthly plant flows and associated rainfall values for January through May of the most recent year (Figure 4-4). The MMWWF is estimated as the flow at the plant corresponding to the 1-in-5 year January rainfall. The 1-in-5 year January rainfall is 9.17 inches (Table 4-1). Therefore, from Figure 4-4, the MMWWF is taken as 10 mgd. This is supported by the February 1998 monthly average flow of 9.4 mgd and total rainfall of 9.1 inches. The highest monthly average flow on record, 11.9 mgd, occurred in December 1996. Two large storm events occurred in December 1996. The total rainfall for the month was 22.2 inches, which far exceeded the previous record of 16.06 inches associated with the flood of December 1964. Figure 4-5 plots daily flows and rainfall for December 1996. Because the 11.9-mgd flow occurred during a month with record rainfall, it is reasonable to expect that the 1-in-5-year MMWWF would be significantly lower.

In similar fashion, the MMDWF is approximated as the flow associated with a 1-in-10 year May rainfall (2.93 inches). From Figure 4-4, the MMDWF is 6.7 mgd.

As with ADWF and AWWF, the DEQ supports estimating MMDWF and MMWWF as values corresponding to the best-fit line shown on Figure 4-4. This approach would yield a value of 5.3 mgd for MMDWF and 8.6 mgd for MMWWF. For planning purposes, we selected to use the more conservative values as discussed above.

Peak Flows. The peak flows of interest are the peak day flow, the peak week flow, and the PWWF. The peak day flow is estimated as the flow associated with the 1-in-5 year, 24-hour storm event. For Grants Pass, this storm event is 3.5 inches of rainfall. However, it is also important to ensure that the antecedent conditions contribute to maximum I/I to the collection

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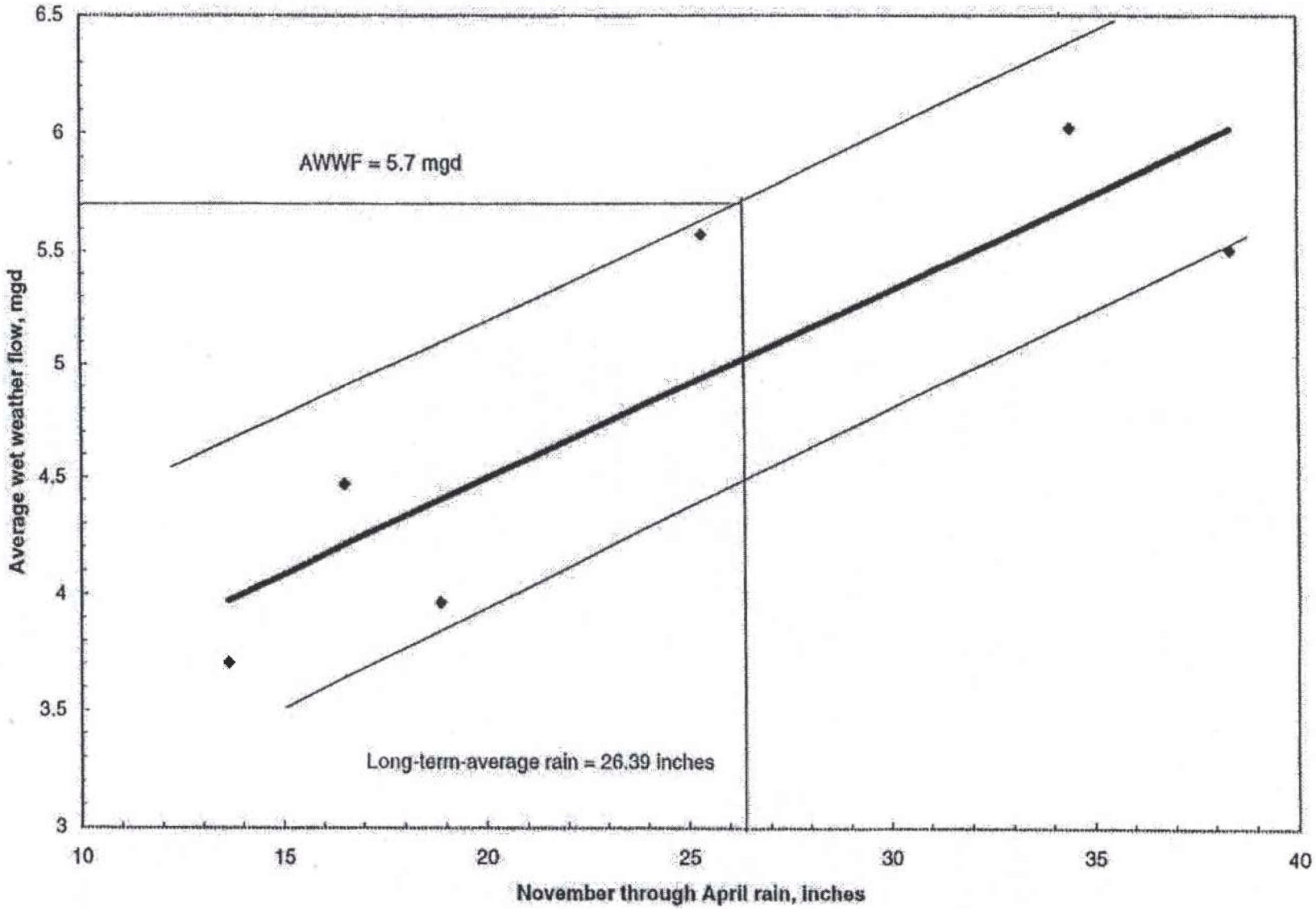

Figure 4-3. Average Wet Weather Flow

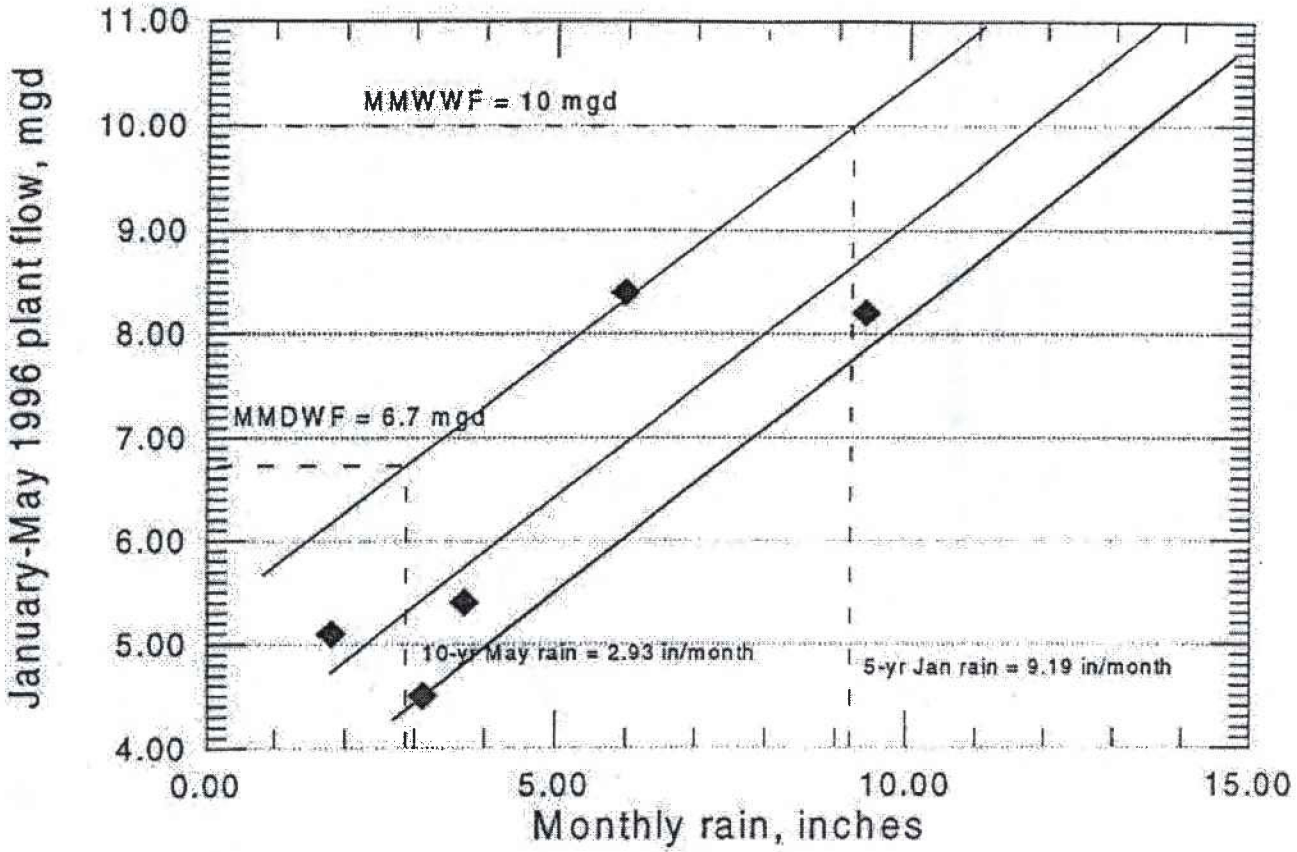

Figure 4-4. Maximum Month Wet Weather Flow

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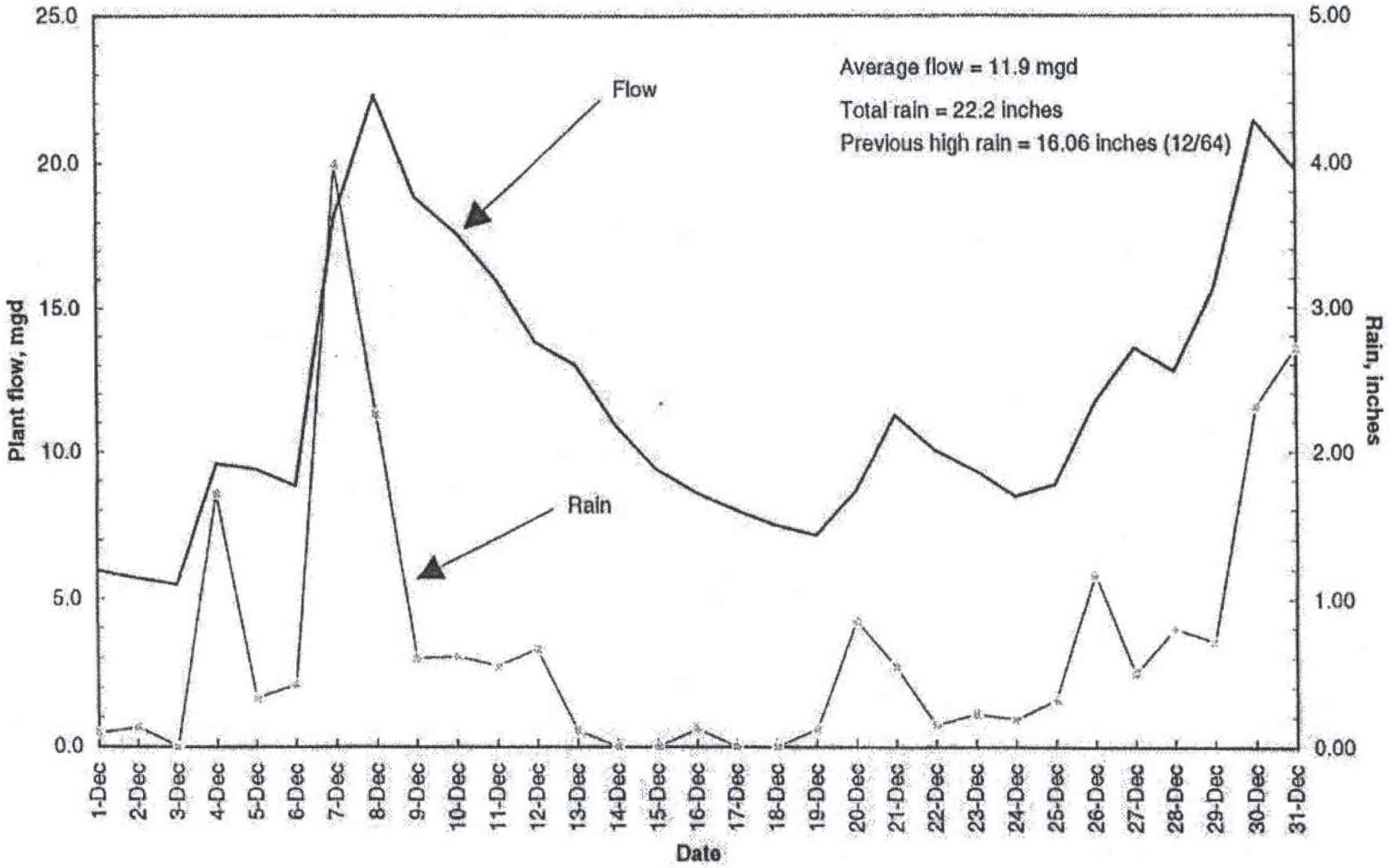

Figure 4-5. December 1996 Flow and Rainfall

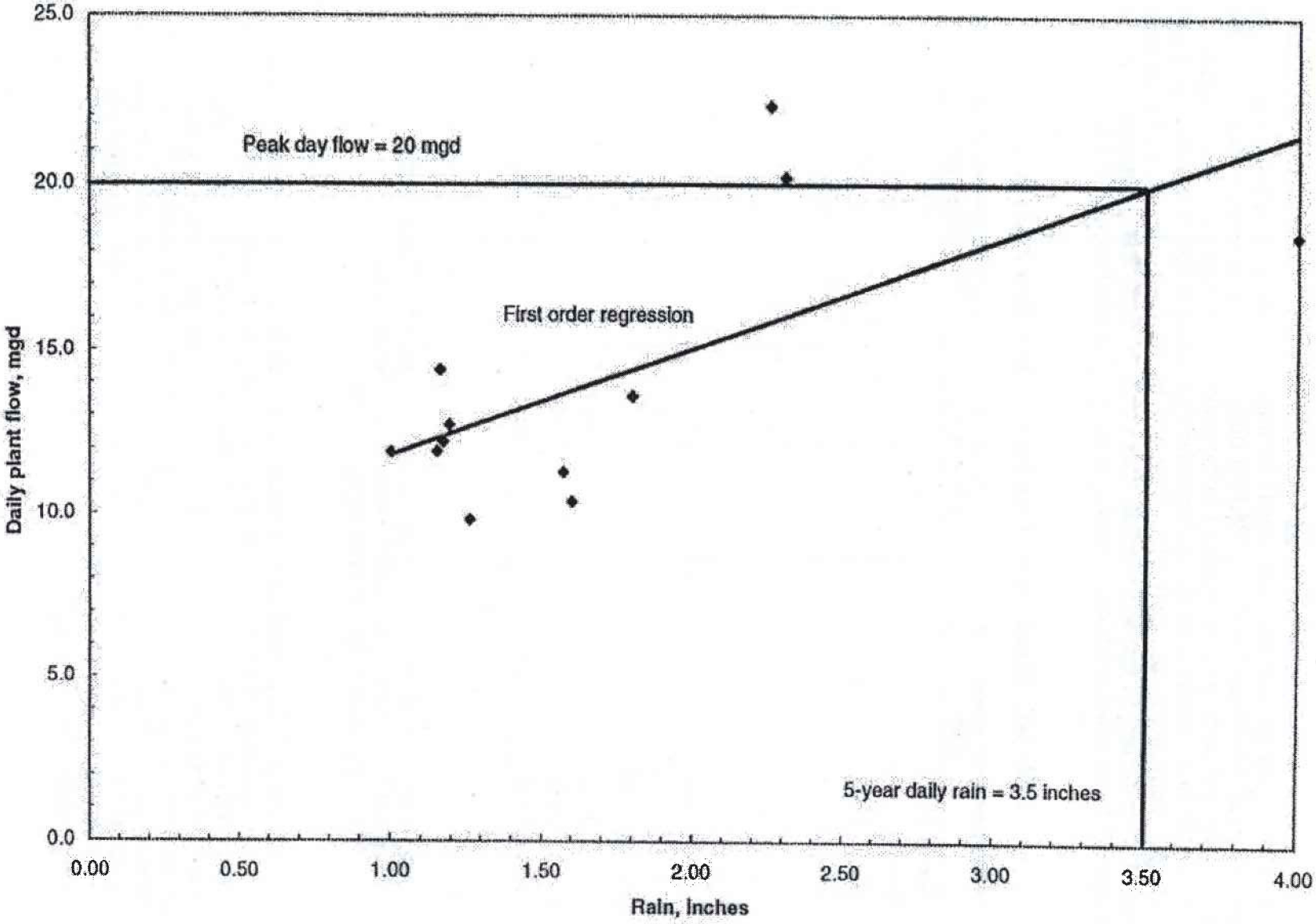

Figure 4-6. Peak Day Flow Analysis

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system. That is, the groundwater level should be high and there should be several days of significant rainfall prior to the 1-in-5 year, 24-hour storm event to ensure soil saturation. Table 4-3 summarizes the storm events considered in the peak day flow analysis, while Figure 4-6 plots these daily flows against rainfall. A first-order regression predicts that during the 1-in-5 year, 24-hour storm event, the daily plant flow will be 20 mgd. In this case, the flow corresponding to the best fit line was selected. This value, 20 mgd, was selected because it is well supported by two data points corresponding to daily rainfalls of 4.0 and 2.3 inches.

Table 4-3. Storm Events Used for Peak Day Flow Analysis

Date	Rain, inches	Plant flow, mgd[1]
Dec 7, 1996	4.00	18.7[2]
Dec 30, 1996	2.31	20.2
Dec 8, 1996	2.26	22.8[2]
Dec 31, 1996	2.72	24.4[2]
Jan 9, 1995	1.80	13.6
Jan 26, 1996	1.60	10.4
Mar 4, 1991	1.57	11.3
Mar 19, 1995	1.26	9.8
Feb 20, 1996	1.19	12.7
Dec 26, 1996	1.17	12.2
Jan 11, 1995	1.16	14.4
Jan 23, 1996	1.15	11.9
Jan 20, 1996	1.00	11.9

Notes:

[1] Evaluation limited to storm events where a high groundwater table was anticipated and rainfall occurred for several days before storm.
[2] Daily flow includes estimated plant bypass volume.

DEQ suggests using probability methods to estimate other peak flows. From the above analysis of rainfall and historical flow data, three flow rates and their corresponding recurrence probability are known: annual average flow, MMWWF, and peak day flow. The annual average flow has a recurrence probability of 50 percent. Assuming that the wet weather flows of interest all occur during a year with 1-in-5 year recurrence probability rainfall, the MMWWF has a recurrence probability of 1 month in 12 months, or 8.33 percent. Similarly, the peak day flow has a recurrence probability of 1 day in 365 days, or 0.27 percent. As predicted in the DEQ flow calculation guidelines, plotting these three points on log-probability scales approximates a straight line (Figure 4-7).

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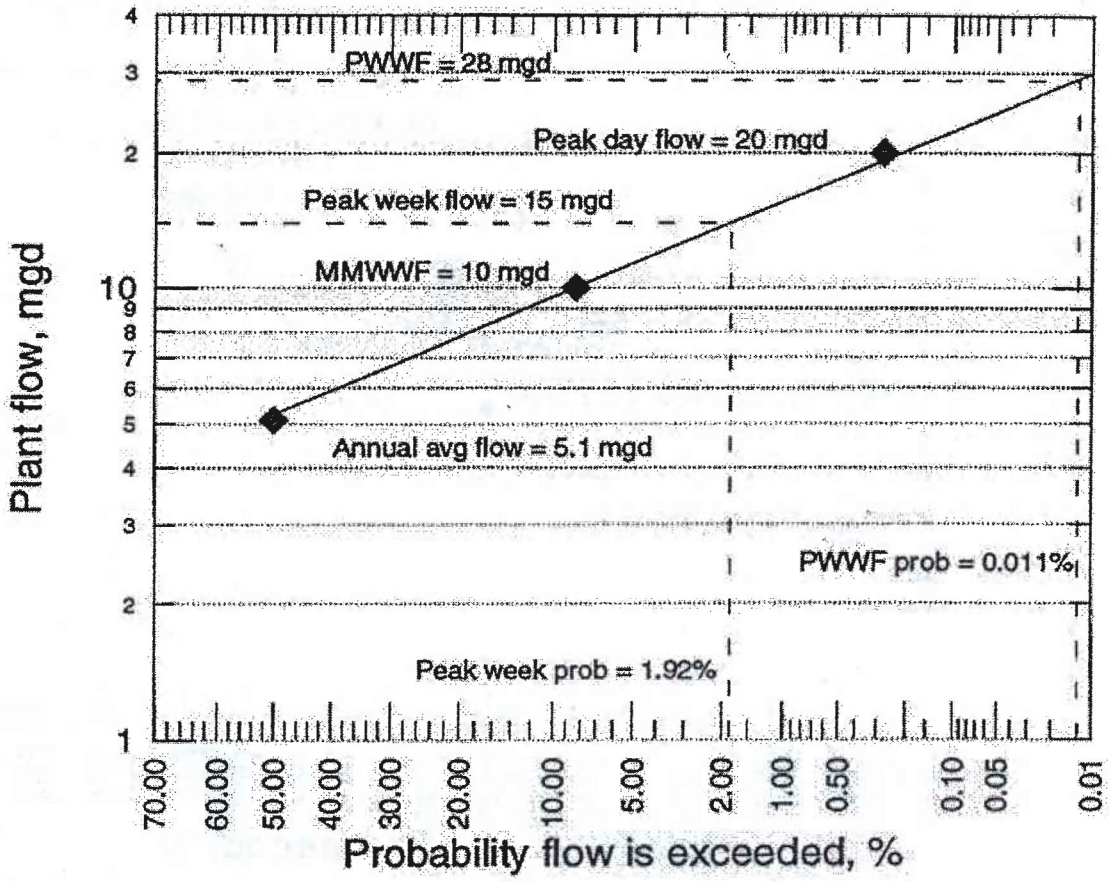

Figure 4-7. Flow Probability

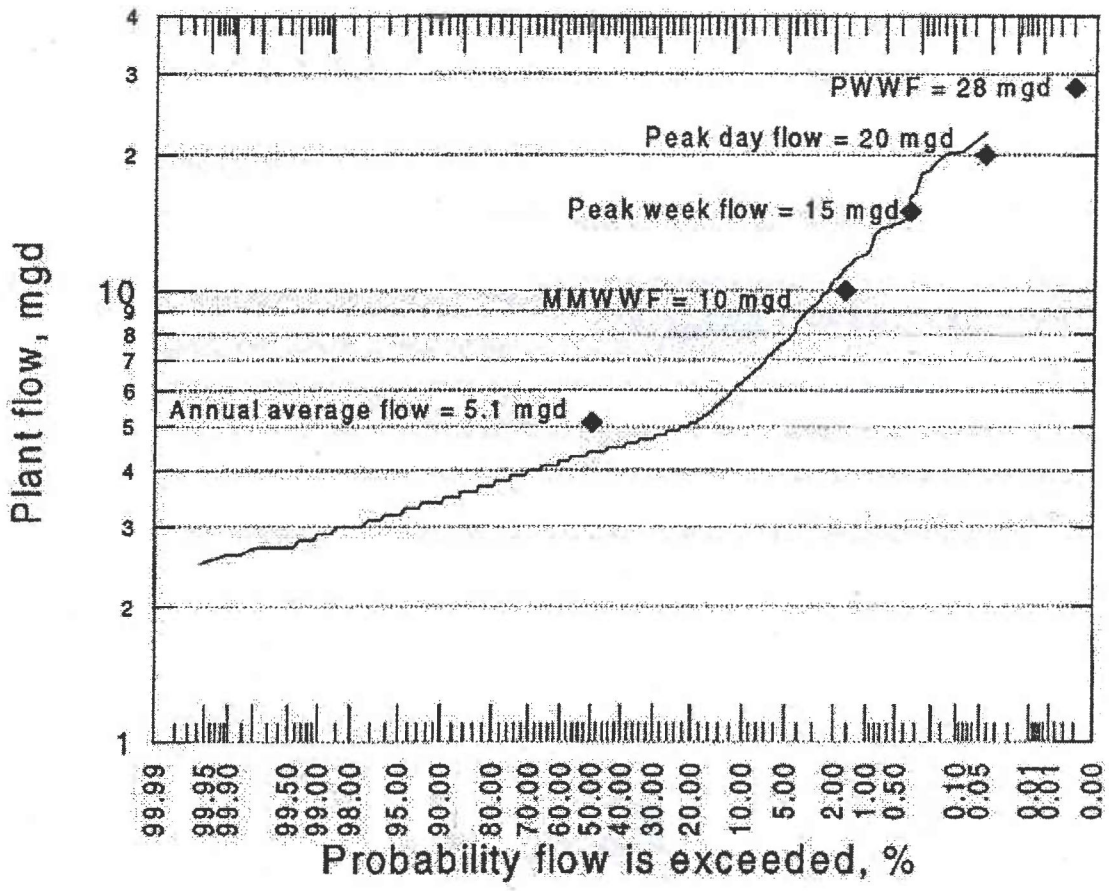

Figure 4-8. Comparison of Actual and Calculated Flows

Figure 4-7 can now be used to estimate PWWF and peak week flow. PWWF is defined as the peak flow sustained for 1 hour. PWWF has a recurrence probability of 1 hour in 8,760 hours (1 year), or 0.011 percent. From Figure 4-7, the current PWWF is about 28 mgd. This corresponds well with measured values; the highest instantaneous flow measured between January 1991 and December 1996 was 26.5 mgd.

Peak week flow is estimated in the same manner as PWWF. The peak week flow has a recurrence probability of 1 week in 52 weeks (1.92 percent); this corresponds to a flow of about 15 mgd.

As a check of the flow estimates, we also analyzed plant flows without consideration of rainfall using flow-probability methods. All daily plant flows were sorted and assigned a recurrence probability. The actual plant flows are plotted on Figure 4-8 along with the flows estimated using DEQ rainfall methods. As shown in the figure, the actual plant flows compare well with the calculated flows.

Current flows for the Grants Pass WRP are summarized in Table 4-4. This table presents wastewater flows estimated by both selecting values corresponding to the upper bound of the rainfall-flow plots and the values corresponding to the best-fit line on the rainfall-flow plots. The only flow conditions where there is a significant difference in these approaches are MMDWF and MMWWF. For planning purposes, we selected to use the more conservative, higher values. However, these values may be revisited during predesign and alternate values selected.

Table 4-4. Current Wastewater Flows

Description	Flow, mgd	
	Upperbound	Best fit
ADWF	4.5	4.2
Average annual flow	5.1	4.6
AWWF	5.7	5.0
MMDWF	6.7	5.3
MMWWF	10	8.6
Peak week flow	15	15
Peak day flow	20	20
PWWF	28	28

Nutrient Loading

Nitrogen and phosphorus are key nutrients for sustaining a biological treatment system. Discharge of high levels of these nutrients can have significant effects on the receiving stream. These nutrients enter the treatment process in the raw wastewater and in recycle streams within the plant.

The principal source of phosphorus is sanitary wastes, which include detergents that contain phosphorus. Historically, the phosphorus content of the raw wastewater has not been a concern to plant operations, and therefore, samples were not taken. A phosphate detergent ban is currently in affect.

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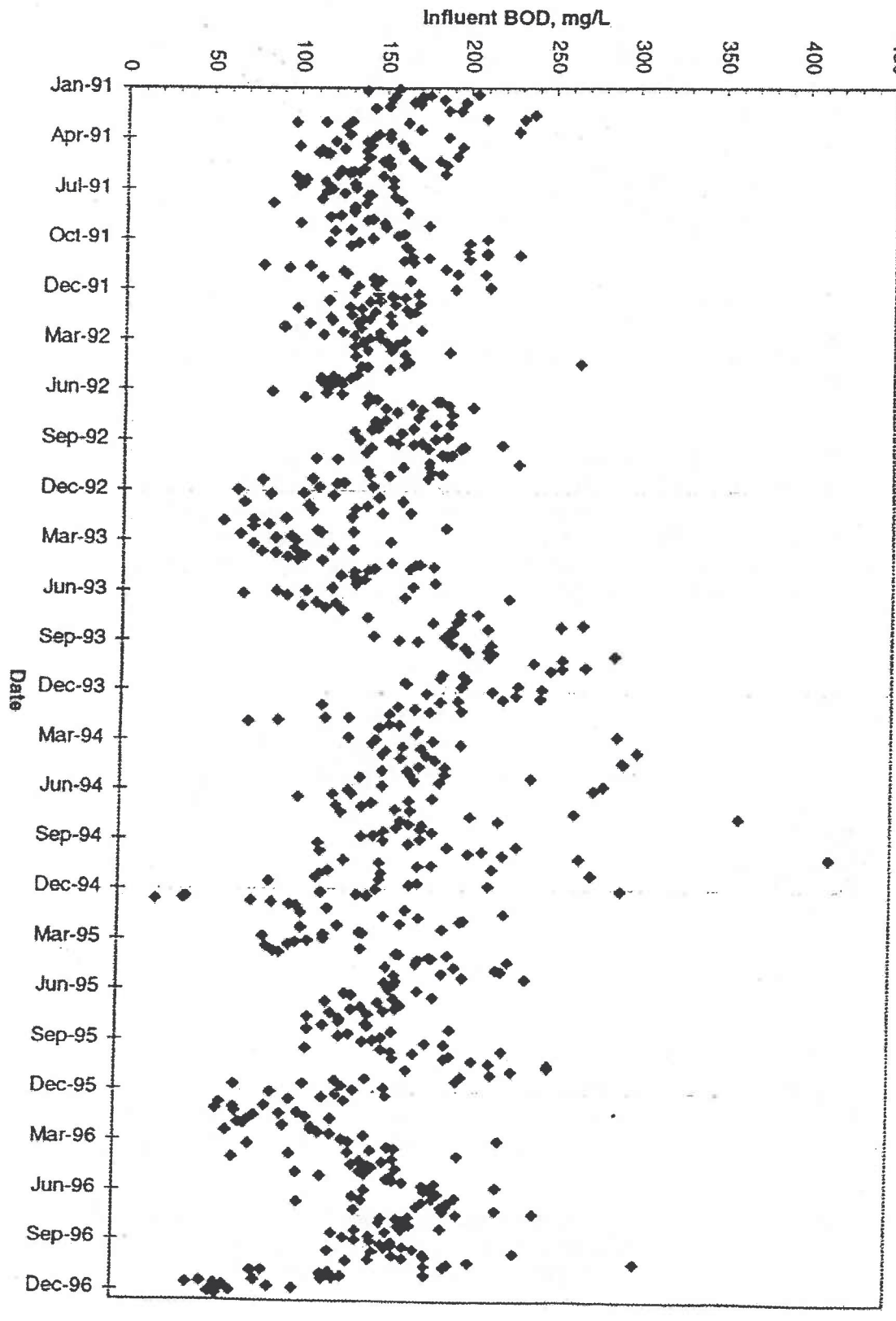

Figure 4-9. Influent BOD Concentration

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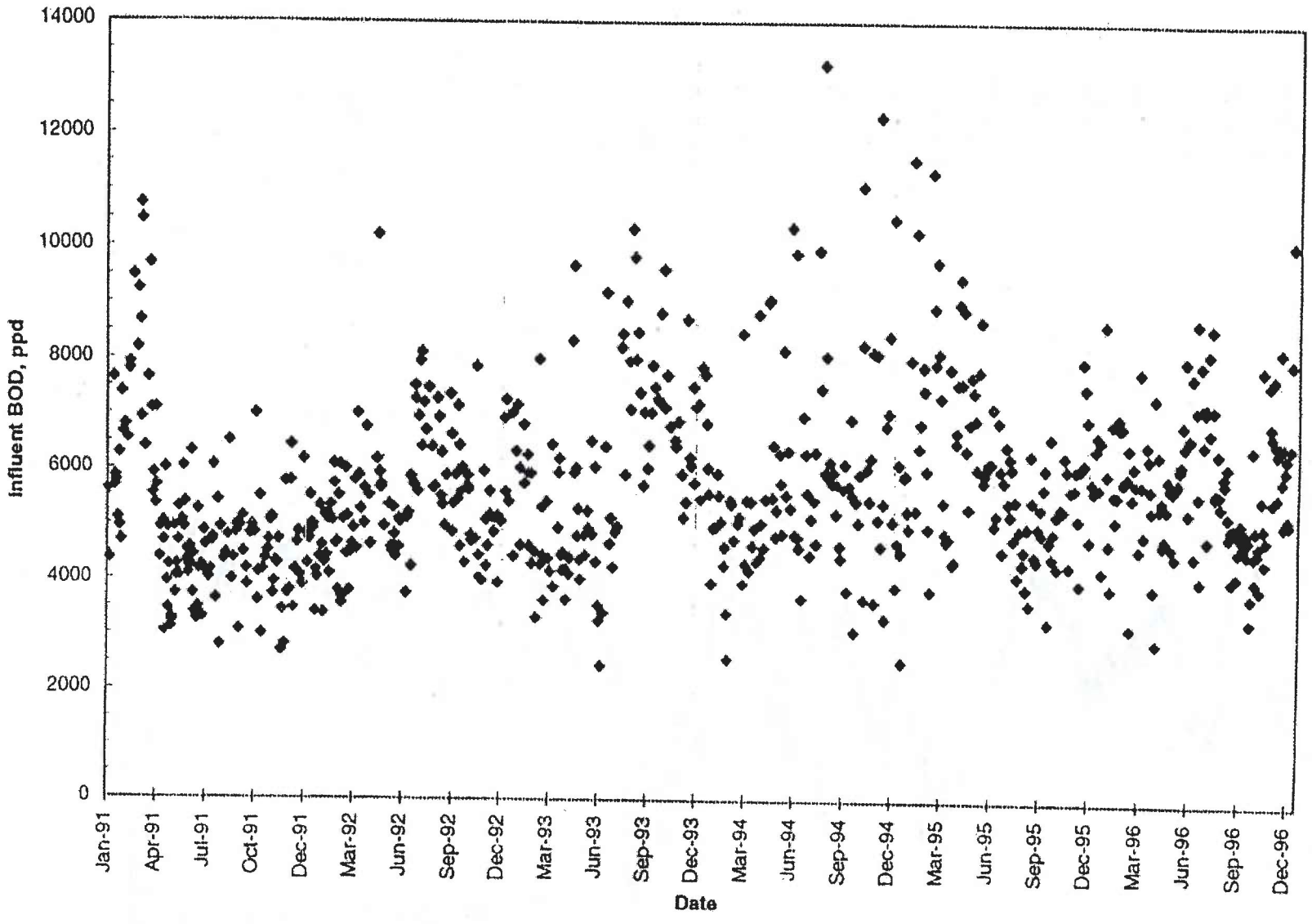

Figure 4-10. Daily Influent BOD Loading

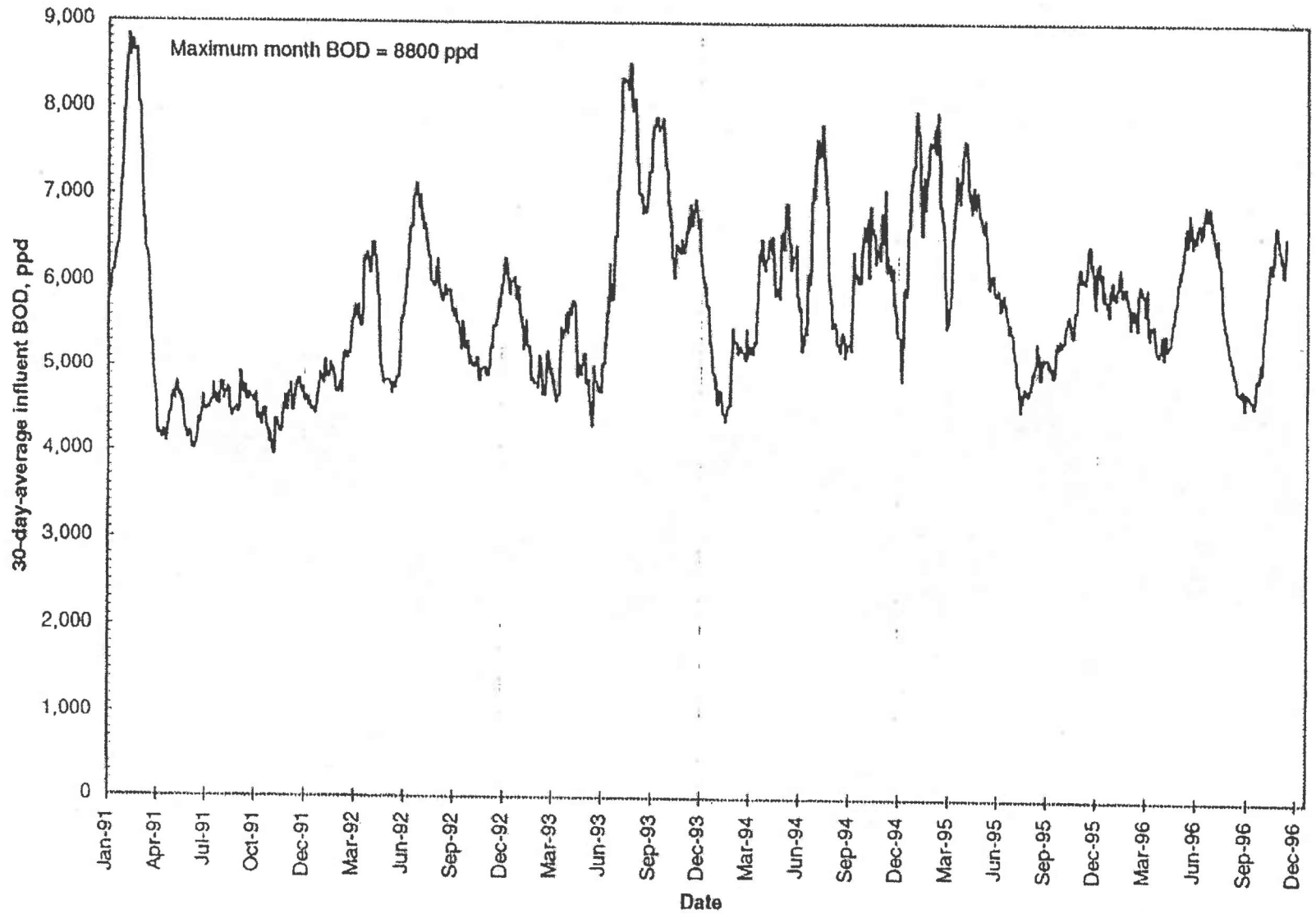

Figure 4-11. 30-Day-Average Influent BOD Loading

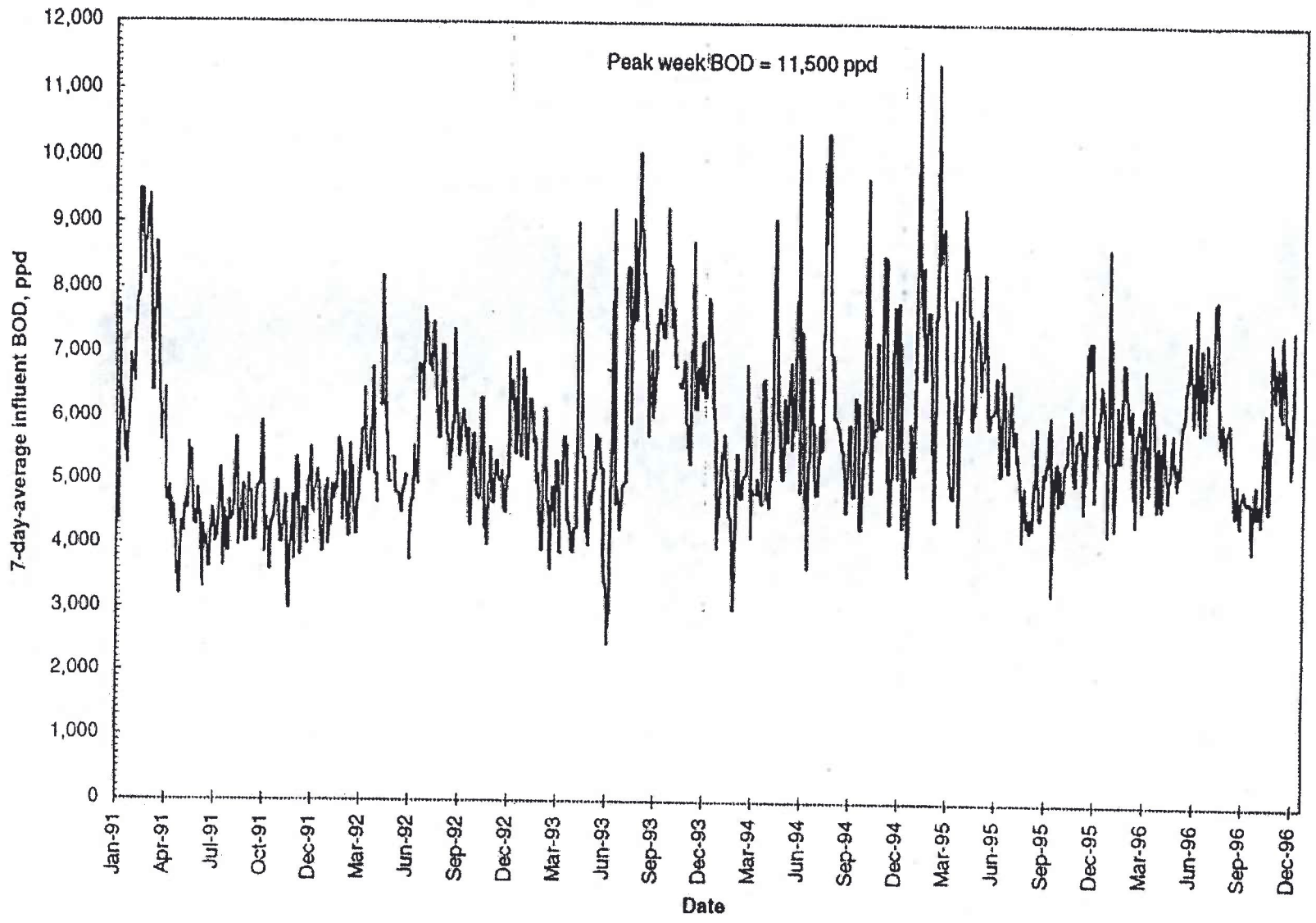

Figure 4-12. 7-Day-Average Influent BOD Loading

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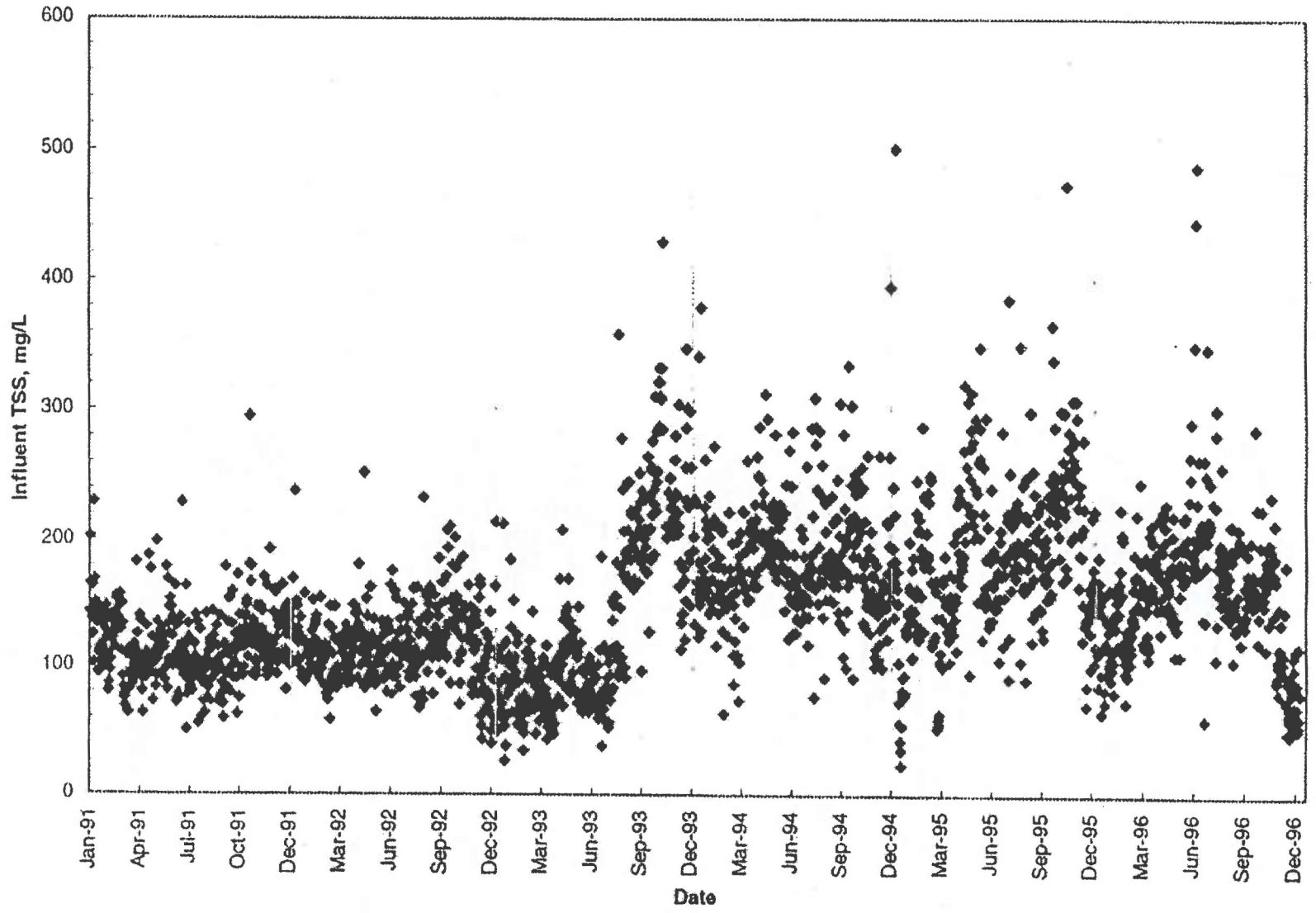

Figure 4-13. Influent TSS Concentrations

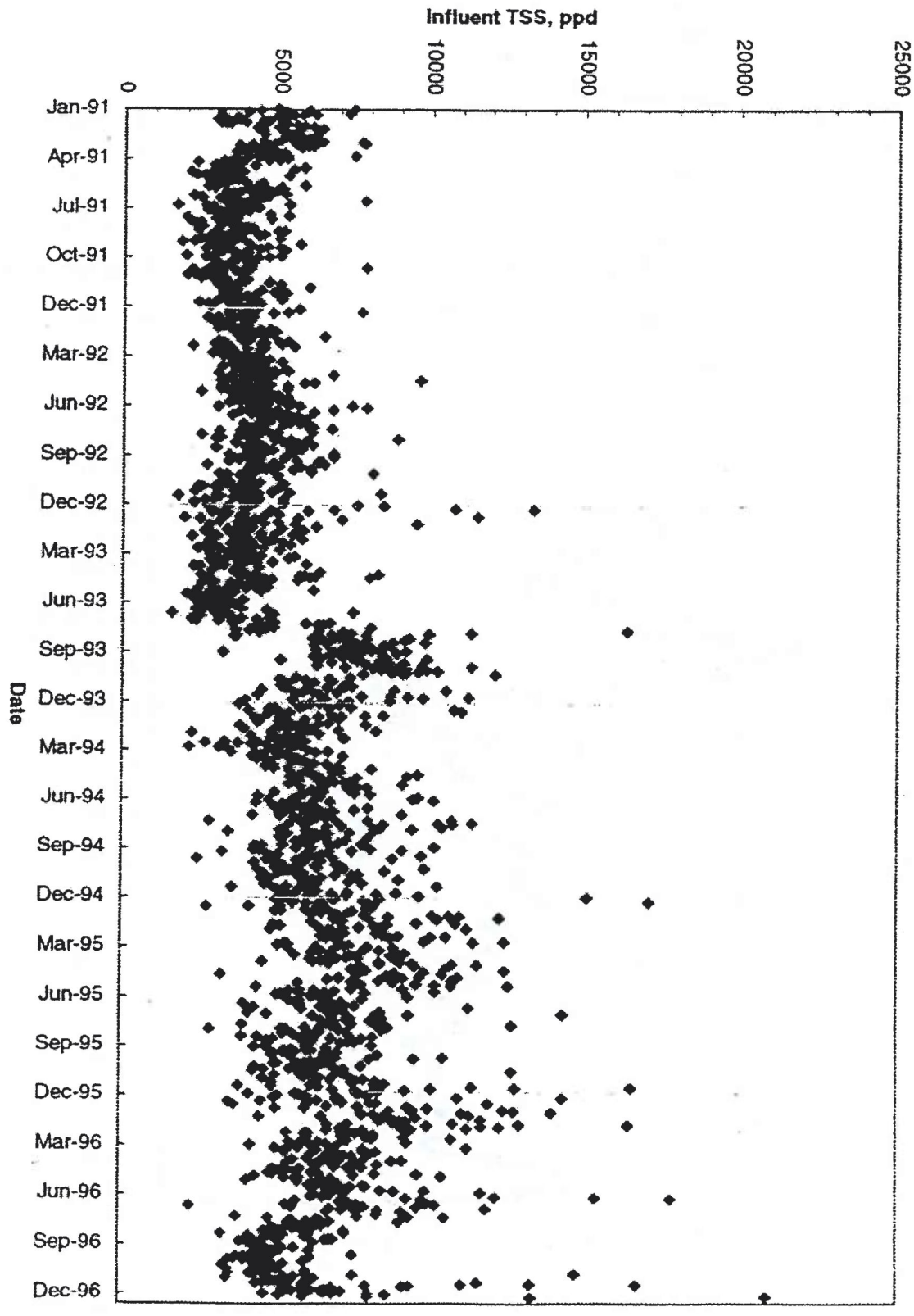

Figure 4-14. Daily Influent TSS Loading

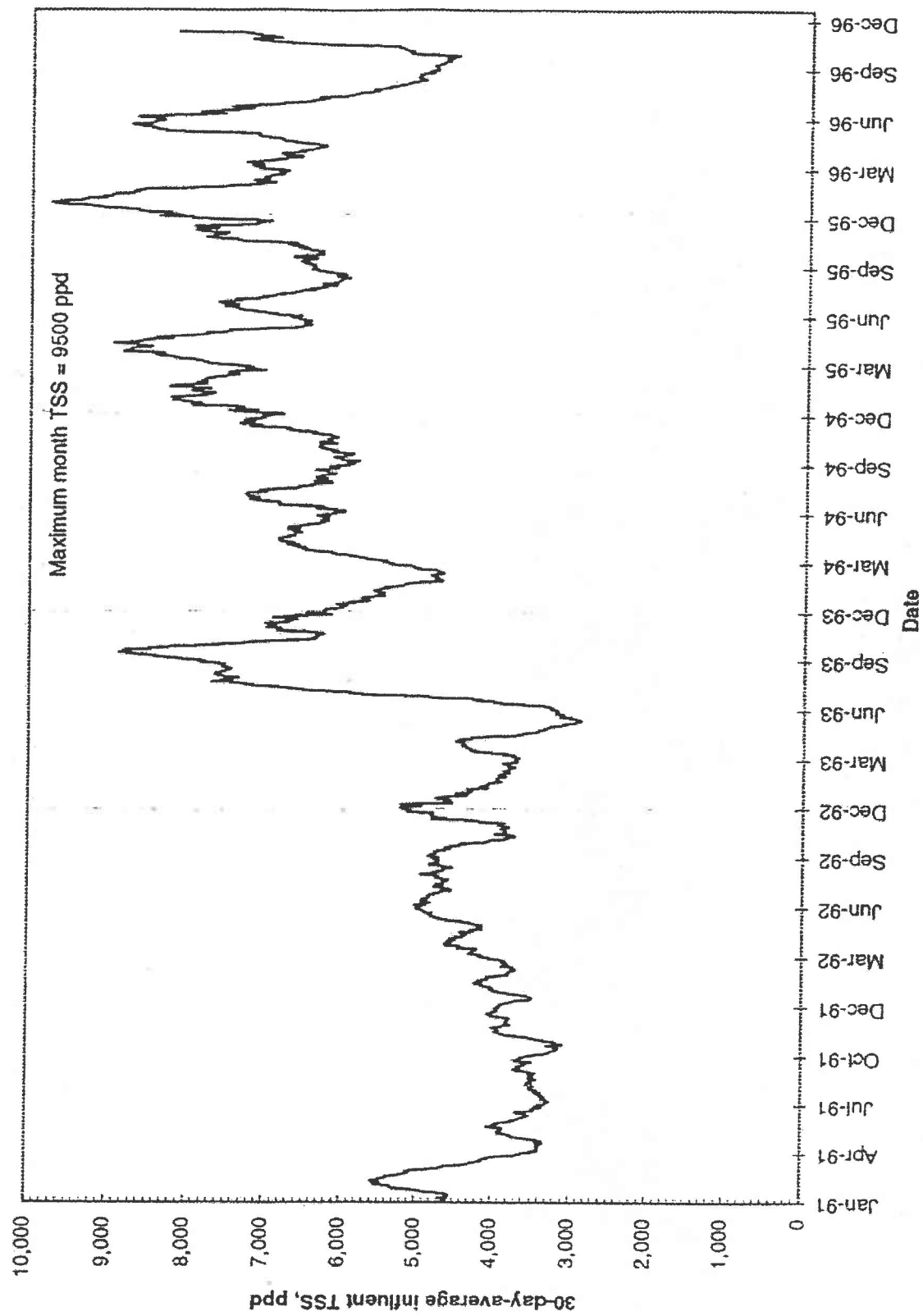

Figure 4-15. 30-Day-Aver Average Influent TSS Loading

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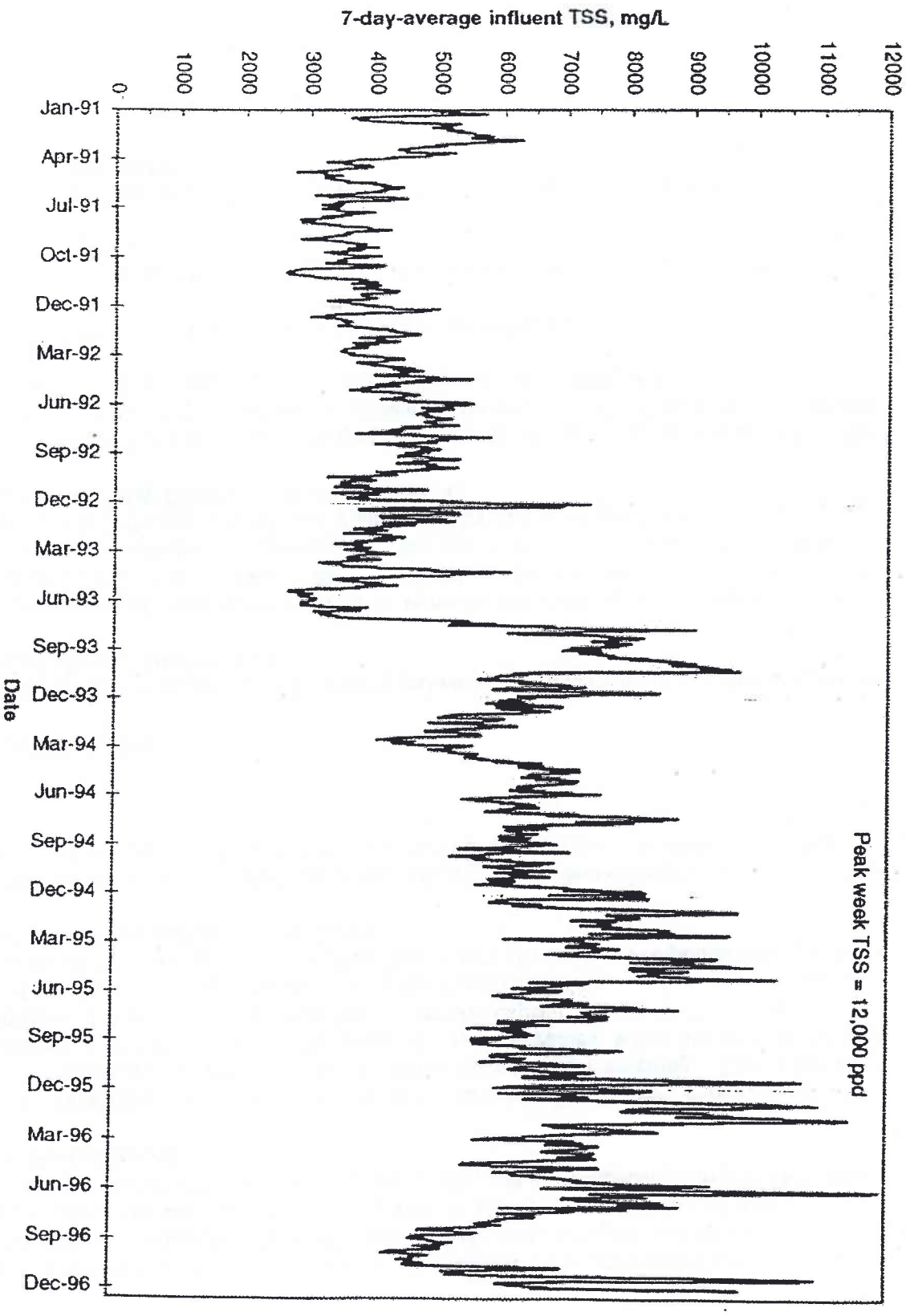

Figure 4-16. 7-Day-Average Influent TSS Loading

Nitrogen in the raw wastewater is primarily in the form of ammonia and organic nitrogen. Ammonia typically represents about 60 percent of the total nitrogen load to the plant. Raw wastewater ammonia sampling was not required in past years, so historical plant ammonia loading data are unavailable. Similarly, nitrate, nitrite, and total Kjeldahl nitrogen (TKN) levels have not been measured.

The raw wastewater and plant effluent were sampled five times during June 1991. Table 4-5 summarizes the results of the raw wastewater nutrient sampling. The ammonia-N concentration of 11 mg/L is within the range of expected values, while the total phosphorus concentration (2.7 mg/L) is somewhat lower than the expected range of 4 to 7 mg/L. The limited data allow for rough estimates of plant nutrient loading. Assuming that the data represent average conditions, ammonia-N loading is approximated as 328 pounds per day (ppd) and total phosphorus loading is about 90 ppd.

More recently, the plant has started measuring influent and effluent ammonia and nitrate. This data, presented in Appendix H, confirms average effluent ammonia concentrations of about 11 to 12 mg/L.

BOD and TSS Loads

For WRPs which provide conventional primary and secondary treatment, two loading factors are of principal interest: BOD and TSS.

BOD is a measure of wastewater strength in terms of the quantity of oxygen required for biological oxidation of the organic matter contained in the wastewater. The BOD loading imposed on a treatment plant influences both the type and degree of treatment that must be provided to produce the required effluent quality. Unless otherwise noted, all references to BOD in this report are 5-day BOD tests incubated at 20° C.

TSS is a measure of the quantity of suspended material contained in the wastewater. The quantity of TSS removed during treatment influences the sizing of sludge handling and disposal processes, as well as the effectiveness of disinfection with ultraviolet light.

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

- Secondary process sizing. The design of a secondary process is based on the BOD load.
- Aeration system design. The capacity of the aeration system is determined by the peak BOD load.

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Table 4-5. Plant Nutrient Loading

Date sample received	Plant flow, mgd	Raw ammonia-N, mg/L	Raw ammonia-N, ppd	Raw nitrite-N, mg/L	Raw nitrate-N, mg/L
6/13/91	3.8	12.26	389	0.022	0.177
6/18/91	4.1	11.42	390	0.02	0.25
6/20/91	4.1	9.52	326	0.01	0.25
6/26/91	4	11.76	392		0.14
6/28/91	4	10.75	359	0.012	0.275
Number	5	5	5	4	5
Max =	4.1	12.26	392	0.022	0.275
Min =	3.8	9.52	326	0.01	0.14
Avg =	4.00	11.14	371	0.02	0.22

Date sample received	Raw TKN, mg/L	Raw total phosphorus, mg/L[a]	Raw total phosphorus, ppd	Raw ortho phosphorus, mg/L[a]	Raw ortho phosphorus,
6/13/91	18.36	2.86	91	0.034	1
6/18/91	11.53	2.69	92		
6/20/91	9.69	2.14	73	0.05	2
6/26/91	18.48	2.64	88	1.59	53
6/28/91	18.7	2.93	98	1.73	58
Number	5	5	5	4	4
Max =	18.7	2.93	98	1.73	58
Min =	9.69	2.14	73	0.034	1
Avg =	15.35	2.65	88	0.85	28

[a] Phosphorus data was collected prior to the ban on high phosphorus detergents.

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

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This section evaluates current plant BOD and TSS loading.

BOD and TSS Records. As with plant flows, it is important to identify any limitations or irregularities in the historical data. For the Grants Pass BOD and TSS records, it is important to note that a new composite influent sampler was installed in August 1993. Measured TSS values increased substantially after the installation of the new sampler.

Monthly Plant Loading. Table 4-6 summarizes influent plant BOD and TSS concentrations and loads for January 1991 through December 1996. As discussed above, BOD and TSS values reported before September 1993 are not representative of actual plant loading.

For the current sewer service area population of about 25,000, the average BOD load of 6,000 ppd corresponds to a unit load of 0.24 pounds per capita per day (pcd). This is consistent with the textbook value of 0.2 pcd. Dividing the average TSS load of 7,000 ppd by the current population results in a unit load of 0.28 pcd—somewhat higher than the textbook value of 0.2 pcd.

Peak Plant Loading. Daily influent BOD concentrations and loads are shown in Figures 4-9 and 4-10, respectively. Figure 4-11 presents the 30-day running average influent BOD load. From this plot, the maximum month plant BOD load is estimated at 8,800 ppd. The 7-day running average BOD load was used to estimate the peak week BOD of 11,500 ppd (Figure 4-12).

Figures 4-13 through 4-16 are similar plots for influent TSS. From these, the maximum month and peak week BOD loads are estimated at 9,500 ppd and 12,000 ppd, respectively.

PEAKING FACTORS

A peaking factor is the relationship between current average dry weather flow (ADWF) and higher flow conditions. This relationship is characteristic of hydraulic conditions in the system, and topographic and I/I characteristics. For example, in Table 4-4, data shows that the ADWF was 4.5 mgd in 1996, while the PWWF was 28 mgd. From these numbers we calculate a ratio of 28/4.5, or a PWWF peaking factor of 6.22. Table 4-7 contains the peaking factors calculated from the 1996 data in Table 4-4 for Grants Pass and Harbeck-Fruitdale. Lower peaking factors are estimated for Redwood and North Valley, based on newer, tighter sewer construction. These factors are also presented in Table 4-7.

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Table 4-6. Monthly Plant Loading

Month	Flow, mgd	Influent wastewater TSS mg/L	TSS ppd	BOD mg/L	BOD ppd	Month	Flow, mgd	Influent wastewater TSS mg/L	TSS ppd	BOD mg/L	BOD ppd
Jan-91	4.07	138	4,716	170	5,843	Jan-94	3.82	194	6,158	192	6,063
Feb-91	5.03	127	5,293	193	8,002	Feb-94	3.95	169	5,541	142	4,662
Mar-91	6.14	101	5,027	145	7,686	Mar-94	3.67	158	4,812	176	5,323
Apr-91	3.97	110	3,616	138	4,483	Apr-94	3.36	194	5,445	184	5,241
May-91	3.61	123	3,693	155	4,683	May-94	3.73	216	6,744	200	6,291
Jun-91	3.95	113	3,716	127	4,196	Jun-94	4.18	193	6,708	194	6,794
Jul-91	4.11	98	3,361	129	4,484	Jul-94	4.41	163	6,009	140	5,203
Aug-91	4.05	103	3,498	135	4,802	Aug-94	4.36	199	7,237	204	7,459
Sep-91	3.82	111	3,517	143	4,496	Sep-94	4.05	184	6,218	162	5,537
Oct-91	3.20	132	3,491	179	4,662	Oct-94	3.43	210	6,008	208	5,861
Nov-91	3.39	123	3,475	146	3,958	Nov-94	4.63	169	6,332	168	6,080
Dec-91	3.64	127	3,827	158	4,693	Dec-94	4.65	174	6,699	181	7,069
Jan-92	3.72	125	3,906	154	4,644	Jan-95	8.06	131	7,219	90	5,138
Feb-92	4.27	114	4,046	138	4,857	Feb-95	5.45	181	7,924	174	7,723
Mar-92	4.41	103	3,779	130	4,718	Mar-95	6.97	150	7,849	126	7,520
Apr-92	4.37	115	4,193	152	5,623	Apr-95	5.66	163	7,661	131	6,087
May-92	4.43	122	4,518	170	6,452	May-95	4.50	230	8,596	194	7,402
Jun-92	4.79	109	4,377	121	4,843	Jun-95	4.48	200	7,474	174	6,653
Jul-92	4.96	120	4,963	144	5,935	Jul-95	4.57	181	6,876	151	5,753
Aug-92	4.73	117	4,629	170	6,781	Aug-95	4.37	196	7,135	134	4,848
Sep-92	4.45	128	4,745	159	5,931	Sep-95	3.96	185	6,071	147	4,944
Oct-92	3.97	149	4,922	178	5,688	Oct-95	3.48	223	6,446	174	5,056
Nov-92	3.79	124	3,849	165	5,080	Nov-95	3.09	256	6,630	220	5,529
Dec-92	5.41	98	4,133	124	4,900	Dec-95	6.00	164	7,789	136	6,043
Jan-93	6.69	91	4,946	117	6,276	Jan-96	8.16	125	8,298	102	6,066
Feb-93	6.29	86	4,311	117	5,594	Feb-96	8.36	136	9,216	91	5,947
Mar-93	5.43	87	3,890	106	4,754	Mar-96	5.44	156	7,089	130	5,696
Apr-93	5.81	78	3,712	105	4,847	Apr-96	5.07	172	7,148	135	5,464
May-93	4.47	120	4,481	149	5,703	May-96	4.49	171	6,397	143	5,275
Jun-93	4.33	87	3,152	134	4,808	Jun-96	4.40	215	7,870	178	6,576
Jul-93	4.79	84	3,211	130	4,916	Jul-96	4.49	219	8,237	173	6,577
Aug-93	4.92	148	6,085	189	7,749	Aug-96	4.14	183	6,296	179	6,296
Sep-93	4.61	198	7,636	190	7,373	Sep-96	3.87	158	5,104	153	4,912
Oct-93	4.24	231	8,126	218	7,818	Oct-96	3.42	168	4,779	165	4,589
Nov-93	3.32	256	7,131	244	6,719	Nov-96	4.50	153	5,329	165	6,238
Dec-93	4.11	207	6,917	201	6,585	Dec-96	11.89	85	8,093	76	6,532
Maximum								256	9,216	244	7,818
Minimum								85	4,779	76	4,589
Average								183	7,000	164	6,000

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Table 4-7. Calculated Wastewater Flow Peaking Factors

Description	Peaking Factor	
	GP/F-H	Red/NV
ADWF	-	-
Average annual flow	1.13	1.13
AWWF	1.27	1.27
MMDWF	1.49	1.49
MMWWF	2.22	2.00
Peak week flow	3.33	2.60
Peak day flow	4.44	3.20
PWWF	6.22	5.00

WASTEWATER FLOW AND LOAD PROJECTIONS

Flow and load projections provide a basis for the design of future facilities. Future flows and loads reflect expected population growth and any reduction in I/I due to improvements in the wastewater collection system.

Wastewater Flows

Flow to the Grants Pass WRP is expected to increase for three distinct reasons:

1. Population growth in the existing sewerage service area will generate more flow to the plant.

2. The Redwood Sanitary Sewer Service District treatment plant is expected to increase flow by about 10 percent to the Grants Pass WRP for treatment by the winter of 2000.

3. The North Valley (Merlin) residential area, which currently depends on on-site septic systems, may be sewered in the next few years. This could constitute another new source of sanitary flow to the Grants Pass WRP.

Population Growth in Grants Pass and Harbeck-Fruitdale. The population of Grants Pass and Harbeck-Fruitdale will continue to experience steady growth of 1.5 percent and 1.6 percent, respectively. Therefore, the 1998 population of 25,101 is predicted to increase to 34,959 by year 2020.

Redwood District. The Redwood District's population is currently estimated at 4,905. Over the next 20 years, this area is expected to experience a 3.1 percent growth, which will almost double the current population. This growth has put pressure on the District's wastewater treatment plant. As houses have been constructed closer and closer to the plant, complaints about odors and other concerns have prompted Josephine County's decision to decommission the Redwood plant. In the fall of 2000, sewage from the Redwood District will be pumped to the Grants Pass WRP for treatment.

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North Valley. Portions of the North Valley area may be sewered in the next 10 years, adding its sewage flow to the Grants Pass WRP. Meanwhile, the population of the area will be steadily growing. At a 2 percent annual growth rate, the North Valley's 1,200 population is expected to grow to 1,900 by the year 2020. At this time it is uncertain how much of the wastewater may be conveyed to Grants Pass.

Flow Projections. Flow projections are based on existing wastewater flow plus a typical flow per capita assumed for all future connections. These future flow factors in gallons per capita per day (gpcd) are shown in Table 4-8 and were also used in the Redwood Facilities Plan. Although the existing wastewater flow per capita is relatively high, the per capita flow assumed for future connections is typical for newer construction. If new sewer pipes are properly installed and inspected, these pipes should not allow the amount of I&I flow that the sewers currently experience. Table 4-9 shows the wastewater flow projections for years 2010 and 2020.

Table 4-8. Future Flow Factors

Flow Type	Contribution (gpcd)
Average Flow	105
Maximum Month Dry Weather Flow (MMDWF)	100
Maximum Month Wet Weather Flow (MMWWF)	130
Peak Day	275
Peak Wet Weather Flow	360
Summer Average	85
Winter Average	125

Table 4-9. Wastewater Flow Projections

	YEAR					
	1999		2010		2020	
	Calculated (mgd)	Calculated (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)
Summer, Max month	7.3	173	8.4	160	9.4	150
Winter, Max month	10.8	256	12.2	233	13.5	216
Peak Day	21.5	514	24.6	471	27.5	438
PWWF	30.1	715	34.0	652	37.8	603
Summer Avg	5.0	120	6.0	113	6.8	109
Winter Avg	6.4	155	7.80	149	9.1	145
Annual Avg	5.7	137	6.9	131	8.0	127

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To develop the wastewater flow projections presented in Table 4-9, the following process was used:

- Wastewater flows for the Redwood and North Valley service areas were calculated using their respective 1997 population data (Table 2-4) multiplied by the future flow factors (Table 4-8).
- Wastewater flows from the Redwood and North Valley service areas were added to the current wastewater flows (Table 4-4). The resultant values are presented in Table 4-9 as 1999 calculated flows.
- Future additional wastewater flows from the service population were calculated using population growth data (Table 2-4) multiplied by the future flow factors (Table 4-8).
- Future additional wastewater flows were added to the 1999 calculated flows. The resultant values are presented in Table 4-9 as 2010 and 2020 design flows.

Organic and Solids Loading

Another important design parameter for wastewater facilities is the load in pounds per day of BOD and TSS. For future connections, wastewater load projections are based on the existing wastewater load plus a typical load per capita. A summary of existing and future pollutant load estimates is shown in Table 4-10.

Table 4-10. Organic and Solids Loading
(pounds per day)

	YEAR		
	1999	2010	2020
Average BOD	7,490	9,400	11,400
Maximum Monthly BOD	10,985	13,700	16,800
Average NH_3-N	327	390	450
Maximum Monthly NH_3-N	636	760	870
Average TSS	8,564	10,500	12,700
Maximum Monthly TSS	11,623	14,300	17,300

Nutrients

Ammonia-nitrogen (NH_3-N) and phosphorous are the two primary nutrients of concern. According to current plant data, the average influent NH_3-N is 327 pounds per day (ppd), while the maximum monthly influent NH_3-N is 636 ppd. As the population increases, the NH_3-N levels will also increase. Table 4-10 has both the current and projected NH_3-N levels. The future average NH_3-N for year 2020 is predicted to be 450 ppd and the maximum monthly is 870 ppd.

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CHAPTER 5

WASTEWATER PLANNING CONSIDERATIONS

Wastewater and the residuals generated during the treatment of wastewater must be disposed of in a way that protects public health and preserves the quality of surface water and groundwater resources. The biological, chemical, and physical characteristics of the wastewater and the receiving water body are other important factors that can dictate both the degree and type of treatment required.

This chapter presents general design criteria for wastewater treatment and disposal, outlines a basis for cost estimating, and examines the methods for conducting economic and noneconomic comparisons.

WASTEWATER DISPOSAL CRITERIA

The two primary disposal criteria are that the public health be protected at all times and that none of the current or future beneficial uses of the Rogue River be impaired. Governmental regulations concerned with discharge requirements are devised to ensure that the public health and beneficial uses are protected. The nature of the desired beneficial uses has an important impact on the strictness of the regulations.

Regulation of water quality and waste discharges is under the control of the Oregon Department of Environmental Quality (DEQ) at the state level and the United States Environmental Protection Agency (EPA) on a national scale. The DEQ has the responsibility for establishing and enforcing water quality standards, as well as developing effluent limits for inclusion into National Pollutant Discharge Elimination System permits and water quality certifications.

DESIGN CRITERIA

Design flows and loads are presented in Chapter 4. Water quality issues and discharge requirements are presented in Chapter 6. The design criteria for the facilities incorporated in the alternative plans developed in later chapters are discussed below.

Design Period

Selection of the design period is affected by several factors, including population projections, future development, and anticipated discharge limitations. A balance between the service life and the economic life of the facilities should be reached. Alternative plans for the expansion

of the wastewater treatment plant are based on providing service for 20 years. Allowing for design and construction, initial plant improvements identified in this report would be placed in service during the year 2000. Therefore, a 20-year planning period would correspond to year 2020.

Wastewater Treatment Plant Design

While many factors influence the design of wastewater treatment plants, the two most important are the required capacity and desired degree of treatment. The plant must have sufficient capacity to handle future peak hydraulic flows and organic loads. The degree of treatment necessary is governed by the discharge permit requirements. The sections below address other important design considerations.

Flexibility. Flexibility in a wastewater treatment plant implies that portions of unit processes or entire unit processes can be bypassed with little effect on effluent quality. Operators should be able to take units out of service for maintenance without overloading the units remaining in service. Flexibility in design means that future treatment units can be added to the existing units with minimum interference in plant operations.

Reliability. Reliability of unit processes is attained by designing the unit processes with capacity to treat peak flows and loads. Pumps and other mechanical equipment are sized conservatively to achieve reliability. Redundancy is important to ensure that failure of treatment units or mechanical equipment does not seriously affect effluent quality. Unit processes that are less complex and more stable are favored over those processes that require significant operator attention.

Automation. Automated process controls are desirable whenever consistent, reliable operation is required and a cost saving over manual operation is possible. Complex processes are much more suitable for automation than simple processes, because there is a greater potential for reducing operator workload.

Human Factors. While flexibility, reliability, and automation all contribute greatly to the successful operation of a wastewater treatment plant, providing a suitable working environment may be even more important. The design of a treatment plant must look at the plant from an operational standpoint. Important human factors include convenient access to equipment and valves, sufficient lighting, adequate ventilation, proper odor control, noise control, satisfactory health and safety arrangements, conveniently located washdown areas, and adequate locker and shower facilities. Control of the plant should be consolidated in as few locations as possible to reduce operator fatigue. Overall plant layout and design of buildings and process units all influence the ease of operation of a wastewater treatment plant.

EVALUATION CRITERIA

The evaluation of alternative plans must consider both economic and noneconomic factors. This section describes the process for comparing alternative plans using both economic and noneconomic criteria.

Economic Evaluation

The economic evaluation is based on a comparison of present worth values for each alternative. Present worth is affected by maintenance requirements, energy costs, land purchases, and salvage value. For this report, the present worth calculations use a discount rate of 8.75 percent and an economic life of 20 years. Facilities that have an economic life other than 20 years will incur a salvage value. Salvage values are estimated with a straight-line depreciation model.

Construction costs for each alternative are based on preliminary plant layouts. Cost estimates were prepared using construction costs of similar treatment plants, cost curves, and EPA design manuals.

Annual costs are comprised of both fixed and variable expenses. Fixed expenses include interest and depreciation. Variable expenses are primarily operation and maintenance (O&M) costs. O&M costs vary with the amount of wastewater treated and the type of treatment provided. O&M costs are based on a labor rate of $30 per hour and electrical costs of $0.05 per kilowatt hour.

Precision of Cost Estimates. The degree to which alternatives are developed influences the precision of the cost estimates. The cost estimates presented in this report are order-of-magnitude estimates. Order-of-magnitude estimates are approximate and are obtained with cost-capacity curves, scaling factors, and ratios. Order-of-magnitude estimates are expected to be accurate within +50 percent to -30 percent.

Basis for Costs Over Time. The increased cost of equipment, material, and labor over time will have a corresponding effect on the costs presented in this report. The relative cost differential between alternative plans, however, should remain valid.

Costs undergo long-term changes in relation to inflation, interest rates, and the national economy. The standard measure of these long-term cost increases is the *Engineering News-Record* (ENR) construction cost index. It is computed from average costs of labor and materials and based on an ENR index of 100 in 1913. Figure 5-1 shows the increase in the ENR index since 1976. The costs in this report are based on an ENR construction cost index of 5839. The ENR cost index reached 5850 by the end of 1997 and approached 5950 by the end of 1998.

Engineering, Administration, and Contingencies. The cost of engineering services for wastewater treatment projects includes a predesign study, surveying, a geotechnical investigation, preparation of contract drawings and specifications, construction management, start-up services, and preparation of operation and maintenance manuals. Engineering costs typically range from

13 to 20 percent of the contract cost. Actual engineering costs depend on the size of the project, the complexity of the design, and the extent that existing facilities are remodeled. Engineering costs for the Grants Pass Water Restoration Plant project are estimated at 16 percent.

The City of Grants Pass will incur administrative costs during the construction of any project. The administrative activities include planning, budgeting, contract handling and processing, legal services, and contacts with regulatory agencies. Administrative costs to the City of Grants Pass are estimated to be 4 percent of the contract cost.

Unforeseen difficulties, such as adverse construction conditions, may tend to increase the final construction cost. An allowance of 30 percent is applied to the contract cost to allow for contingencies. The total allowance for engineering, administration, and contingencies is 50 percent of the base construction cost.

Land Acquisition. Land purchase prices for each alternative were estimated from county tax assessor appraisal records or other site-specific information.

Noneconomic Evaluation

A comparison of the noneconomic factors is usually somewhat difficult. There is no easy way to quantify noneconomic considerations and compare them on an equivalent basis. Noneconomic factors include treatment plant performance, environmental issues, and ease of implementation.

Performance. The overall performance of a treatment plant is determined by many factors, including treatment effectiveness, operability, reliability, flexibility, and energy consumption. Any acceptable treatment alternative must produce effluent of the quality required by the wastewater discharge permit. Each alternative must have the capacity to treat expected peak flows and loads. In general, alternatives which are less complex are more desirable. Alternatives with lower energy requirements or those that produce energy are preferred. The recommended alternative must be easily operable. It should be reliable and have sufficient redundancy so that effluent quality is not seriously affected during a system failure. The plant should have enough flexibility to adapt to future conditions such as more stringent discharge requirements and increased flows and loads.

Environmental. Environmental criteria include air and water quality, noise, traffic, aesthetics, land use compatibility, and reclamation potential. The major air quality concern at wastewater treatment plants is odor control. Water quality is governed by the limits set forth in the discharge permit. Noise is an issue during both construction and daily operation of equipment. Traffic is a concern during construction and can also become a long-term consideration if sludge is hauled by truck to distant disposal sites. The aesthetics of a treatment plant are particularly important if the plant is visible from residential or recreational areas. Land use compatibility pertains to the desire to locate a wastewater treatment plant near industrial or commercial developments and away from residential areas. Reclamation potential refers to the ability to use

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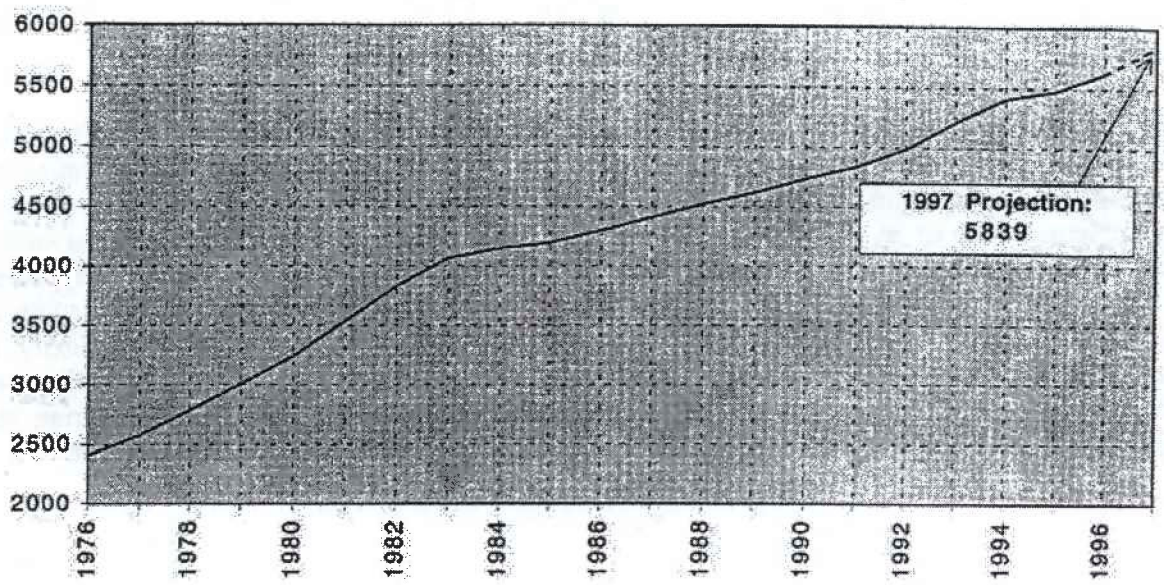

Figure 5-1. ENR Construction Cost Index
(20-city average)

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sludges as fertilizer on agricultural land and effluent as irrigation water. The reuse of effluent for irrigation is particularly attractive in areas that have arid climates or experience frequent water shortages.

Implementation. The ability to implement an alternative is as important as the technical evaluation. A plan that cannot be implemented has no value. It is difficult to determine the feasibility of implementing a plan with a normal engineering analysis. Political factors influence the implementation of an alternative. The recommended plan must be acceptable to the public, local governmental agencies, and elected officials. The recommended plan must also be approved by local health agencies, the DEQ, and agencies that provide economic assistance. In addition, it must be constructable. Construction considerations include the size and complexity of structures, the type of soils on site, groundwater levels, and the need to keep existing unit processes in operation during construction. The control of liability is a concern during any project. The areas of sludge management and effluent reuse require particular attention.

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CHAPTER 6

WASTEWATER TREATMENT CRITERIA

The City of Grants Pass has long recognized that the Rogue River represents a vital asset for the city and surrounding community. The Rogue River provides both an amenity for the local residents and, through tourism, a significant contribution to the local economy. Protecting and enhancing this resource has always been one of the city's priorities. The community is well aware that the river's fish, wildlife habitat, scenic, and recreational opportunities are closely tied to water quality.

This chapter presents a summary of the Rogue River's water quality and identifies the most critical water quality aspects. We then set forth anticipated treatment criteria for the Grants Pass Water Restoration Plant (WRP). This effluent criteria is designed to protect the river's water quality and to secure the benefits that the river offers the community.

REGULATORY AUTHORITY

The unique characteristics of the Rogue River were recognized by congress in 1968 in the "Wild and Scenic Rivers Act," Public Law 90-542. This legislation designated certain selected rivers as possessing outstanding scenic, natural, and recreational values. These rivers would be preserved in their free-flowing condition and their water quality protected. The Rogue National Wild and Scenic River starts at the mouth of the Applegate River, about 5 miles downstream from the Grants Pass WRP. It extends about 84 miles downstream to the Lobster Creek Bridge.

The Bureau of Land Management and the U.S. Forest Service are the designated administrators of the Rogue National Wild and Scenic River. In 1972 these agencies published a joint plan for the development, operation, and management of the Rogue National Wild and Scenic River.[1] This management plan outlines responsibilities for coordination between the State of Oregon and federal and local governments. The responsibility for water quality is addressed as follows:

> "Maintenance of water quality and implementation of water quality standards on the Rogue River is under the jurisdiction of the Oregon Department of Environmental Quality. Water quality and waste treatment standards for the river were adopted by that Department on July 24, 1969, and should be instrumental in restoring the river to an unpolluted condition."

Because of the Department of Environmental Quality's (DEQ's) administrative responsibility, we based the water quality evaluation on the standards set forth in Chapter 340 of the Oregon Administrative Rules (OARs). The general policy followed in these rules is one of antidegradation of surface waters (OAR 340-41-026).

> "The purpose of the Antidegradation Policy is to guide decisions that affect water quality

such that unnecessary degradation from point and nonpoint sources of pollution is prevented, and to protect, maintain, and enhance existing surface water quality to protect all existing beneficial uses. The standards and policies set forth in OAR 340-41-120 through 962 are intended to implement the Antidegradation Policy."

The Oregon Environmental Quality Commission may designate certain high-quality waterbodies in park, wilderness, and scenic areas as Outstanding Resource Waters and impose special requirements to protect these waterbodies. Information related to these waterbodies is evaluated by the DEQ. If a waterbody is designated as Outstanding Resource Waters, the DEQ is then responsible for developing a management plan to preserve the water quality in these areas.

WASTE LOAD ALLOCATION PROCESS

The DEQ has reviewed water quality data for the Rogue River to assess compliance with Oregon's water quality standards. It has been determined that at certain times the river is not in compliance with water quality criteria, therefore beneficial uses are not being supported. The Rogue River has been listed as water quality limited under Section 303(d) of the Clean Water Act.

The water quality parameters of concern, as indicated on the 303(d) listing, consisted of summer temperature and high bacteria levels for the reach including Grants Pass. Downstream from the confluence with the Applegate River, listing parameters consisted of pH during all seasons, summer temperature, mercury, and bacteria.

Recently, Coho Salmon has been listed as a threatened species. Rogue River water quality must support passage, spawning, and rearing for Coho.

The DEQ has determined that the Rogue River is a high priority concern for implementing management strategies to attain compliance with water quality standards. The first step will be to develop the total maximum daily load (TMDL) pollutant loading allocation. The TMDL will include both point and nonpoint sources. This process is scheduled to start in 1998. The current schedule calls for the TMDLs on the tributaries to be completed first. The TMDL for the Rogue River main stem is not expected to be completed until year 2003.

Following TMDL allocation, new discharge permit requirements will be established for the Grants Pass WRP. A 2- or 3-year implementation period is typically allowed for the plant to be upgraded to meet the new permit requirements.

ROGUE RIVER WATER QUALITY

Rogue River water quality data were obtained through the DEQ from the STORET database, from the USGS Water-Data Reports for Oregon, and from the Grants Pass plant monitoring records. Data from six sampling points were used in this analysis: Lobster Creek Bridge (River Mile 12), Agness (River Mile 30), Robertson's Bridge (River Mile 86), west of Grants Pass

(River Mile 98), Below Gold Hill (River Mile 117), and Dodge Bridge (River Mile 139). A map of the Rogue River showing these locations is presented as Figure 6-1. The period of record used for this analysis was determined by the data available and the construction of Lost Creek Dam in 1977. The dam has the potential to alter water quality, so records before 1977 are not comparable to those after 1977. The period of record used at the station west of Grants Pass was 1980-1982, since no sampling was performed after 1982. The period of record used for the other five sampling points was 1981-1988.

The WRP contributes a very minor portion of the Rogue River's flow at Grants Pass. Flow duration statistics for the 1978-1987 period (published by the USGS in 1990[2]), show that the month of lowest flows was October. During this month the flow exceeded 1220 cubic feet per second (cfs) 95 percent of the time. In other words, there is only a 5 percent chance of the flow being less than 1220 cfs during October. DEQ guidelines recommend considering the 7Q10 flow when evaluating the effects of wastewater treatment plant (WWTP) discharges on river water quality. The 7Q10 flow is defined as the lowest 7-day average flow that occurs once every 10 years. For the Rogue River at Grants Pass, the 7Q10 flow is 888 cfs. (The 7Q10 was verified by contacting the USGS. Unpublished and provision computer analysis for period of record 1978-1997 produced a 7Q10 flow at Grants Pass of 895 cfs.) Current average dry weather flow (ADWF) from the plant is 4.5 million gallons per day (mgd), or 7 cfs. If this discharge occurred during a 7Q10 river flow of 888 cfs, the plant would contribute less than 0.8 percent of the river's flow. The projected future ADWF is 8 mgd (12.4 cfs), or 1.4 percent of the river's flow.

The situations outlined above show that even under worst-case scenarios the plant contributes less than 1.5 percent of the Rogue River flow at Grants Pass. This relatively small volume is the main reason why the effect of the WRP effluent on Rogue River water quality appears to be so limited.

As required by Section 303 (d) of the Clean Water Act, the DEQ recently published a list of all streams that do not comply with applicable water quality standards. These waterways are referred to as water quality limited. The Rogue River is listed as water quality limited for the following parameters:

- Fecal coliform year-round.
- Summer dissolved oxygen (DO).
- Summer temperature.
- pH year-round.
- Mercury in tissue.

The temperature listing may require Grants Pass to participate in the development of a temperature management plan for the Rogue River basin. It should be noted that the DO listing is only for segments of the Rogue well downstream of Grants Pass (from the mouth to the confluence with the Illinois River.)

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Water quality listing for pH is due to swings in pH typically caused by excessive algal growth or excessive attached periphyton. Growth of algae and periphyton is typically in response to excessive nutrients.

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

- Flow modification. More data is needed to determine if stream flow alterations are a problem.
- Nutrients. More data is needed to evaluate nutrients and algae growth.
- Sediment. More data is needed to evaluate sediment.

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

Biological Criteria

This rule provides the state with a basic foundation for protecting biological communities dependent on water quality.

Biological Standards. 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).

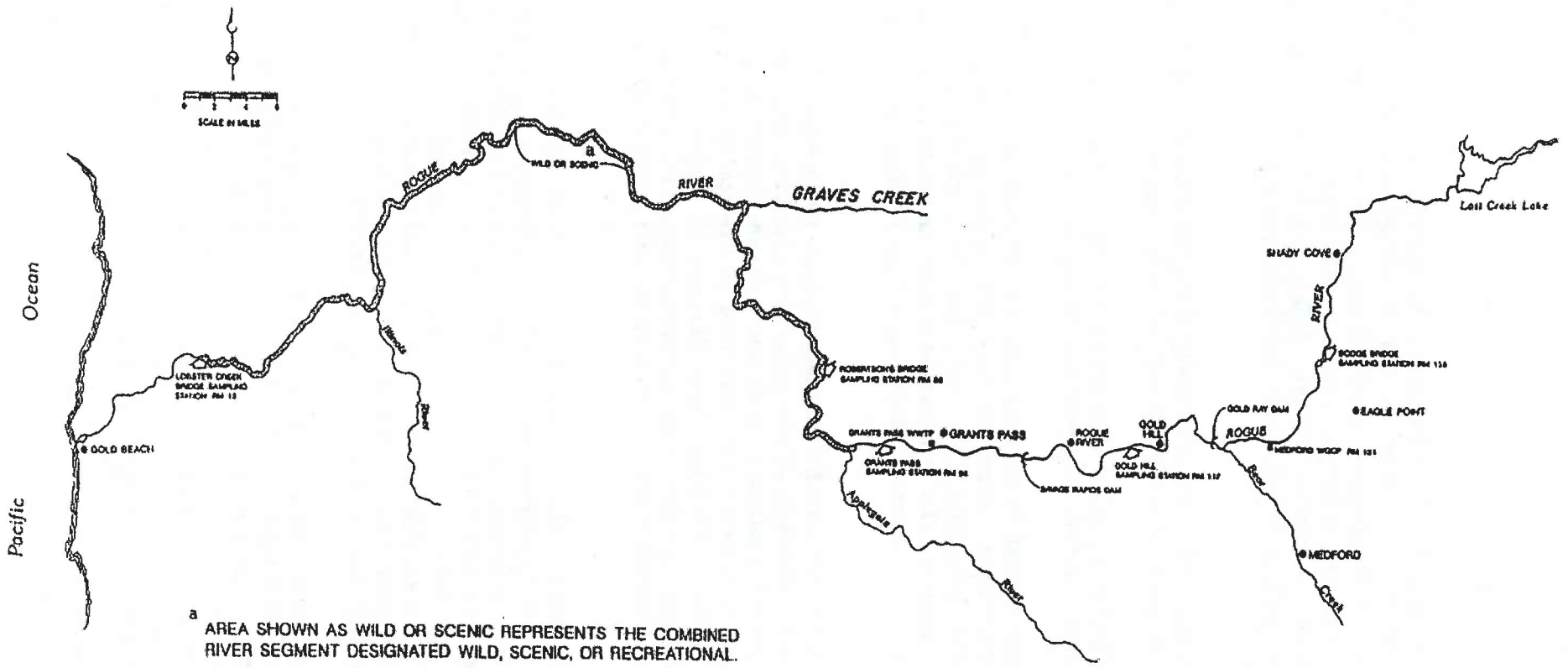
Current Rogue River Biological Status. Only scattered and incomplete biological survey information exists for the Rogue River. The final rule language does not mention specific methods for conducting biological surveys. Draft implementation plans suggest possible methods, but final decisions have not been made.

The other water quality parameters dealt with in this chapter all affect the river's biological community. The impact of these other parameters is discussed below.

Dissolved Oxygen

Minimum DO concentrations are necessary to protect and enhance aquatic life. Salmonid fish, particularly those in the embryonic and larval stages of development, are highly sensitive to low levels of DO.

DO Standards. The DO standards were significantly modified in 1996. Several factors, such as the presence of salmonid fish, dictate which portion of the new standard is applicable in a given stream segment. For the Rogue River, another important consideration is the life stage of the salmonid fish; eggs and fry are highly sensitive to low DO levels. The relevant parts of the DO standard for the Rogue River near Grants Pass (OAR 340-41-365 (2)) are:



[a] AREA SHOWN AS WILD OR SCENIC REPRESENTS THE COMBINED RIVER SEGMENT DESIGNATED WILD, SCENIC, OR RECREATIONAL.

FIGURE 6-1 ROGUE RIVER

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- For waterbodies which provide salmonid spawning, during the period from spawning to fry emergence, the DO shall not be less than 11 milligrams per liter (mg/L). However, if the minimum intergravel dissolved oxygen (IGDO) concentration, measured as a spatial median, is 8 mg/L or more, the DO criterion is 9 mg/L. In addition, if conditions of barometric pressure, altitude, and temperature preclude attainment of the 11 mg/L or 9 mg/L limits, DO levels shall not be less than 95 percent of saturation.

- For waterbodies which provide salmonid spawning, during the period from spawning to fry emergence, the spatial median IGDO shall not be less than 6 mg/L.

- A spatial median IGDO of 8 mg/L shall be used to identify areas where salmonid spawning, egg incubation, and fry emergence may be impaired.

- For water bodies supporting cold-water aquatic life, the DO shall not be less than 8 mg/L as an absolute minimum. Where conditions of barometric pressure, altitude, and temperature preclude attainment of the 8 mg/L limit, DO levels shall not be less than 90 percent of saturation. This limit may be modified at the discretion of the DEQ to 8 mg/L as a 30-day minimum, 6.5 mg/L as a 7-day minimum, and 6 mg/L as an absolute minimum.

For the Rogue River near Grants Pass, salmonid fish are present year-round; therefore, the river supports cold-water aquatic life and must comply with the cold-water DO standard (minimum DO concentration of 8 mg/L or 90 percent of saturation) at all times. Steelhead and Coho salmon spawn in tributaries to the Rogue River, but not in the main stem, so Steelhead and Coho eggs and fry are not present in the main stem of the Rogue River. However, Chinook salmon spawn in the main stem—spawning occurs in September. Eggs incubate throughout the winter and fry emerge in April. Therefore, the Rogue River must comply with the most stringent DO criteria from September through April.

The IGDO standards were added to the OARs in 1996 in an effort to better protect salmonid spawning areas. Because it is a new requirement, very little IGDO information is available. With the listing of Coho salmon as an endangered species, DEQ intends to focus their initial IGDO sampling work on the Rogue's tributaries, where Coho spawn. Therefore, IGDO sampling on the mainstem Rogue River may not occur for several years. Until detailed sampling is performed, it will be unclear if the Rogue River complies with the IGDO standards. It should be noted that DEQ probably will require that an IGDO survey be completed before approving a biochemical oxygen demand (BOD) mass discharge increase for a WWTP.

Current Rogue River Dissolved Oxygen. Average values for the level of DO saturation along the river during the summer are shown in Figure 6-2. The shallow, fast-moving river has a large capacity for reaeration as indicated by seasonal DO concentrations consistently near saturation. Winter values are shown in Figure 6-3. The average levels of DO saturation are above 100 percent for both the summer (May through October) and winter (November through April) months for all sampling points. The sampling station west (downstream) of Grants Pass has the highest mean summer and winter DO saturation of the five sampling stations. The average DO

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of this station was more than 100 percent of saturation during every month of 1980-82, as indicated in Figure 6-4. The average monthly DO saturation at the other stations occasionally drops below 95 percent.

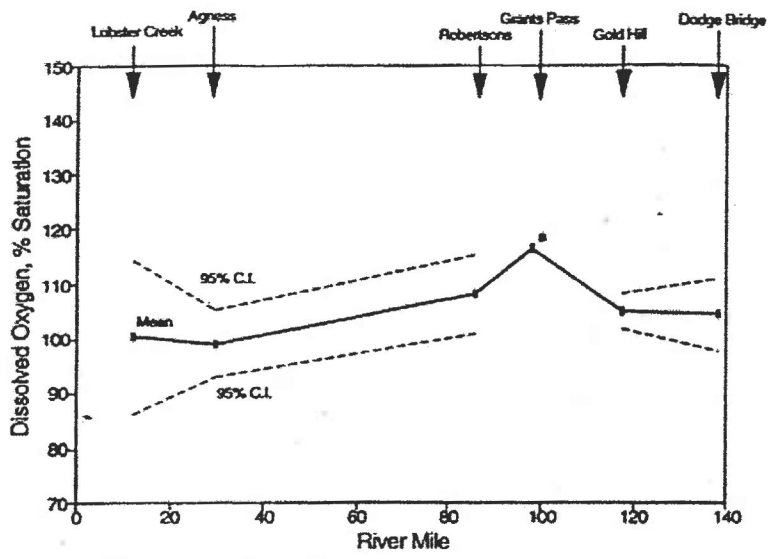
The average DO concentrations along the Rogue River are shown in Figure 6-5 for summer months and in Figure 6-6 for winter months. In the winter all six stations have mean concentrations around 12 mg/L. During the summer, the Grants Pass station (River Mile 98) is the only one to average 11 mg/L DO or greater (Table 6-1). The DO concentration at Dodge Bridge is almost always above 10.1 mg/L (Figure 6-7), while the minimum DO concentration detected west of Grants Pass during 1980 to 1982 was 10.0 mg/L (Figure 6-8).

The most important factor usually influencing DO concentrations is river temperature. The river is supersaturated with DO, and the WRP does not appear to bring DO concentrations below the saturation point. The importance of temperature is illustrated with a comparison of temperature and DO values from the 1970s and 1980s. Table 6-2 shows average summer temperatures and DO levels at Dodge Bridge before and after the construction of Lost Creek Dam. Lost Creek Dam, built in 1977, provides water storage for the summer months and serves to reduce the summer river temperature. The dam decreased temperatures by 2.5 degrees F, which increased DO concentrations by 0.5 mg/L. The degree of saturation, however, dropped by 0.7 percent. To improve DO concentrations, river temperature must be reduced.

Effects of Future Discharges on Dissolved Oxygen. The impact of plant effluent on DO was estimated using a Streeter-Phelps dissolved oxygen sag model obtained from the Washington Department of Ecology. Copies of the spreadsheet output are included as Appendix H. Both summer and winter conditions were examined. The summer condition used the 7Q10 river flow of 888 cfs, a plant effluent flow of 8 mgd, mass discharge CBOD of 667 pounds per day (ppd), and an effluent ammonia concentration of 10 mg/L. To focus only the effect of the plant discharge, the background river conditions consisted of saturation level for dissolved oxygen, and low values for CBOD and ammonia. The model output showed an initial DO depletion of 0.11 mg/L due to mixing of low DO plant effluent with the river. The critical DO deficit was 0.16 mg/L. The critical deficit was located about 29 miles downstream.

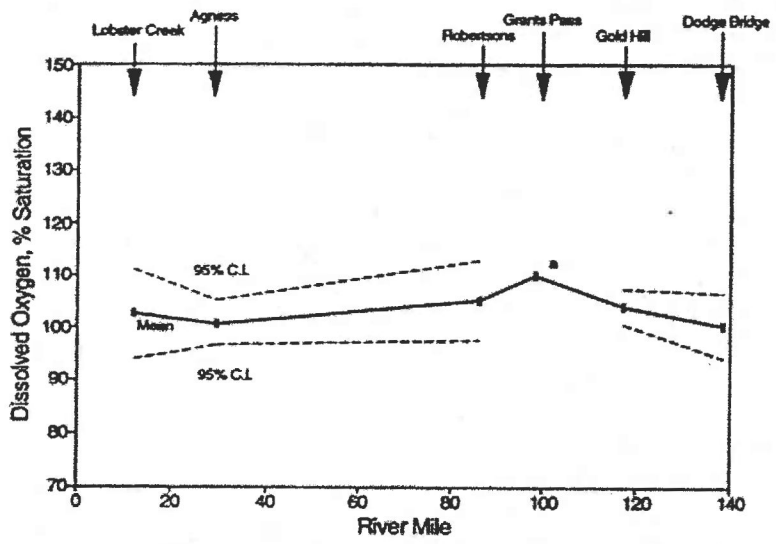
The winter condition examined also used the 7Q10 flow. Plant effluent flow was set at 10 mgd, mass discharge for CBOD at 2,500 ppd, and effluent ammonia concentration of 10 mg/L. Again, to separate out background conditions, the upstream DO was set at saturation, with CBOD and ammonia concentrations at low values. In this case, the initial DO depletion due to effluent mixing was 0.05 mg/L and the critical DO deficit was 0.22 mg/L. The critical deficit was located about 52 miles downstream.

This model demonstrates that the WRP discharge will have a negligible effect on river DO. The DO concentrations are primarily dictated by temperature as it affects oxygen saturation.



a. 95 percent Confidence Interval (C.I.) not shown because of limited data at this sampling station

Figure 6-2. Summer Dissolved Oxygen Saturation



a. 95 percent Confidence Interval (C.I.) not shown because of limited data at this sampling station

Figure 6-3. Winter Dissolved Oxygen Saturation

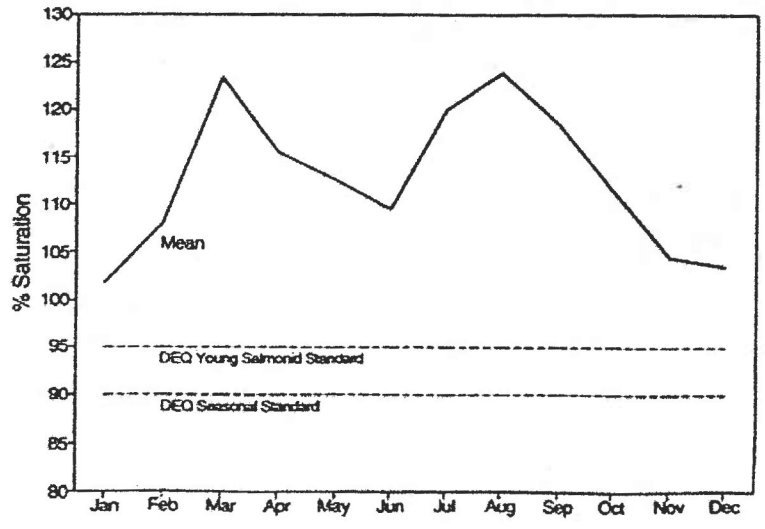
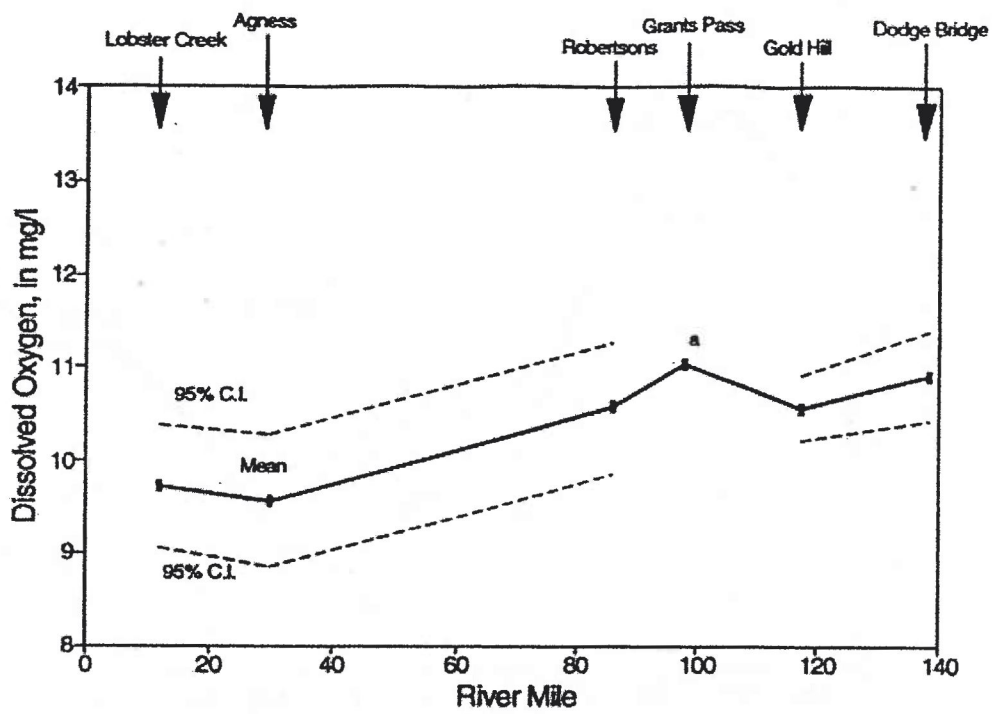
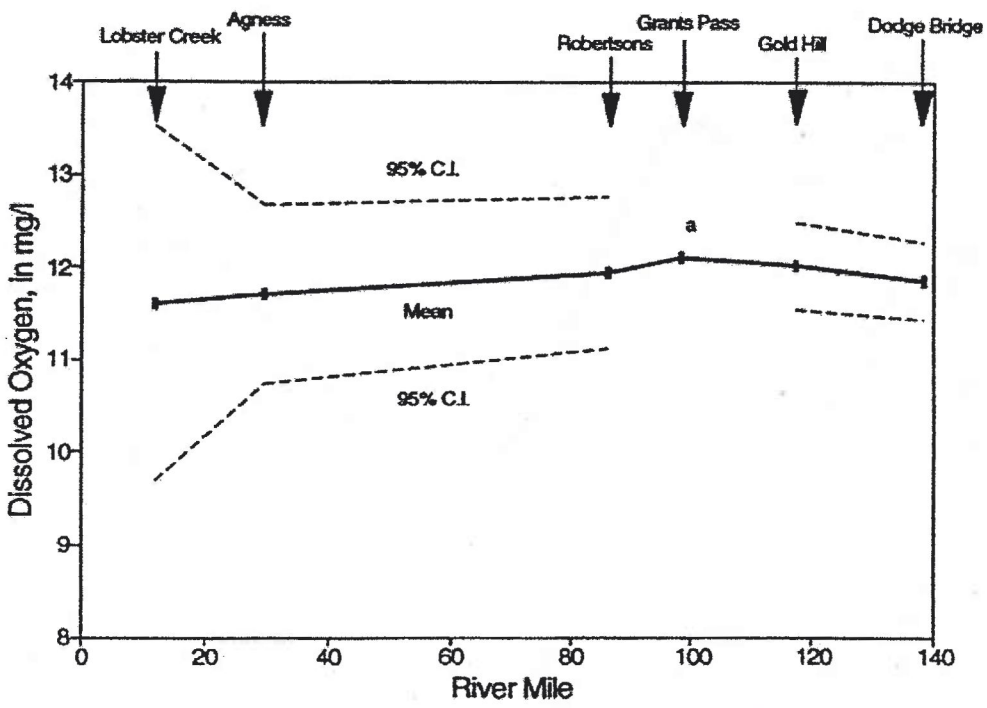


Figure 6-4. Dissolved Oxygen Saturation at Grants Pass



a. 95 percent Confidence Interval (C.I.) not shown because of limited data at this sampling station

Figure 6-5. Summer Dissolved Oxygen Concentration



a. 95 percent Confidence Interval not shown because of limited data at this sampling station

Figure 6-6. Winter Dissolved Oxygen Concentration

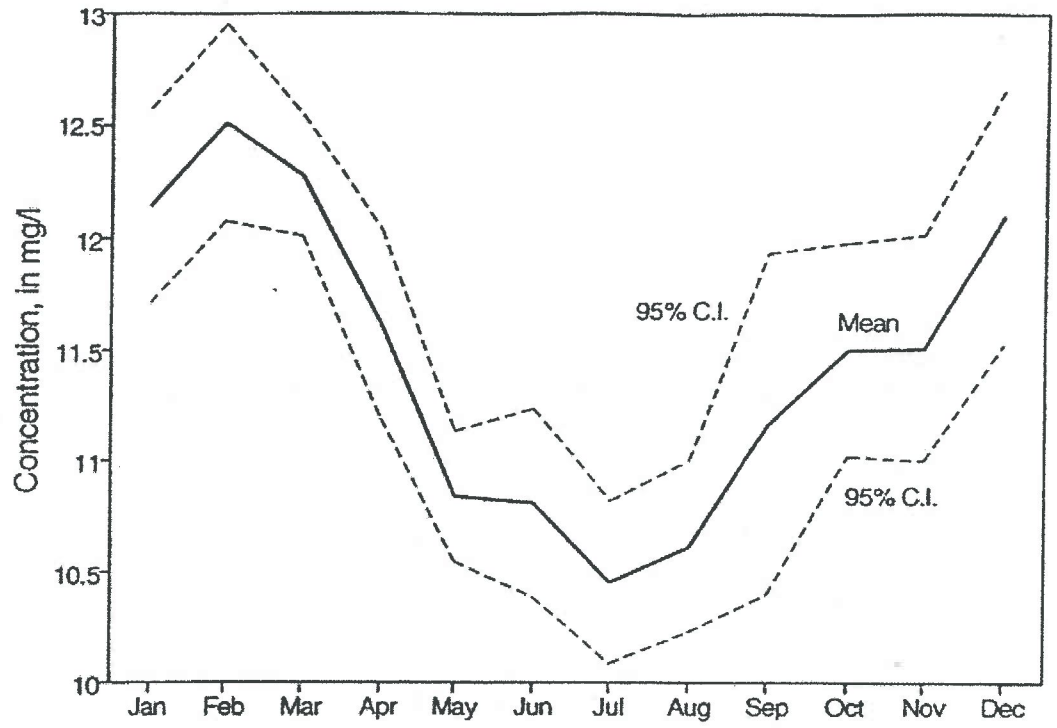


Figure 6-7. Dissolved Oxygen Concentration at Dodge Bridge

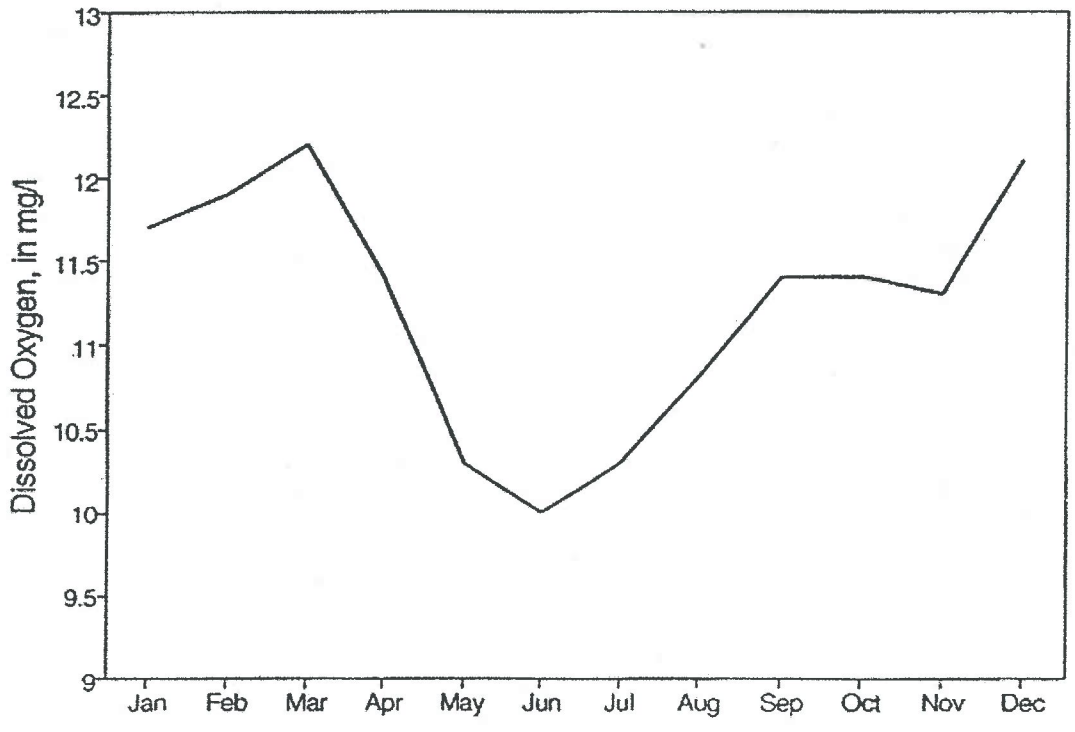


Figure 6-8. Minimum DO Concentrations at Grants Pass

Table 6-1. Mean Dissolved Oxygen Values

River Mile	DO Saturation %					
	12	29.7	86	98	117.3	138.4
Jan	101.0	101.3	100.0	101.8	101.1	98.4
Feb	107.3	99.9	106.2	108.0	104.6	100.7
Mar	104.0	101.1	108.6	123.3	108.6	105.4
Apr	103.0	100.6	106.1	115.5	104.7	102.6
May	96.0	100.1	114.7	112.5	104.2	104.6
Jun	101.0	99.8	111.0	109.5	106.1	103.0
Jul	99.4	99.2	110.1	120.0	104.4	105.6
Aug	103.6	99.1	112.7	124.0	106.6	109.0
Sep	98.0	99.6	102.4	118.5	104.0	104.7
Oct	100.0	98.4	105.3	111.5	104.0	100.6
Nov	106.7	103.9	101.3	104.5	103.1	93.2
Dec	99.5	99.0	99.9	103.5	101.8	98.6
Summer	99.7	99.4	109.4	116.0	104.9	104.6
Winter	103.6	101.0	103.7	109.4	104.0	99.8
Annual	101.6	100.2	106.5	112.7	104.4	102.2

River Mile	DO Concentration, mg/L					
	12	29.7	86	98	117.3	138.4
Jan	12.4	12.6	12.2	12.4	12.5	12.1
Feb	12.2	12.2	12.6	12.4	12.7	12.5
Mar	11.6	11.7	12.1	12.8	12.3	12.3
Apr	10.9	10.6	11.6	11.9	11.5	11.6
May	10.0	10.5	11.1	10.8	10.7	10.8
Jun	9.7	9.4	10.5	10.3	10.5	10.8
Jul	8.7	9.0	9.9	10.6	9.9	10.5
Aug	9.4	9.0	10.5	11.2	10.3	10.6
Sep	9.9	9.8	10.6	11.6	10.7	11.2
Oct	10.9	10.5	11.3	11.7	11.5	11.5
Nov	12.4	12.3	11.6	11.9	11.8	11.5
Dec	11.8	12.1	12.4	12.6	12.6	12.1
Summer	9.8	9.7	10.6	11.0	10.6	10.9
Winter	11.9	11.9	12.1	12.3	12.2	12.0
Annual	10.8	10.8	11.4	11.7	11.4	11.5

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Table 6-2. Effect of Lost Creek Dam on Temperature and DO at Dodge Bridge

Parameter	1970s[a]	1980s[a]
DO, mg/L	10.4	10.9
DO, percent saturation	105.4	104.7
Temperature, degrees F	56.4	53.9

[a] Summer average.

Temperature

Water temperature directly influences the solubility of dissolved oxygen. Increases in temperature result in decreased dissolved oxygen concentrations, which adversely affect aquatic life. Young salmonid fish are most susceptible to changes in temperature.

Temperature Standards. The temperature standard was also updated in 1996. For the Rogue River near Grants Pass, the new standard requires that no measurable increase in water temperature is allowed if:

- River temperature exceeds 64 degrees F.
- River temperature exceeds 55 degrees F during periods of salmonid spawning, egg incubation, and fry emergence.
- The temperature increase would impair the biological integrity of federally listed threatened or endangered species.
- DO levels are within 0.5 mg/L or 10 percent saturation of the water column or IGDO criterion.

However, an exceedance of the above limits is not considered to be a temperature standard violation if it occurs when the air temperature during the warmest 7-day period of the year exceeds the 90th percentile of the 7-day average daily maximum air temperature.

Because the Rogue River is listed as water quality limited for summer temperature, the City of Grants Pass, along with other dischargers to the river, may be required to develop and implement a temperature management plan. The plan, which will be a part of the next National Pollutant Discharge Elimination System (NPDES) permit, must describe the best management practices and control technologies that will be used to reduce the warming trend of the Rogue River. The river temperature achieved after all feasible steps have been taken will be the new temperature criterion. Once the WRP is in compliance with the temperature management plan, it will not be considered as a contributor to Rogue River temperature standard violations.

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Current Rogue River Water Temperature. The temperature of the Rogue River is heavily influenced by Lost Creek Dam, which is operated by the U.S. Army Corps of Engineers (COE). The dam's outlet structure allows water to be discharged from four different elevations, permitting reasonably accurate temperature control. The Oregon Department of Fish and Wildlife (ODFW) provides the COE with a temperature regulation scheme designed to enhance anadromous fish management. ODFW has been studying fish runs in the river for more than 20 years trying to determine how to best operate the dam. ODFW has found that fish survival rates are much higher when river temperatures are lowered during spring and fall spawning than when they are lowered during the summer. The COE could lower summer river temperatures; however, river temperatures during the critical spring and fall months would then increase. We compared actual river temperatures to river temperatures requested by ODFW for the years 1987 through 1991. During the dry weather months, the COE matched the temperatures requested by ODFW very precisely. However, during the wet weather season, actual river temperatures were 3.6 to 12.6 degrees Fahrenheit above the requested temperatures.

The average summer (May - October) water temperatures for the five monitoring sites along the river are shown in Figure 6-9. The average temperature increases as the water travels downstream. The average summer temperature at Dodge Bridge is 53.1 degrees F. The temperature increases downstream to 62.5 degrees at Lobster Creek, as shown in Table 6-3.

Table 6-3. Mean Temperatures

Station	Mean river temperature, degrees F					
	Lobster Creek	Agness	Robertson's Bridge	Grants Pass	Gold Hill	Dodge Bridge
River mile	12	29.7	86	98	117.3	138.4
Jan	44.2	42.5	43.0	42.6	41.1	41.3
Feb	46.0	44.0	45.2	46.9	43.1	40.7
Mar	50.8	48.6	50.2	48.5	48.1	45.2
Apr	57.0	56.0	51.8	53.6	49.6	46.7
May	57.7	56.5	60.9	56.2	56.0	53.2
Jun	64.4	64.8	62.8	61.6	58.8	53.8
Jul	71.9	67.6	63.1	65.2	62.5	58.2
Aug	68.1	67.6	60.3	64.2	61.3	59.8
Sep	59.9	61.3	59.0	59.2	55.2	47.6
Oct	53.2	54.7	53.7	52.9	49.8	45.8
Nov	48.2	47.9	47.7	46.4	46.3	47.5
Dec	46.7	43.9	41.9	43.2	41.5	41.1
Summer[a]	62.5	62.1	60.0	59.9	57.3	53.1
Winter[b]	48.8	47.1	46.6	46.9	45.0	43.7
Annual	55.7	54.6	53.3	53.4	51.1	48.4

[a] The 6-month period May - October.
[b] The 6-month period November - April.

The trend toward increased temperature downstream results from impoundments, discharges from wastewater treatment plants, tributary inflows, and irrigation return flows. Exposure to direct sunlight also increases water temperature. Plant effluent from the WRP, for instance, is typically above 70 degrees F during the summer months. Water from the Applegate River is 0.5 to 4.0 degrees warmer than that in the Rogue River at Grants Pass. Temperatures in the Rogue River increase downstream of these influences.

In comparing the river temperatures with the new temperature standard, it is necessary to distinguish periods of salmonid spawning, egg incubation, and fry emergence. Chinook salmon spawn in the mainstem of the Rogue River in September. Their eggs incubate during the winter and fry emerge in April. Coho salmon and steelhead spawn in tributaries to the Rogue; therefore, they are not a factor in determining applicable temperature limits. The Rogue must comply with the most stringent temperature limits (those for salmonid spawning, egg incubation, and fry emergence—55 degrees F) during September through April. For the remainder of the year, the Rogue's temperature must conform to the standards for salmonid fish rearing—64 degrees F.All sampling stations downstream of Dodge Bridge report periods in September through April where the mean river temperature does not comply with the 55 degree F standard (Table 6-3). Grants Pass, Agness, and Lobster Creek all report mean river temperatures that exceed 64 degrees F in May through August. This comparison shows that mean river temperatures often exceed the standards; however, the temperature limits actually represent maximum allowable values. Therefore, it is clear that the Rogue River is in violation of the new temperature standards.

Flow in the Rogue River since 1978 has been largely regulated by the Lost Creek Dam and mean temperatures have remained relatively constant (Appendix C, Figure 1). Prior to completion of the dam, mean temperatures at Grants Pass showed a slow increase from about 50.5 degrees in 1932 to about 52.5 degrees in 1977 (Appendix C, Figure 2). The dam's greatest impact on river temperature, however, has not been on mean temperature, but rather on decreasing the peak temperatures. Plotting the average of the ten highest temperatures each year from 1932 to 1977 shows an average decrease of nearly 3 degrees F (Appendix C, Figure 3). Most of the highest temperatures occurred during July and August. In July 1968, for instance, the river temperature reached 78 degrees. The 1980s, on the other hand, have seen mean peaks of 69 degrees or less (Appendix C, Figure 4).

Effects of Future Discharges on River Temperature. Plant effluent and Rogue River temperature data are summarized in Appendix H. This data shows that the plant effluent temperature ranges from 8 to 17 degrees F warmer than the upstream river temperature. On average, the plant effluent is 12.7 degrees F warmer. The projected year 2020 plant ADWF of 8.0 mgd could increase the average river temperature by about 0.17 degrees F over present background temperatures and 7Q10 flows. This temperature change, although technically in violation of the water quality standard, is within the practical error tolerance for measuring the river temperature.

The effects of future discharges to the river temperature will be addressed in a Temperature Management Plan, to be completed at a later date.

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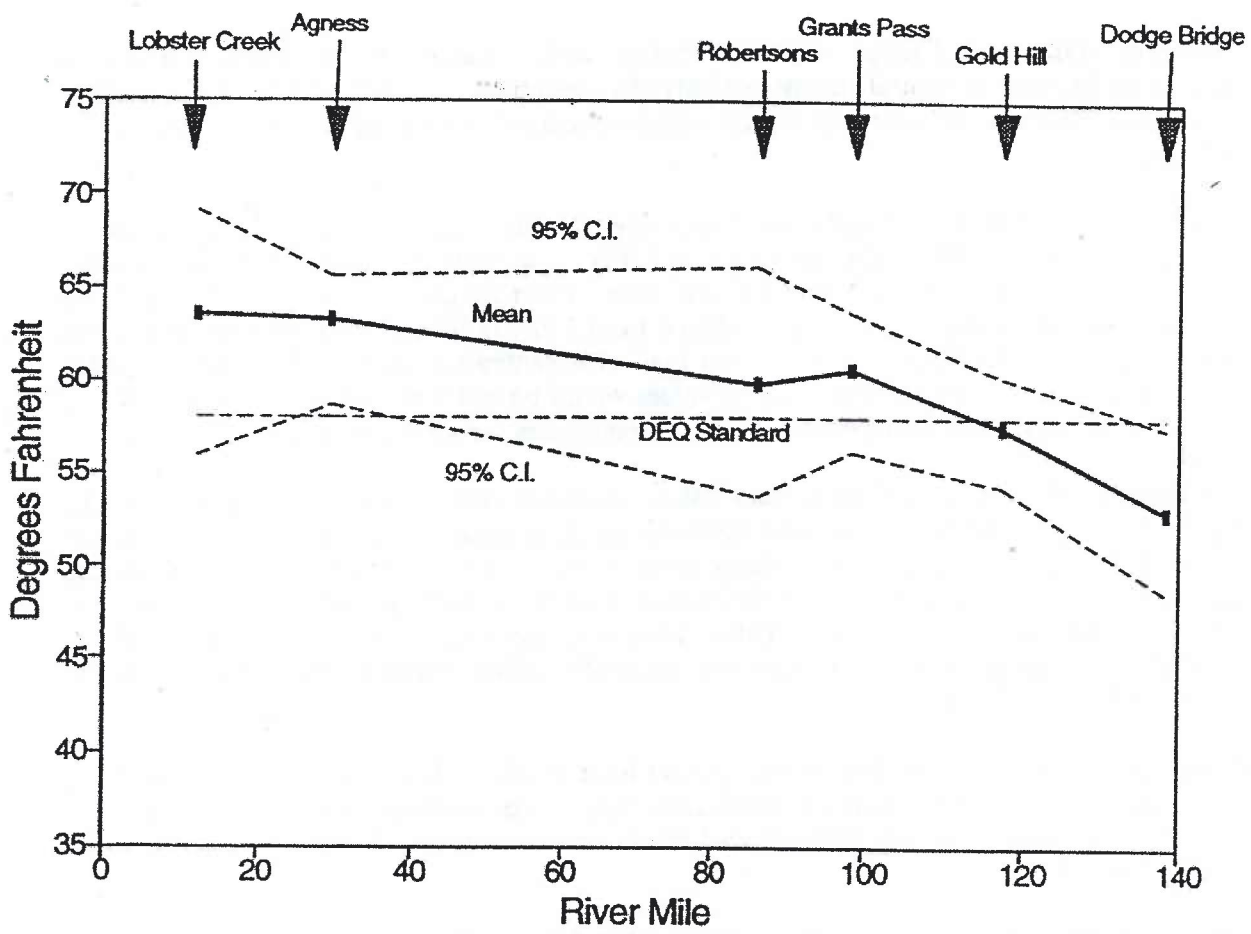

Figure 6-9. Summer Temperature

Turbidity and Suspended Solids

High levels of turbidity in a water body can adversely affect aquatic life, recreation, and use as a domestic water supply. Turbidity and particulate matter affects the ability of fish to locate prey, causing a reduction in growth. Turbidity lessens the aesthetic value of a water body, making it less desirable for recreational purposes. Turbid waters are more difficult and expensive to treat for domestic water uses.

Turbidity Standards. The current DEQ turbidity standard states that no more than a 10 percent cumulative increase in natural stream turbidity (Nephelometric Turbidity Units, NTU) shall be allowed, as measured relative to a control point immediately upstream of the turbidity-causing activity.

Current Rogue River Turbidity and Suspended Solids. The average turbidity at Dodge Bridge is 4.1 NTU. This value increases to 5.0 NTU at Gold Hill. The WRP staff collect turbidity data for both plant effluent and the river. Plant effluent ranged from 4 to 11 NTU whereas monthly average river turbidity ranged from 3 to 137 NTU. Average suspended solids concentrations in the Rogue River are also low. Concentrations include 5.1 mg/L at Dodge Bridge and 8.4 mg/L at Gold Hill. These values would be still lower except for a few isolated storm events which raised suspended solids concentrations for short periods.

The average 1991 through 1996 suspended solids concentration of the plant effluent is 7.2 mg/L, higher than the 2.5 mg/L average value measured by plant personnel in the river upstream from the water plant. However, since plant flow averages less than 1 percent of the river flow during summer, downstream concentrations only increase 0.07 mg/L, to 2.57 mg/L. During the winter the plant flow constitutes an even smaller portion of the river flow, 0.43 percent, and the difference between upstream and downstream suspended solids concentrations is less, increasing from 10.08 to 10.09 mg/L.

If the plant discharged at its dry weather permit limit of 20 mg/L during minimum river flow when the river suspended solids concentration was at its maximum dry weather value of 7.0 mg/L, the maximum level of suspended solids after mixing the effluent in the river would be less than 7.1 mg/L.

Effects of Future Discharges on River Suspended Solids. Assuming the Grants Pass WRP ADWF increases from 4.5 mgd to 8.0 mgd, the worst-case concentration of suspended solids would still be less than 7.2 mg/L.

pH

pH changes can affect aquatic life and recreation. Although fish are able to handle moderately large changes in pH, certain chemicals can be rendered more toxic by changes in pH.

pH Standards. The current DEQ standard states that pH values shall be between 6.5 and 8.5.

Current Rogue River pH. The pH of the Rogue River fluctuates greatly. Diurnal pH swings are typically associated with biological activity associated with algal growth and attached periphyton. This effect is amplified when the water has low alkalinity levels. Swings in pH is often indicative of excessive algae and periphyton growth due to high nutrient levels. Figure 6-10 shows that the 95 percent confidence limits for pH during the summer remain between the minimum standard of 6.5 and the maximum of 8.5 along the entire length of the Rogue River. Winter pH at the most downstream point, Lobster Creek, occasionally falls below 6.5 as shown by the 95 percent confidence interval on Figure 6-11. Plant effluent pH generally falls between 6.4 and 7.5, somewhat lower than that of the river. However, plant effluent alkalinity is three to four times that of the river. The additional alkalinity should help to stabilize the pH of the river. Under a worst-case scenario, with minimum river and effluent pH during the summer, the plant effluent would drop the river pH less than 0.01 percent.

Effects of Future Discharges on River pH. pH should not be a concern to future water quality. In the future, the effluent would still drop the river pH less than 0.01 percent. The slightly acidic effluent is of even less concern when the trend towards increasing pH in the river is taken into account (Figures 6-12, 6-13, and 6-14).

Bacteria

Pathogenic, or disease-causing, bacteria are deleterious to the beneficial uses of bathing and fishing. High levels of pathogenic bacteria can cause gastrointestinal illness in bathers and others who come in direct contact with the river water.

Bacteria Standards. The previous DEQ standard stated that fecal coliform shall not exceed a log mean of 200 per 100 milliliters (mL) based on a minimum of five samples in a 30-day period. No more than 10 percent of the samples during a 30-day period shall exceed 400 per 100 mL.

The newly adopted DEQ standard requires that the 30-day log mean of E. coli, based on a minimum of 5 samples, shall not exceed 126 per 100 mL. Single-sample densities shall not exceed 406 E. coli per 100 mL.

Bacteria standards for WWTP effluent are similar to the in-stream limits. However, if the 406 colonies/100 mL single sample limit is exceeded, then five consecutive resamples shall be taken at 4-hour intervals. If the log mean of the five resamples is less than 126/100 mL, then the single-sample exceedance is not considered a violation.

To help limit bacteria levels in water bodies, the OARs also restrict the discharge of raw sewage. From November 1 through April 30, raw sewage discharges are prohibited except during a storm event larger than the one-in-five year, 24-hour storm. From May 1 through October 31, raw sewage discharges are prohibited except during a storm event larger than the one-in-ten year, 24-hour storm. Cities that have difficulties complying with these requirements may apply for exceptions.

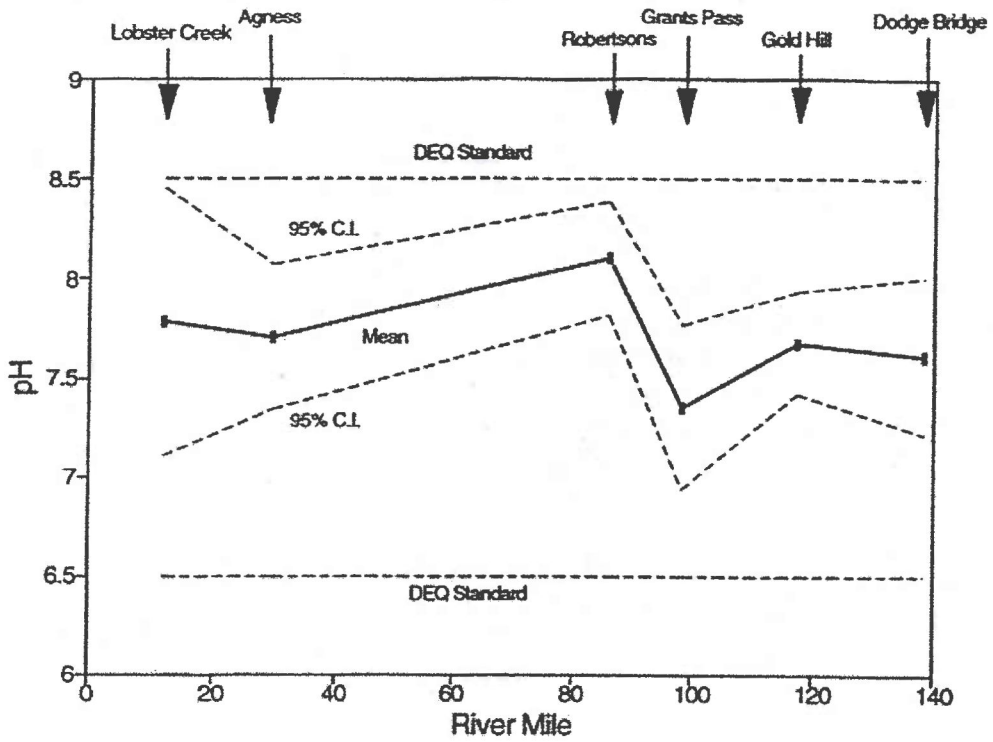

Figure 6-10. Summer pH

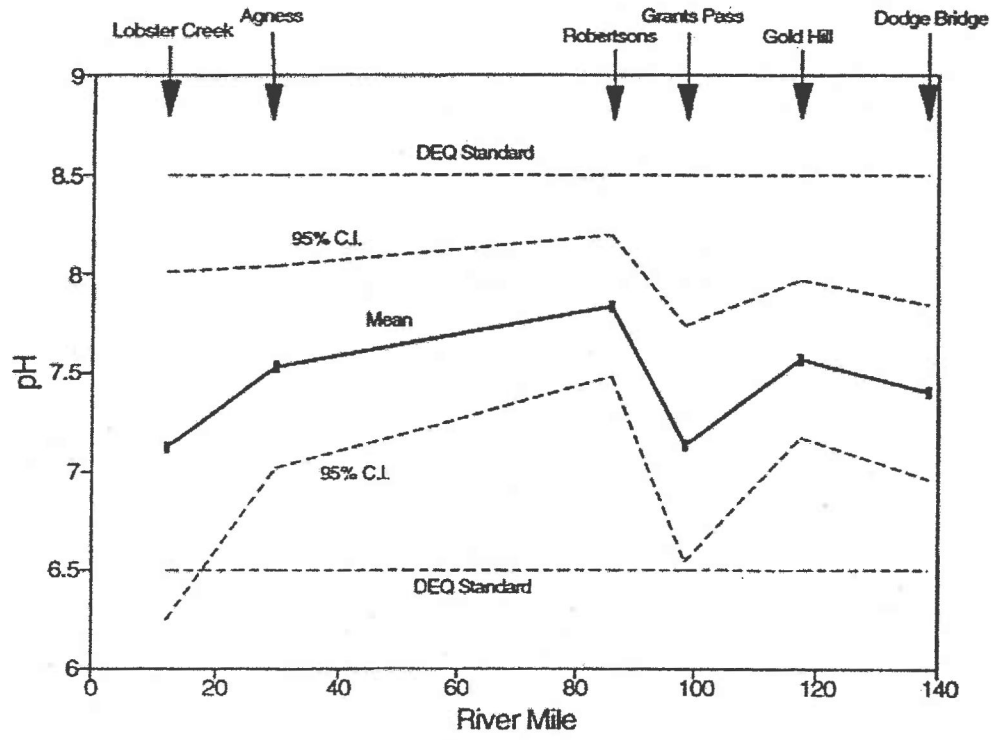

Figure 6-11. Winter pH

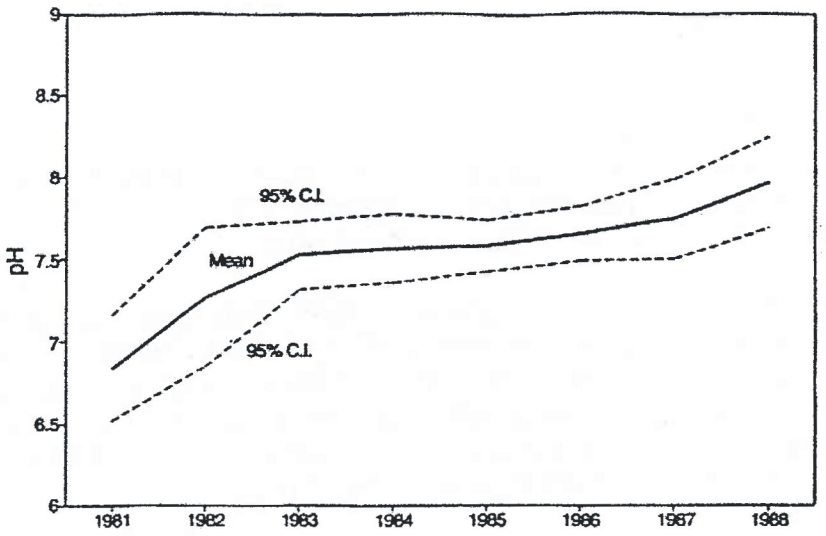

Figure 6-12. Yearly pH at Dodge Bridge

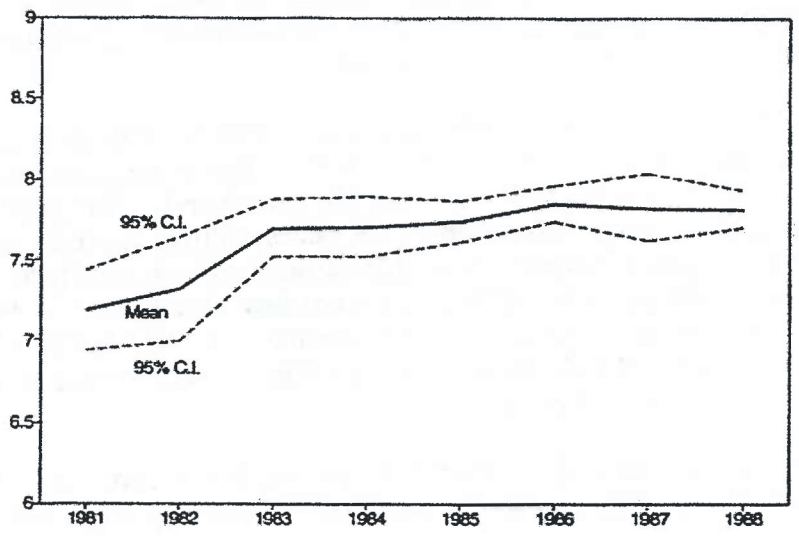

Figure 6-13. Yearly pH at Goldhill

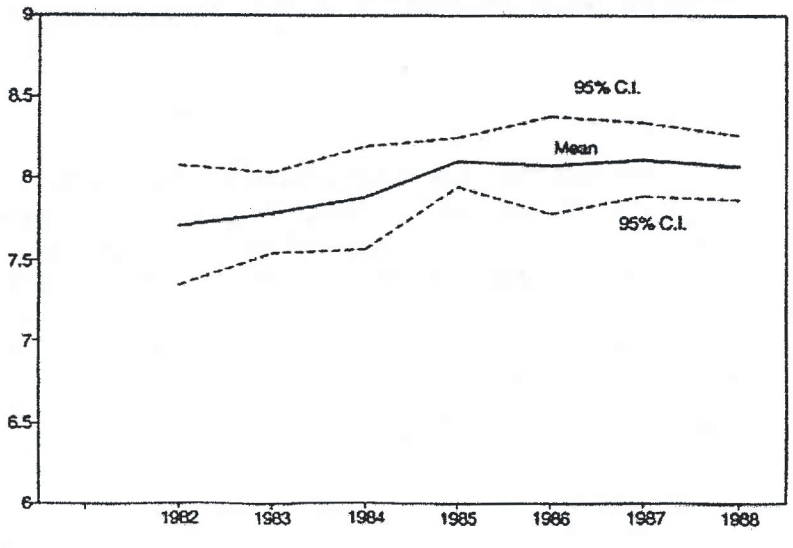

Figure 6-14. Yearly pH at Robertson's Bridge

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Because the Rogue River is listed as water quality limited for bacteria, the DEQ can require the development and implementation of a bacteria management plan. For the Grants Pass WRP, the NPDES permit is considered to be the bacteria management plan.

Current Rogue River Bacterial Status. The geometric mean of the fecal coliform counts at Dodge Bridge is 15 per 100 mL, and the maximum is 240 per 100 mL. Single-sample counts occasionally exceed the previous allowable level. The geometric mean of the E. Coli densities is 10 per 100 mL, and the maximum is 100 per 100 mL. The Rogue River at Dodge Bridge easily meets both the previous fecal coliform and the newly adopted E. Coli monthly average standards; however, single-sample standards have been exceeded.

Fecal coliforms at Gold Hill have a geometric mean of 47 per 100 mL and a maximum of 460 per 100 mL. The geometric mean of the E. Coli counts is 14 per 100 mL and the maximum is 310 per 100 mL. While the geometric means of the fecal coliform and E. Coli counts meet previous and newly adopted DEQ standards, the maximum single sample values exceed the allowable densities for both indicator organisms.

The fecal coliform counts for the Grants Pass WRP effluent from 1988 to 1990 have a geometric mean of 0.85 per 100 mL and a maximum of 1,000 per 100 mL. This maximum occurred only once during the three years; the next highest count was 224 per 100 mL. The plant effluent meets the standards for fecal coliforms. Comparing these values with the upstream river fecal coliform densities measured by plant personnel shows that the plant effluent actually has a lower geometric mean and lower average number of fecal coliforms than does the river. However, the maximum of 1,000 per 100 mL is greater than the maximum of 410 per 100 mL found upstream. Thus, compared with river samples, the treatment plant effluent shows a lower mean but wider variance in fecal coliform concentrations.

E. Coli have not been routinely measured at the WRP, but we believe that these indicator organisms will be present at reasonably low levels. Disinfection capability for E. Coli is very similar to fecal coliform.

Effects of Future Discharges on River Bacteria. Future discharges of bacteria should not be a concern, since the plant effluent has lower mean bacteria densities than the Rogue River.

Toxic Substances

Toxicity Standards. The DEQ standards state that toxic substances shall not be introduced above natural background levels in amounts that may be harmful to public health or aquatic life. Levels of toxic substances shall not exceed the criteria established by the Environmental Protection Agency (EPA) and published in Quality Criteria for Water (1986). The DEQ may allow a mixing zone within which waters shall be free of materials in concentrations that cause acute toxicity. Waters outside the boundary of a mixing zone shall be free of materials in concentrations that cause chronic toxicity.

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Acute toxicity is defined as lethality to aquatic life as measured by approved bioassay. Lethality in 100 percent effluent due to ammonia or chlorine may be allowed when immediate dilution in the mixing zone reduces toxicity below acutely toxic levels. The DEQ may, on a case-by-case basis, establish a zone of immediate dilution for parameters other than ammonia and chlorine if appropriate.

Current Rogue River Toxics. Over the past ten years, the Rogue River has been sampled for heavy metals four times per year. Metals, including cadmium, zinc, and silver, were detected at concentrations exceeding the EPA acute toxicity standards on seven occasions. Cadmium was measured at 2 micrograms per liter (ug/L) twice—once in 1980 and once in 1989. Silver was measured at 4 ug/L in 1984, and at 1 ug/L in 1987 and 1988. A zinc concentration of 150 ug/L occurred in 1982 and one of 58 ug/L occurred in 1984. However, cadmium was below detection limits 75 percent of the time and silver was below detection limits 92 percent of the time. Toxicity limits were calculated using formulas in EPA's Quality Criteria for Water, 1986. These limits are dependent upon water hardness. Because the Rogue River has a relatively low water hardness (20 to 40 mg/L), the allowable concentration approaches the detection limit of some metals. Since the accuracy of an analytical test is questionable at concentrations near the detection limit, and because of a limited amount of available data, it is inconclusive whether a heavy-metal toxicity problem exists in the Rogue River. The DEQ current toxicity criteria do not include the water hardness dependency on metal toxicity. However, hardness dependent criteria will likely be adopted soon.

The river has also been tested for ammonia. Ammonia toxicity depends on temperature and pH as well as ammonia concentration. Both chronic and acute toxicity increase as temperature and pH increase. For typical summertime river temperatures (50 to 60 degrees F) and pH levels (7.4 to 7.7), acute toxicity corresponds to a total ammonia concentration of about 10.5 to 12 mg/L. For these same conditions, chronic toxicity corresponds to total ammonia concentrations of about 2.0 to 2.2 mg/L. Ammonia concentrations in the Rogue River at all three sites are well below the chronic toxicity levels. The ammonia levels at Dodge Bridge average 0.025 mg/L, with a maximum value of 0.130 mg/L. The levels are slightly higher at Gold Hill, 0.062 mg/L on average and 0.160 mg/L maximum. At Grants Pass, measured values are 0.088 mg/L on average and 0.210 mg/L maximum.

Spreadsheets from the Washington Department of Ecology were used to estimate water quality based discharge permit limits for protection from acute and chronic ammonia toxicity. These spreadsheets along with WRP influent and effluent ammonia data are included in Appendix H. Dilution factors from the outfall mixing zone study (Appendix A) were applied. The mixing available in the zone of immediate dilution is 1.7:1 and at the edge of the mixing zone 10:1. Both summer and winter conditions were examined. During the summer the waste load allocation for the acute (1-hour) condition was estimated at 15.5 mg/L and for the chronic (4-day) condition 16.2 mg/L. Allowing a 0.6 coefficient of variance for effluent concentration and eight effluent samples per month resulted in a daily maximum ammonia permit limit of 15.5 mg/L. The monthly average ammonia limit was 6.8 mg/L. Under cooler winter conditions the waste load allocation for acute toxicity was 16.5 mg/L and for chronic conditions 21.2 mg/L. These values corresponded to effluent limits of 16.5 mg/L and 7.3 mg/L for daily maximum and monthly average respectively.

Comparing the estimated permit limits to the available WRP effluent ammonia data (presented in Appendix H) indicates that the WRP could potentially exceed both the daily maximum and monthly average limits during both summer and winter seasons. However, the existing plant data consisting of two to eight grab samples per month does not adequately describe the actual plant effluent. It is evident that improved dilution would allow for permit conditions that the WRP could comply with without the need for nitrification. Additional effluent ammonia data should be collected and an outfall diffuser predesign study conducted to determine potential mixing available and to establish appropriate ammonia discharge limits.

Chlorine toxicity is the third class of substances of concern. Since the WRP switched to UV disinfection, the plant no longer discharges any chlorine to the Rogue River.

Effects of Future Discharges on River Toxics. Thorough studies of the mixing zone and the plant effluent are needed before future effects can be predicted. However, based on analyses at similar plants, only ammonia is expected to be of concern in the future.

Nuisance Phytoplankton

Nuisance phytoplankton can diminish the aesthetic value of a water body and also impair fish growth (Welch, 1980). Algae blooms discourage swimming and other recreational activities. In extreme cases, algal respiration can deplete DO, harming fish and other aquatic life. Typically, nuisance phytoplankton growth is the result of increased nutrient levels in a waterbody. The nutrients of concern are phosphorus and nitrogen. Algae require seven times as much nitrogen as phosphorus for optimum growth. If the concentration of the two nutrients remains above a 7-to-1 nitrogen-to-phosphorus ratio, phosphorus is considered the limiting nutrient. In other words, more phosphorus must be added before any additional algal growth will be seen. If the nitrogen-to-phosphorus ratio drops below 7 to 1, nitrogen becomes the limiting nutrient. The existing water quality standards address phytoplankton growth but have no provisions for the regulation of nutrient levels.

Phytoplankton Standards. The current DEQ goal states that average chlorophyll-a values should not exceed 15 ug/L in reservoirs, rivers, estuaries, and lakes that do not thermally stratify. Chlorophyll-a is a photosynthetic pigment found in algae that is used to estimate the concentration of algae in a water body. When allowable chlorophyll-a concentrations are exceeded, the DEQ is authorized to conduct studies to describe the present water quality, determine impacts on beneficial uses, determine probable causes of the excessive phytoplankton growth, and develop a control strategy. If natural conditions are found to be responsible for the growth, the allowable chlorophyll-a concentration may be modified.

Current Rogue River Phytoplankton. Chlorophyll-a concentrations for several stations along the Rogue River are shown in Figure 6-15. The highest average summer chlorophyll-a concentration is 2.8 ug/L at Lobster Creek, River Mile 12. This value is less than one-fifth of the DEQ standard of 15 ug/L.

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Phosphorus is the nutrient that most often limits the growth of algae. Therefore, phosphorus removal is typically looked at as the best method of reducing the size of algae blooms. Phosphorus levels are expressed in two different ways: total phosphorus and ortho phosphorus. Total phosphorus includes all organically and inorganically bound phosphorus, while ortho phosphorus is phosphorus bound in simple inorganic phosphates (PO_4^{3-}). While total phosphorus may be converted to ortho phosphorus in many natural lake and reservoir systems, this process is too slow to be a significant factor in most streams (EPA, 1984).[3]

Figure 6-16 shows summer ortho phosphorus concentrations for the Rogue River. The highest average ortho phosphorus concentration is 0.087 mg/L, occurring at Gold Hill. The average summer ortho phosphorus concentration at Grants Pass is 0.069 mg/L.

The other nutrient of concern, nitrogen, is also contributed by the WRP. The forms of nitrogen most readily available to phytoplankton are the inorganic forms: nitrite, nitrate, and ammonia. Figure 6-17 shows the average concentrations of inorganic nitrogen occurring during the summer along the Rogue River. The highest average, 0.208 mg/L, is found at Grants Pass.

The inorganic nitrogen-to-ortho phosphorus ratio ranges from 2 to 3 at the Rogue River monitoring sites. Nitrogen thus appears to be the limiting nutrient. This is somewhat unusual, but not unheard of. Water resources data published by the USGS indicate that the South Umpqua River, for instance, has an inorganic nitrogen-to-ortho phosphorus ratio of 3.6.[1] If nitrogen is limiting, one would expect that blue-green algae might be more numerous than other types of phytoplankton because the blue-greens can fix their own nitrogen. Only one analysis of algal types on the Rogue has been published. The USGS collected samples in 1980-1981 at the Agness station (USGS, 1981).[2] They show that during the summer diatoms are the dominant phytoplankton, not blue-green algae.

Figure 6-18 shows the measured and expected chlorophyll-a concentrations at the Rogue River sites. The figure, derived from one given in an EPA report (EPA, 1983), clearly shows that the nutrient values fall within the nitrogen-limited area, less than a 7-to-1 nitrogen-to-phosphorus ratio. However, the figure also shows that the measured chlorophyll-a values are only a fraction of what would be expected. Grants Pass, for instance, would be expected to have average chlorophyll-a values of almost 30 ug/L. The observed average is less than 2.5 ug/L. Even the maximum chlorophyll-a observed since 1981 at Grants Pass has been only 9.4 ug/L. The other sites monitored along the river have similarly low chlorophyll-a maximums during this time period. Lobster Creek had 6.3 ug/L, Robertson's Bridge 12.5 ug/L, Gold Hill 10.6 ug/L, and Dodge Bridge 8.5 ug/L. Three of the five sites, including Grants Pass, recorded their maxima during April. Dodge Bridge chlorophyll-a peaked during May, and Lobster Creek's maximum occurred in October.

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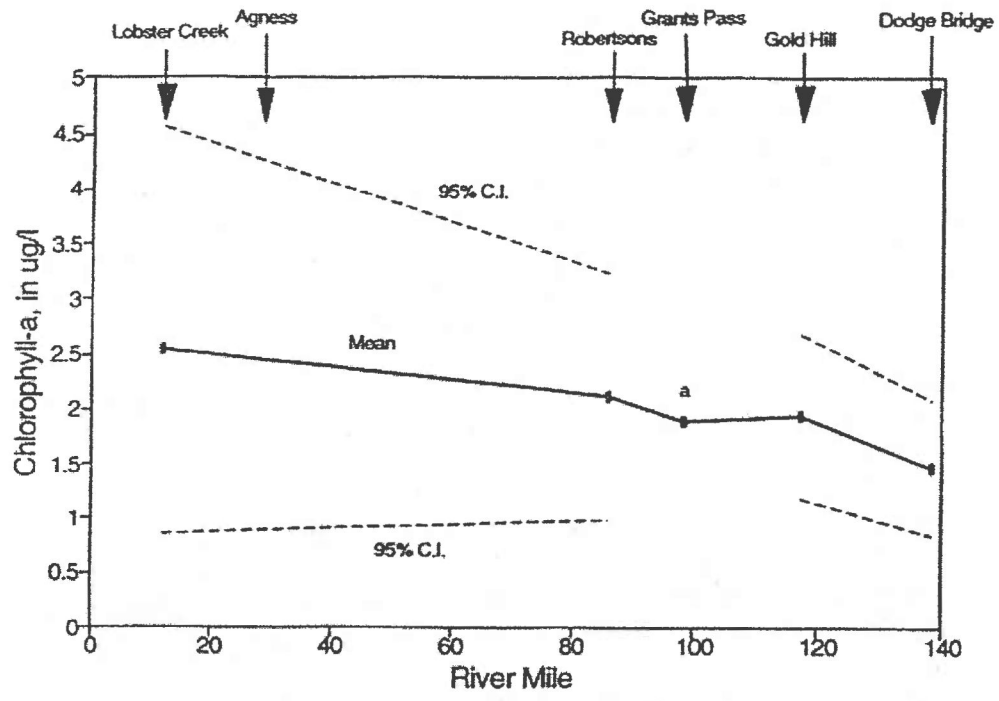
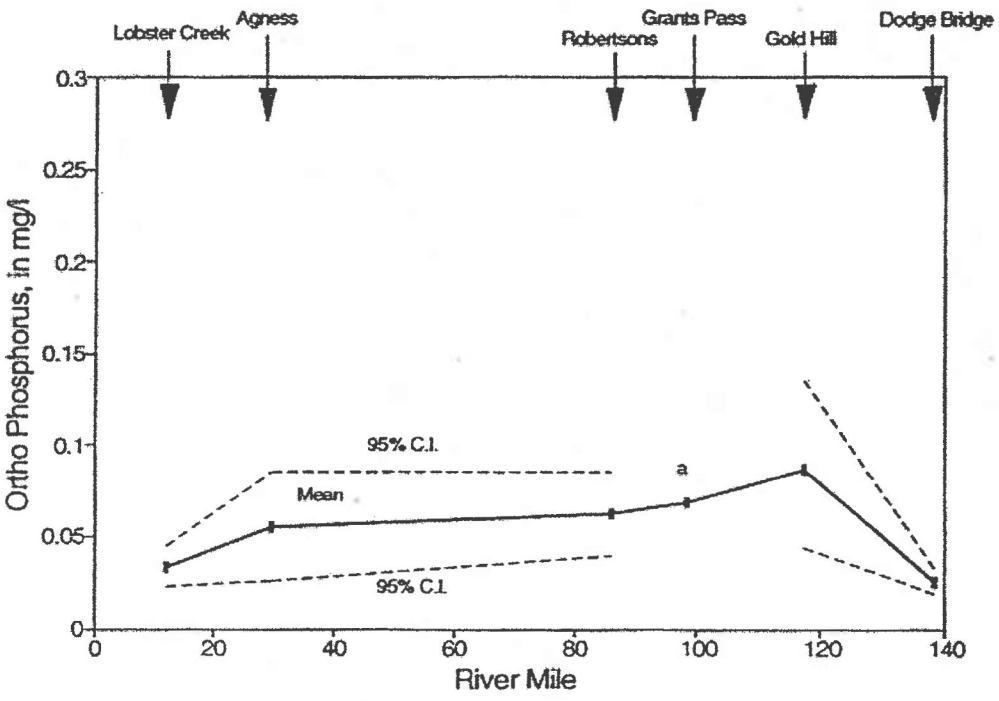
1 USGS. Water Resources Data for Oregon. Volume 2. Western Oregon. Water Years 1980-1990.

2 USGS OR-80-2. Water Resources Data for Oregon. Volume 2. Western Oregon. Water Year 1980.



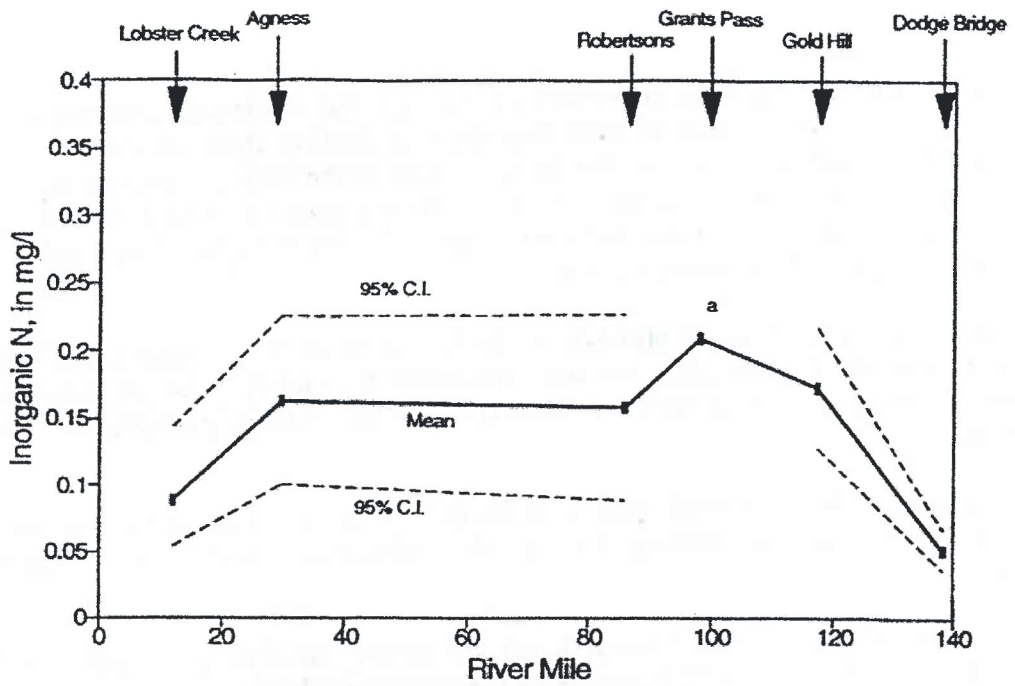
a. 95 percent Confidence Interval (C.I.) not shown because of limited data at this sampling station

Figure 6-15. Summer Chlorophyll-a



a. 95 percent Confidence Interval not shown because of limited data at this sampling location

Figure 6-16. Summer Ortho Phosphorus



a. 95 percent Confidence Interval not shown because of limited data at this sampling location

Figure 6-17. Summer Inorganic Nitrogen

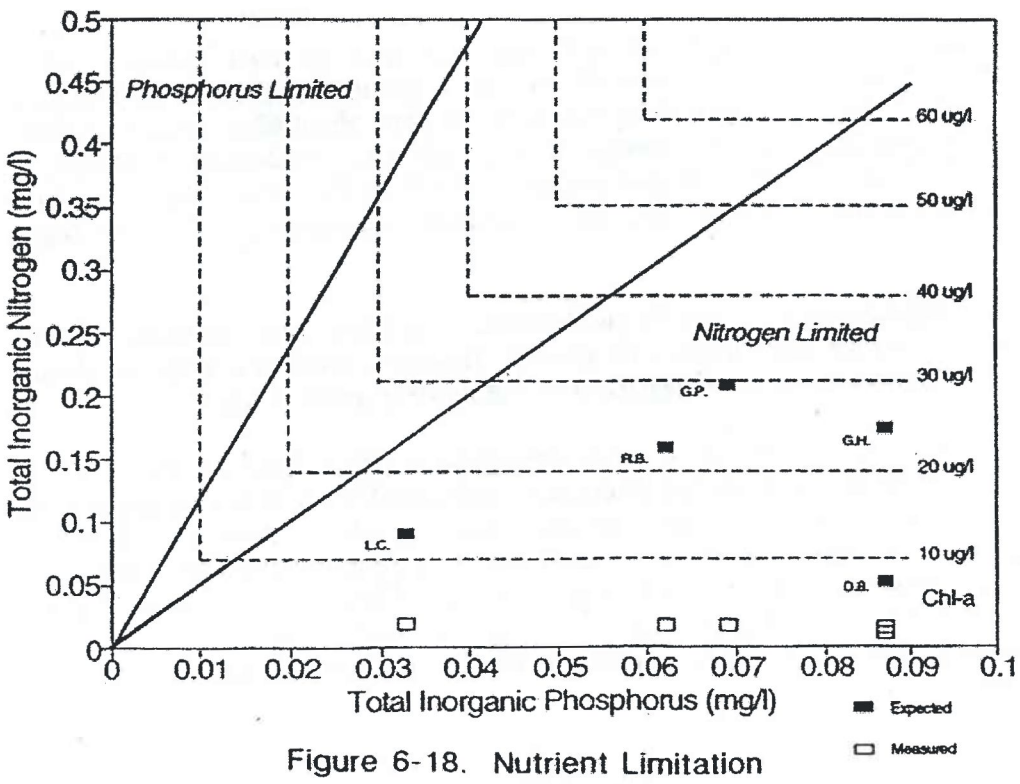


Figure 6-18. Nutrient Limitation

The less-than-expected chlorophyll-a concentrations indicate that while nitrogen may be the deficient nutrient, some other factor is more important in limiting algal abundance. Some preliminary modeling efforts indicate that the limiting factor is the short residence time of the algae in the river. It would take the algae four to five days to reproduce enough to use up all of the nitrogen present in the river at the upstream stations. By that time, the algae would have been swept over 200 miles downstream and out to sea.

Nutrient uptake by pheriphyton could also explain the low chlorophyll-a concentrations. Since pheriphyton are attached to rocks they are not represented by chlorophyll-a sampling data. However, we are aware of no data available to demonstrate the extent of pheriphyton growth in the Rogue River.

The nutrients present in the river come from a variety of sources. For comparative purposes, it was assumed that nutrient concentrations in Grants Pass effluent are similar to those from the Medford plant.

If the concentrations of nutrients in the two effluents are similar, calculations indicate that the Grants Pass WRP contributes about 12.5 percent of the phosphorus load present in the river at River Mile 98, while the Medford treatment plant is responsible for approximately 39.5 percent of the load at this point. A mass balance which considered the Medford plant, Bear Creek, the Grants Pass WRP, and natural conditions (phosphorus levels at Dodge Bridge) predicted that the annual average total phosphorus concentration below Grants Pass would be 0.092 mg/L; this compares to the measured average of 0.122 mg/L. Considering the presence of nonpoint sources, these values are in reasonably close agreement.

Medford plant effluent averages about 10 mg/L ammonia and 10 mg/L nitrate. These concentrations indicate that the Grants Pass WRP contributes about 20 percent of the inorganic nitrogen load present in the river at RM 98 and the Medford plant about 62.5 percent. A mass balance calculation predicted that the annual average nitrogen concentration would be 0.200 mg/L. This compares to the measured average of 0.208 mg/L. This close fit indicates that the mass balance accounts for all the major sources of inorganic nitrogen entering the Rogue River.

Effects of Future Discharges on River Phytoplankton. As plant flow increases, nutrient concentrations in the river are likely to increase as well. However, algal concentrations should not increase because the residence time appears to be the limiting growth factor.

Nutrient concentrations since 1981 show some interesting trends (Figures 6-19 to 6-24). Average annual ortho phosphorus at Dodge Bridge declined from 1981 to 1984, then rose until 1988 concentrations were once again similar to those found in 1981. At Gold Hill, ortho phosphorus has shown only minor fluctuations since 1981. Ortho phosphorus concentrations at Robertson's Bridge, however, have shown a steady increase. They have more than doubled since 1981. The trend in nitrogen concentrations is similar. Little or no upward trend has occurred at Dodge Bridge and Gold Hill, while a marked increase has been measured at Robertson's Bridge.

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Inadequate data exist to compare chlorophyll-a trends on a monthly basis, but three of the sites have enough data to compare trends based on yearly averages from 1981-1989. As shown in Figure 6-25, averages at all three sites, Dodge Bridge, Gold Hill, and Robertson's Bridge, have declined since the first year of available data. However, Dodge Bridge and Lobster Creek show no clear trend since the first year's decline. Only Gold Hill has shown a relatively consistent downward trend, beginning with 5.0 ug/L chlorophyll-a in 1981 decreasing to 1.3 ug/L in 1989.

Additional Parameters

Other water quality parameters regulated by the DEQ include liberation of dissolved gases, fungi, tastes and odors, sludges and bottom deposits, aesthetics and floating material, radioisotopes, concentration of dissolved gases, and total dissolved solids.

Standards. The current DEQ standards state that the following are not allowed:

1. The liberation of dissolved gases in sufficient quantities to cause objectionable odors or to be deleterious to fish or other uses.

2. The development of harmful fungi or other growths.

3. The creation of tastes or odors that affect the potability of drinking water or the palatability of fish.

4. The formation of appreciable bottom or sludge deposits harmful to aquatic life, public health, recreation, or industry.

5. Offensive aesthetic conditions, objectionable discoloration, scum, oily sleek, or coating of aquatic life with oily films.

6. Radioisotope concentrations shall not exceed maximum permissible concentrations in drinking water, fish, shellfish, wildlife, irrigated crops, livestock, or dairy products.

7. The concentration of total dissolved gas relative to atmospheric pressure shall not exceed 110 percent of saturation except when stream flow exceeds the ten-year, seven-day average flood.

8. Total dissolved solids concentrations shall not exceed 500 mg/L.

Current Rogue River Water Quality. The following are not believed to be problems in the Rogue River:

1. Fungi and other bottom growths.
2. Undesirable tastes or odors.
3. Bottom or sludge deposits.

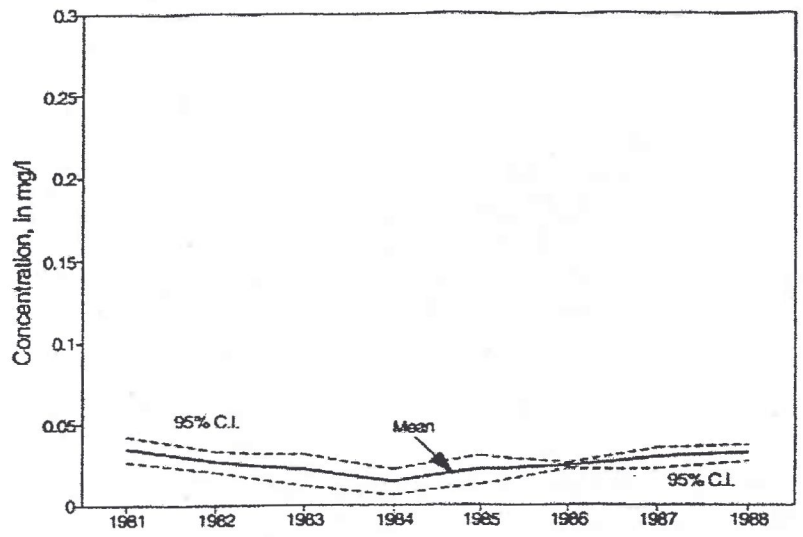

Figure 6-19. Yearly Ortho Phosphorus at Dodge Bridge

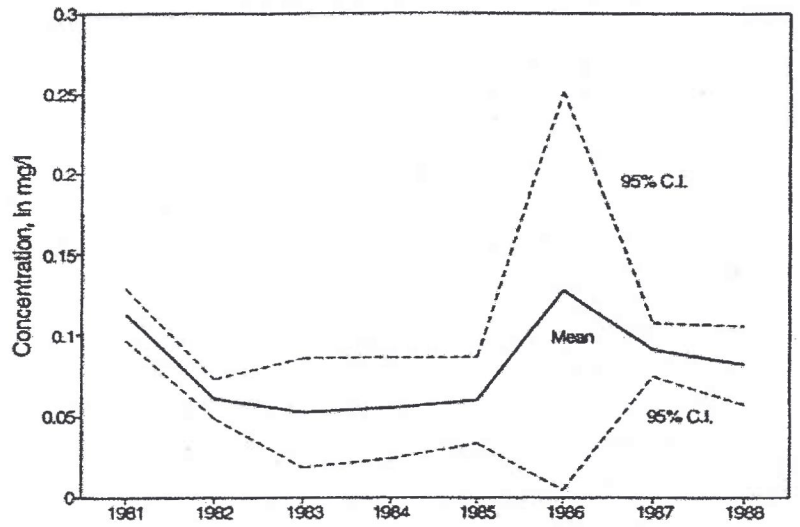

Figure 6-20. Yearly Ortho Phosphorus at Gold Hill

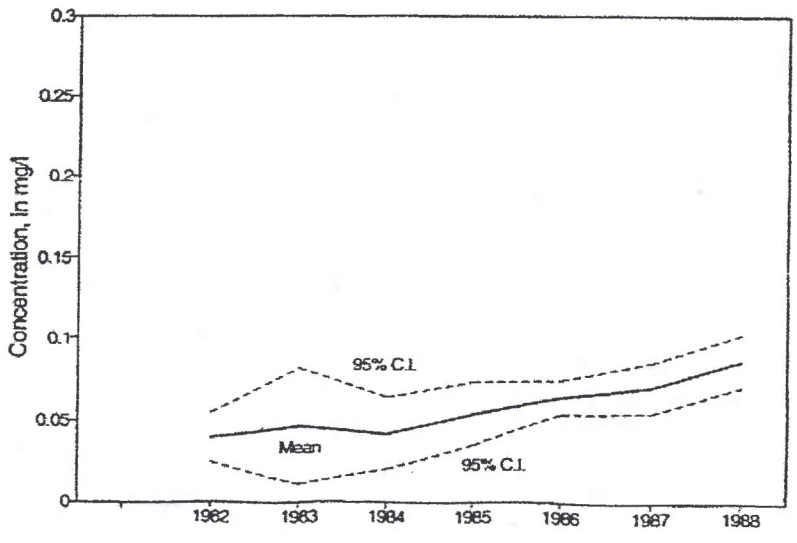

Figure 6-21. Yearly Ortho Phosphorus at Robertson's Bridge

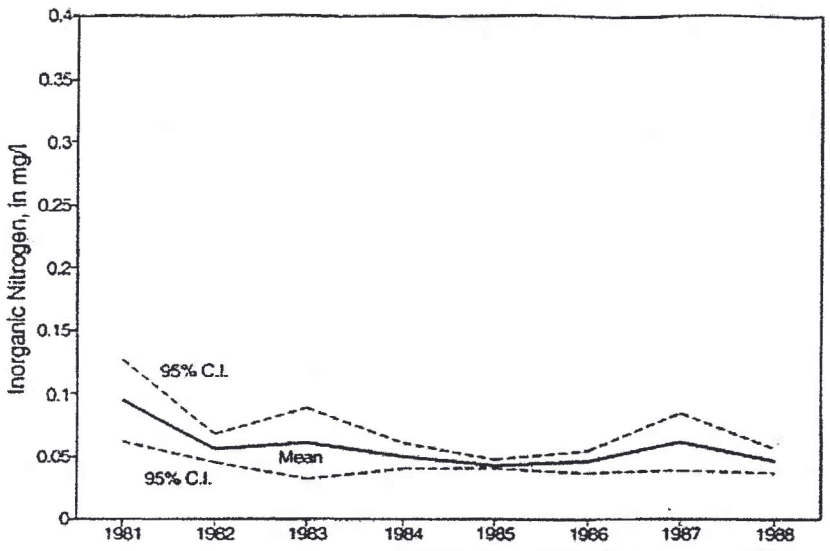

Figure 6-22. Yearly Inorganic Nitrogen at Dodge Bridge

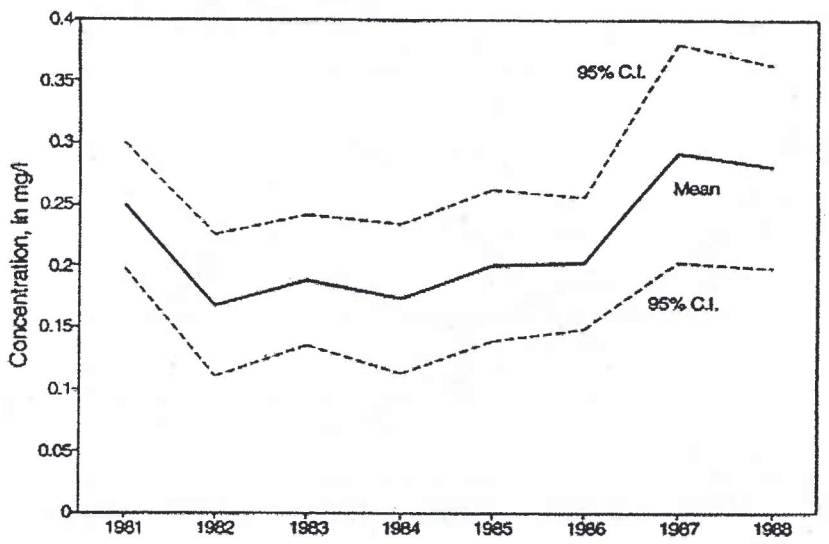

Figure 6-23. Yearly Inorganic Nitrogen at Goldhill

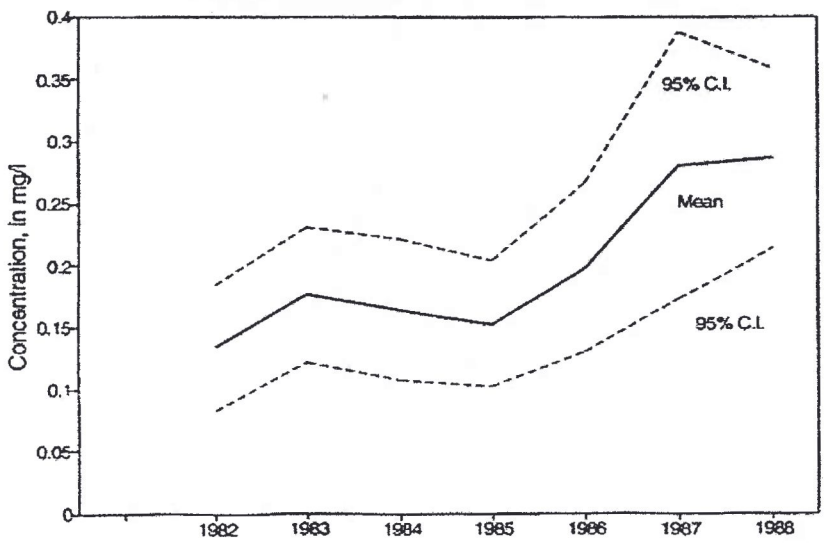

Figure 6-24. Yearly Inorganic Nitrogen at Robertson's Bridge

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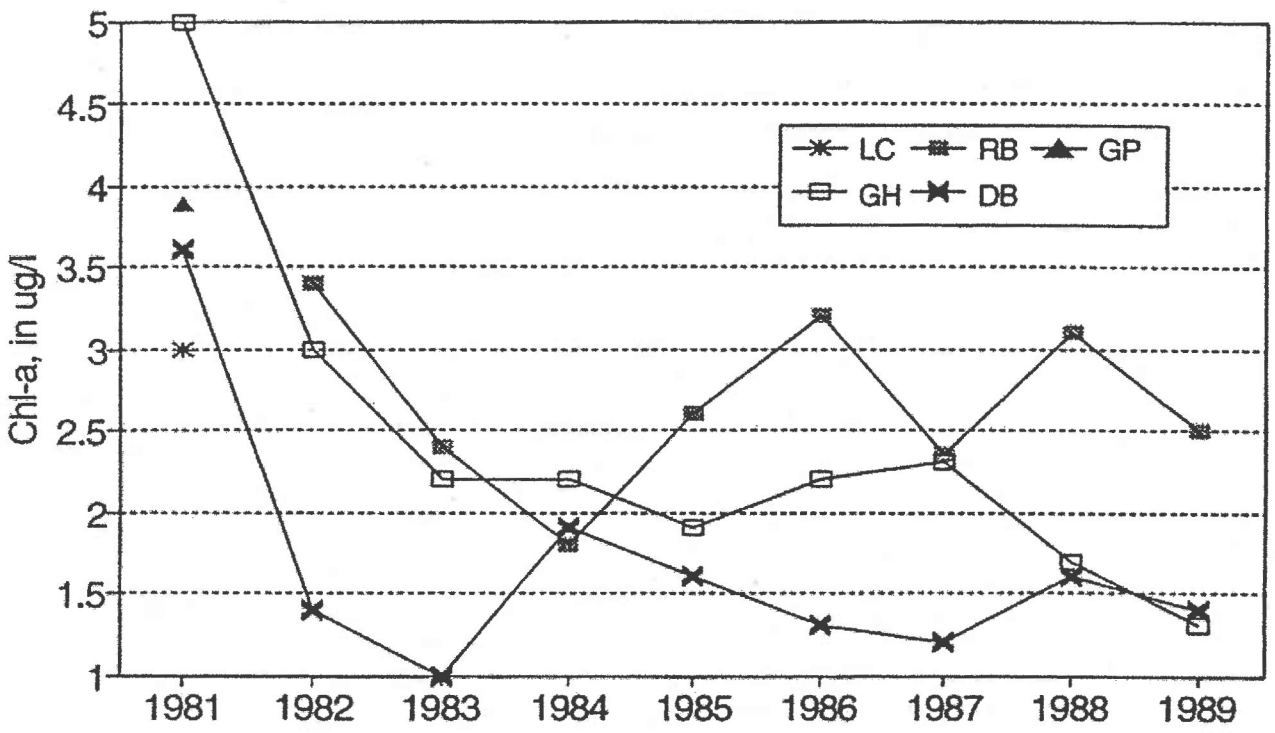

Figure 6-25. Chlorophyll-a Trends

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4. Radioisotopes.
5. The concentration of dissolved gases.

The concentration of total dissolved solids rarely exceeds 100 mg/L, so the Rogue River does not approach the DEQ recommendation of 500 mg/L total dissolved solids.

Effects of Future Discharges on Water Quality. The following are not expected to create problems with Rogue River water quality in the future:

1. Fungi and other growths.
2. Undesirable tastes and odors.
3. Bottom or sludge deposits.
4. Radioisotopes.
5. Total dissolved solids.
6. The concentration of dissolved gases.

TREATMENT CRITERIA

Minimum treatment requirements for wastewater discharges to the Rogue River are set forth in the OAR 340-41-375. The applicability of these requirements depends on a water quality evaluation. This section reviews the plant's current discharge permit and the OAR minimum treatment requirements and outlines proposed treatment criteria developed to protect the receiving stream water quality.

Current Discharge Permit

The plant's current discharge permit was issued in October 1992 and expired in 1997. The permit renewal is currently in progress. The current permit conditions, summarized in Table 6-4, are based on the design capacity of the existing treatment plant. The permit in affect may be revised when the plant is upgraded or expanded.

Minimum Treatment Requirements, OAR 340-41-375

Table 6-5 outlines the minimum treatment requirements set forth in the OAR. These requirements indicate that in the future the dry weather monthly average effluent BOD and total suspended solids (TSS) will be reduced from the current level of 20 mg/L to 10 mg/L. During the wet weather season, when the river's water quality is less sensitive to the plant discharge, the treatment criteria call for a minimum of secondary treatment. This means a monthly average BOD and TSS concentration of 30 mg/L and a minimum of 85 percent removal. The regulations also include the provision that more stringent treatment requirements may be imposed where required by special conditions. Maintaining water quality standards and preventing toxicity are conditions that often warrant more stringent treatment requirements.

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Table 6-4. Current Discharge Permit Requirements

Parameter	Average effluent concentrations, mg/L		Mass discharge, ppd		
	Monthly	Weekly	Monthly, avg.	Weekly, avg.	Daily max.
May 1 — October 31					
BOD	20	30	670	1,000	1,300
TSS	20	30	670	1,000	1,300
Fecal coliform	200[a]	400[a]	b	b	b
November 1 — April 30					
BOD	30	45	1,600	2,400	3,200
TSS	30	45	1,600	2,400	3,200
Fecal coliform	200[a]	400[a]	b	b	b

Other parameters:
1. pH — 6.0 - 9.0.
2. Average dry weather flow — 4.0 mgd.[c] Average wet weather flow — 6.4 mgd.
3. The mixing zone shall not exceed the portion of the Rogue River from 10 feet upstream to 300 feet downstream of the point of discharge.

[a] Number per 100 ml.
[b] Not applicable.
[c] The ADWF may be exceeded as long as the effluent conditions are not violated.

Table 6-5. Minimum Treatment Requirements

Parameter	Requirement
BOD and TSS	May 1 - October 31: monthly average not to exceed 10 mg/L unless more stringent limits required by waste load allocations in accordance with TMDL. November 1 - April 30: minimum secondary treatment or equivalent control and operation of waste treatment facilities at maximum practicable efficiency.[a]
Effluent dilution	Effluent BOD divided by the dilution factor[b] must not exceed 1.
Disinfection	Equivalent to a chlorine residual of 1 mg/L after 60 minutes contact.

[a] Monthly average 85 percent removal for BOD and TSS.
[b] Dilution factor is receiving stream flow divided by plant effluent flow.

Anticipated Future Treatment Criteria

This section outlines proposed treatment criteria applicable for the planning year 2020. The anticipated treatment criteria are intended to protect the Rogue River's water quality by maintaining compliance with the water quality standards outlined in OAR 340-41-375. The criteria presented below outline the basis for an increase in wet weather mass discharge limits for BOD and SS. In all instances, the anticipated future criteria are more stringent than the minimum treatment requirements.

Future Criteria: BOD and TSS. The DEQ's antidegradation policy for water quality is founded on OAR 340-41-026. This article states that high-quality waters shall be maintained and protected, and it is the policy of the Environmental Quality Commission that, unless otherwise approved, growth and development be accommodated by increased efficiency and effectiveness of waste treatment.

During the dry weather season, the plant effluent BOD and TSS should meet the monthly average concentration required by OAR 340-41-375 and the mass discharge limitations listed in the 1992 discharge permit. These criteria call for a monthly average effluent BOD and TSS concentration of 10 mg/L and a mass discharge of 670 ppd. The anticipated treatment criteria, outlined in Table 6-6, show that the plant would be designed to achieve a monthly average BOD and TSS of 10 mg/L and, to meet the mass discharge limit, a weekly maximum level of less than 15 mg/L.

Table 6-6. Anticipated Year 2020 BOD and TSS Treatment Requirements

Condition	Flow, mgd	Effluent concentration, mg/L			Mass discharge, ppd			Percent removal monthly, ppd[b]
		Monthly	Weekly	Daily	Monthly	Weekly	Daily	
Permit								
May-Oct	8	10	15	--	670	1,000	1,300	
Nov-Apr	10	30	45	--	2,500[a]	3,750	5,000	
Treatment criteria								
May-Oct								
ADWF	8	10[c]	--	--	670			1,800
Max. month	11	7[c]	--	--	670			1,800
Nov-Apr								
AWWF	10	22[d]	--	--	2,500			1,800
Max. month	16	13[d]	--	--	2,500			1,800
Max. week	24	--	19[c]	--		3,750		
Max. day	32	--	--	19[c]			5,000	

[a] Computed at AWWF of 10 mgd and effluent concentration of 30 mg/L.
[b] Percent removal equivalent to current permit mass discharge limit. Computed on the basis of 85 percent removal of average BOD load: 12,100 ppd.
[c] Concentration limited by mass discharge.
[d] Concentration limited by percent removal requirement.

During the wet weather season, when river flow is high and the water quality is least sensitive to the plant discharge, the proposed treatment criteria focus on protecting the river's water quality. Our analysis of the water quality data shows that the current plant discharge has virtually no effect on the dissolved oxygen levels of the Rogue River. As discussed previously,

the Streeter-Phelps oxygen sag model demonstrated that an 7Q10 flow of 888 cfs, a mass discharge of 670 ppd BOD would depress the DO about 0.16 mg/L. During wet weather, when the stream flows are two to three times greater, the effect of the plant discharge on dissolved oxygen would not be measurable. Furthermore, during wet weather the river's dissolved oxygen levels always greatly exceed water quality standards.

Since the river dissolved oxygen levels are at their highest during wet weather and the plant discharge has no measurable effect on dissolved oxygen levels, it is appropriate that the treatment criteria be based on the secondary treatment requirements set forth in OAR 340-41-375. The Clean Water Act defines secondary treatment requirements as a monthly average concentration for BOD and TSS of 30 mg/L and a weekly average concentration of 45 mg/L. The current discharge permit contains these secondary treatment limits for the wet weather season.

The wet weather mass discharge limits in the current discharge permit (Table 6-4) were established, in accordance with OAR 340-41-120, on the basis of the plant's average wet weather flow (AWWF) and an effluent BOD and suspended solids (SS) concentration of 30 mg/L. OAR 340-41-120 states that for new or expanded treatment facilities receiving approval after June 30, 1992, the mass load limits shall be based on the facility capabilities and the highest and best practicable treatment to minimize the discharge of pollutants.

The wet weather treatment criteria outlined in Table 6-6 are limited by both the allowable mass discharge limit and the allowable percent removal for BOD and SS. The wet weather mass discharge limit presented in Table 6-6 is computed, in the same manner as the current permit, on basis of the plant's design AWWF and an effluent concentration of 30 mg/L.

However, due to the high wastewater flows during wet weather, the plant's required monthly average performance is actually dictated by the minimum percent removal requirement for BOD and SS. The current permit is based on meeting 85 percent removal. At the year 2020 average BOD and SS load, 12,100 ppd and 14,100 ppd, respectively, 85 percent removal would correspond to a BOD limit of 1,815 ppd and SS limit of 2,115 ppd. Using the (lower) BOD limit to set plant performance, during the maximum month flow of 16 mgd, 85 percent removal would correspond to an effluent concentration of 13 mg/L BOD.

Table 6-6 also summarizes the plant's required performance at more extreme flow conditions. At maximum week flow the mass discharge limit corresponds to an effluent concentration of 19 mg/L. At maximum day flow the mass discharge limit also corresponds to an effluent concentration of 19 mg/L.

The proposed wet weather mass discharge limits exceed those listed in the current permit. OAR 340-41-026 allow for increases in the mass discharge providing that:

(A) No other reasonable alternatives exist except to lower water quality;

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(B) The action is necessary and justifiable for economic or social development benefits and outweighs the environmental costs of lowered water quality;

(C) All water quality standards will be met and beneficial uses protected.

The Rogue River is classified by the DEQ as water quality limited for temperature and pH. However, during the wet weather season, the dissolved oxygen levels always exceed the water quality standards. An increase in BOD mass discharge would have virtually no effect on wet weather dissolved oxygen levels, water quality standards, or beneficial uses. Furthermore, the current mass discharge levels were not established on the basis of water quality protection, but rather on the basis of available treatment technology and wastewater flow. Therefore, to accommodate future wastewater flows while protecting water quality, a reasonable increase in the wet weather BOD mass discharge is warranted.

Future Criteria: Nutrients. Currently there is no nutrient limitation imposed in either the Grants Pass wastewater treatment plant or any other plant discharging to the Rogue River. The purpose of controlling nutrient levels is to prevent the growth of nuisance phytoplankton. The water quality standards have established an average of 15 ug/L of chlorophyll-a as an indication of excessive algae growth. Chlorophyll-a levels in the Rogue River are well below the water quality standard. Our analysis of the water quality data clearly shows that nowhere on the river have the average chlorophyll-a levels exceeded 3 ug/L. On only a few isolated days have levels been reported that even approach the standard. Also, the data collected since 1981 show no clear trend of increasing chlorophyll-a concentrations. Therefore, since the river is accommodating the current nutrient load without creating nuisance algal growth, there is no need to impose treatment requirements intended to reduce nutrient levels.

Further examination of the water quality data indicate that the chlorophyll-a levels are actually lower than one would expect for the reported nutrient concentrations. This indicates that algal growth may be limited by factors other than nutrient concentration. One example could be that the short travel time to the river mouth limits the opportunity for algae to grow. On the basis of the existing water quality data, we cannot define the relationship between nutrient levels and algae growth in the Rogue River. A detailed comprehensive river study would be necessary to identify what factors actually limit algae growth and to determine what nutrient levels the river can assimilate without detrimental water quality effects.

Recognizing that the Grants Pass wastewater treatment plant is a major contributor of nitrogen and phosphorus to the Rogue River and that it is essential to protect the river's water quality, we propose that facilities planning be based on the following approach.

1. No specific nitrogen or phosphorus discharge limits should be imposed for the planning period.

2. Future discharge permits should require that the plant report effluent nitrogen and phosphorus concentrations.

3. Algae levels in the river should be closely monitored for any trend for increasing chlorophyll-a levels. A detailed river study and modeling effort should be conducted to define the acceptable nutrient load to the river.

4. The recommended upgrade and expansion plan should anticipate that nitrogen and phosphorus removal may be added in the future if further studies indicate that they are necessary.

5. Because the river does not currently approach the chlorophyll-a action limit, the plant's current nutrient discharge level should be taken as an acceptable minimum discharge level.

Future Criteria: Toxicity. Residual chlorine and ammonia are constituents that have clearly identified toxicity levels. Recent plant modifications have replaced chlorination with UV disinfection, eliminating chlorine toxicity in the WRP effluent.

Ammonia toxicity will also be regulated. However, the city has three options for complying with ammonia toxicity standards. Operation of the WRP could be modified to perform nitrification, converting ammonia residuals into nitrates. Alternatively, a new outfall could be constructed to improve mixing and reduce ammonia concentrations within the mixing zone. Or, the city could implement an effluent reuse program, eliminating effluent discharge during the summer (see Chapter 7).

Future Criteria: Disinfection. A new DEQ restriction[4] will limit the concentration of E. coli bacteria in WRP effluent. The new limit is expected to be a 30-day log mean of 126 organisms per 100 mL sample averaged over 5 samples, with no single sample exceeding 406 organisms per 100 mL. Very limited data taken in January, February, and March of 1997, immediately after the new UV disinfection equipment came on-line is insufficient to determine whether this limit is being met.

Future Criteria: Temperature. The previous water quality analysis illustrated the significant effect that river temperature has on the river's dissolved oxygen concentration. Since the river temperature routinely exceeds 58 degrees F during warm weather, it is clear that the wastewater treatment plant discharge will exceed the water quality standard for temperature rise. Rather than imposing a plant effluent temperature standard, we propose that the facility plan address the temperature issue by considering alternatives to effluent discharge during warm weather (see Chapter 7 on reuse) and by investigating measures that would mitigate the temperature rise, or the installation of a new diffuser to reduce WRP effluent temperature effects. A Surface Water Temperature Management Plan, as outlined in a recent DEQ guidance document, may be necessary in the future.

References

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CHAPTER 7

FEASIBILITY FOR EFFLUENT REUSE

Reclaimed water (treated effluent) from the Grants Pass Water Restoration Plant (WRP) can be used beneficially instead of being discharged. Low river levels coinciding with the period of peak crop water use make irrigation a potentially feasible alternative for the City of Grants Pass and local agricultural water users. Reclaimed water can irrigate agricultural land, parks, highway landscaping, and golf courses. It can be used to grow wood fiber for fuel, pulp, or lumber. Other potential uses include increased flow to wetlands and storage in reservoirs or other impoundments.

An extra incentive to "get out of the river" during the summer months is provided by concerns over meeting temperature requirements, potential future nutrient limitations, and listing of certain fish species as endangered. Reclaimed WRP effluent can also be viewed as a valuable resource which could help bridge a gap created by the planned removal of the Savage Rapids Dam. Dam removal has sparked a controversy about the availability of irrigation water, and new costs associated with pumping irrigation water from the Rogue River.

This chapter evaluates quality criteria, legal restrictions, and options available for beneficial use of reclaimed water during the summer months. The scale of operations in terms of present and future land and storage needs is explored, and land suitable for such irrigation is identified.

REGULATIONS GOVERNING EFFLUENT REUSE

Oregon law permits beneficial reuse of effluent from wastewater treatment facilities (Oregon Administrative Rules, Chapter 340, Division 55—OAR 340-55). These rules are administered through the Oregon Department of Environmental Quality and are designed primarily to protect public health by minimizing risks of infection and disease transmission. There are also safeguards to protect surface water and groundwater resources.

The quality of plant effluent intended for irrigation depends on chemical and biological characteristics. Reclaimed water quality is divided into four levels according to treatment, total coliform, and turbidity. The levels are defined as follows:

- Level I reclaimed water, undergoing only biological treatment, has no limits on total coliform or turbidity. No sampling is required for quality monitoring.

- Level II reclaimed water must undergo biological treatment and disinfection. Limits must be met for total coliform bacteria count but not turbidity. Samples must be collected once a week for coliform monitoring.

- Level III reclaimed water must receive the same treatment processes as Level II, but it has stricter coliform limits. Samples must be collected three times a week to monitor coliform bacteria.

- Level IV reclaimed water must receive biological treatment, disinfection, clarification, coagulation and filtration to meet strict turbidity as well as coliform limits. Coliform samples must be collected once a day, and hourly turbidity readings are required.

Production and use of Level II, III, and IV reclaimed water are alternatives for the WRP. These water quality levels allow various reuse options. Although more expensive because of the stricter treatment requirements and quality control, Level IV water is the most desirable for reclaimed water use. Table 7-1 outlines the specific treatment and monitoring requirements and reuse restrictions for the different levels.

Table 7-1. Treatment and Monitoring Requirements for Use of Reclaimed Water

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

Source: OAR 340-55-015, Table 1.

Level II reclaimed water is suitable for irrigating pasture, alfalfa, and silage corn as long as no irrigation is done in the 3 days before harvest, and no animals are in the field during irrigation. Crops consumed directly cannot be irrigated with Level II reclaimed water. Surface (furrow or flood) irrigation of processed food crops and orchards is allowed providing that the edible portion of the crop does not touch the ground, and fruit or nuts are not harvested off the ground. Irrigation of golf courses without contiguous residences is allowed as long as warning signs are posted, and irrigation water is kept at least 100 feet from food preparation or serving areas and drinking fountains. The same restrictions apply to highway medians, cemeteries, and areas with limited public access. Irrigation of parks, schoolyards, and golf courses with houses nearby is prohibited. A buffer of 70 feet is required between land irrigated with Level II water and other fields or property. Impoundments of Level II water are limited to "landscape impoundments," which do not allow such activities as boating, fishing, or body-contact recreation. Storage lagoons for Level II water would have to be well-secured to prevent unauthorized public access.

The same crop restrictions and beneficial reuse requirements apply to Level III water with the following exceptions: 1) buffer distances are reduced to 10 feet for all types of irrigation; 2) fewer liability concerns would be associated with a storage facility containing Level III reclaimed water; and 3) impoundments of Level III reclaimed water may be used for boating, fishing, and other nonbody-contact recreational activities [OAR-340-55-010 (171)].

There are few restrictions on the use of Level IV reclaimed water. All types of crops may be irrigated with no restriction on method. Golf courses, schoolyards, and parks also may be irrigated. Reclaimed water is restricted from food preparation areas and drinking fountains. Signs must be posted in public-access areas to inform the public of the reclaimed water use. In addition, impoundments or storage facilities containing Level IV water can be used for recreational purposes, including those involving body contact with the water.

Buffer zones imposed around fields irrigated with Level II water make it the least adaptable alternative for effluent reuse. Level IV water is the most flexible and expensive to produce. Level III reclaimed water is of similar quality but less expensive to produce than Level IV water, and would involve only minor modifications to the WRP facilities which already produce Level II water. Short buffer distances and fewer liabilities regarding public access of storage impoundments make Level III water the most viable option for reclaimed water use.

GRANTS PASS EFFLUENT CHARACTERISTICS

Current average dry weather flow (ADWF) is 4.5 million gallons per day (mgd). By the year 2020, the ADWF is expected to reach 8.0 mgd (see Chapter 4). These flow rates, equivalent to 13.8 and 24.5 acre-feet per day for the years 1996 and 2020, respectively, have been used to estimate the amount of reclaimed water that will be available for beneficial use.

Twenty-four-hour composite samples of plant effluent were collected on June 13, 18, 20, 26, and 28 in 1991 and analyzed by Neilson Research Corporation in Medford, Oregon. The test results are given in Table 7-2.

Table 7-2. Grants Pass Water Restoration Plant Effluent Analysis

Test	Units	Sample dates					Mean	Pounds/ acre-foot
		6/13/91	6/18/91	6/20/91	6/26/91	6/28/91		
Total Kjeldahl nitrogen	mg/L	12.38	11.37	9.52	13.50	12.21	11.80	32
NH_3-nitrogen	mg/L	11.26	11.26	9.35	11.76	10.75	10.88	29
NO_2-nitrogen	mg/L	0.09	<0.01	<0.01	0.05	0.11	0.05	0
NO_3-nitrogen	mg/L	0.14	<0.1	<0.1	0.12	<0.1	0.05	0
Total phosphorus	mg/L	2.40	2.11	1.49	2.93	2.84	2.35	6
Turbidity	NTU	2.0	2.9	4.6	2.6	3.4	3.1	—
Na	mg/L	39.33	40.20	37.40	39.30	36.36	38.52	104
K	mg/L	6.43	6.97	6.55	6.88	7.12	6.79	18
Ca	mg/L	2.55	2.70	2.56	2.79	1.94	2.51	6
Mg	mg/L	9.11	8.62	8.62	9.39	8.10	8.77	23
Electrical conductivity	μmho/cm	380	390	375	370	390	381	—
Sodium adsorption ratio		2.57	2.68	2.50	2.52	2.54	2.56	—
BOD-5 day (uninhib)	mg/L	38.61	27.18	26.07	28.38	25.43	29.13	79
Total coliform	org/100 mL	17	130	540	920	240	192	—
Fecal coliform	org/100 mL	—	2	<2	2	33	33	—

Low specific conductivity (381 μho/cm) and sodium adsorption ratio (2.56) are two reasons why the effluent is an excellent irrigation water. Due to low salts or sodium, this effluent should have little long-term detrimental effect on soil productivity. The nitrate level (NO_3-N) is very low at 0.05 milligrams per liter (mg/L). Total Kjeldahl nitrogen is 11.80 mg/L, with 92 percent in the ammonia form. Respective loading rates of nitrogen and phosphorus at 32.2 and 6.4 pounds per acre-foot of reclaimed water would not limit application to most crops. Biochemical oxygen demand of 29 mg/L should not result in odor problems during storage and would not limit use as irrigation water. The total coliform data shown do not allow a complete comparison with regulatory standards. However, it is likely that some improvements in treatment would be needed to meet the requirements for Level II or III effluent routinely. Major additional treatment costs would be required to meet the requirements of Level IV treatment.

AGRICULTURAL CONSTRAINTS

Use of treated effluent for irrigation is a proven technology with clear benefits for water resources conservation. Still, there are several issues that must be addressed regarding the feasibility of effluent reuse in the Grants Pass area. These include the suitability of soils and topography in the area, local climate, current agricultural practices, and need for supplemental irrigation water.

Climate

Chapter 2 studies the climate in the Grants Pass area. Growing season conditions, which influence crop water use, are a concern in reclaimed water irrigation. Peak crop water use occurs during June, July, and August when crops are actively growing and temperatures are the hottest. Warm, dry weather prevails from May through September; the mean maximum daily temperatures during those months are 65.1, 70.8, 75.5, 76.5, and 69.2 degrees F, respectively. Eleven percent of the average annual rainfall occurs during these months, with June, July, and August accounting for only four percent of the average. Early in the growing season, precipitation and water stored by the soil can meet crop water needs. Later in the season, supplemental irrigation water is needed for optimum production.

Soils

In general, the soil resources of the Grants Pass area are well-suited for agriculture in areas where slope and soil depth are not limiting (Borine, 1984). The chemical nature of most soils in agricultural areas is good, and the primary constraint on productivity is the lack of growing season water supply. Therefore, this assessment focuses on the water supply, management, and soil properties that affect irrigation.

Once it has been released for beneficial use, discharge of reclaimed water into surface waters is prohibited under OAR 340-55-015. Runoff of irrigated reclaimed water must be prevented. Runoff control strongly depends on management practices. Slope, soil permeability (the ability of the soil to absorb water), and soil drainage are important factors in evaluating land for irrigation with reclaimed water. Potential for runoff increases with slope and decreases with soil permeability. Level land with rapidly permeable soils has little potential for irrigation water runoff. Soil drainage refers to the amount of time a soil remains saturated with water. Poorly drained soils tend to pond water and remain saturated, thus increasing the likelihood of runoff. Moderately well-drained soils yield maximum crop productivity, hence maximum crop water use. Higher crop water use, in turn, reduces the acreage required for reclaimed water use.

Figure 7-1 is an overview of the soil resources of the Grants Pass area. Agricultural soils suitable for reclaimed water irrigation are located in the narrow river valleys of the Rogue River and its tributaries, the Applegate River and Jumpoff Joe Creek. For the most part, the local agricultural soils are moderately coarse- to medium-textured and moderately well- and well-drained, with adequate permeability. The dominant soils are Newberg fine sandy loam, Camas gravelly sandy loam, Evans loam with smaller amounts of Kerby loam, Central Point sandy loam soils and Takilma cobbly loam (Figure 7-1, map units 1 and 2). These soils are deep, well- to excessively well-drained, and possess moderately rapid to rapid permeability on 0 to 2 percent slopes. They are well-suited for irrigation.

West of the Grants Pass urban growth boundary (UGB), the floodplain soils are predominantly Newberg fine sandy loam soils. Small areas of Camas gravelly sandy loam and Wapato silty clay loam are scattered in isolated high and low areas, respectively. At the confluence of the Rogue and Applegate rivers is a parcel with large fields of irrigated pasture. The soils are

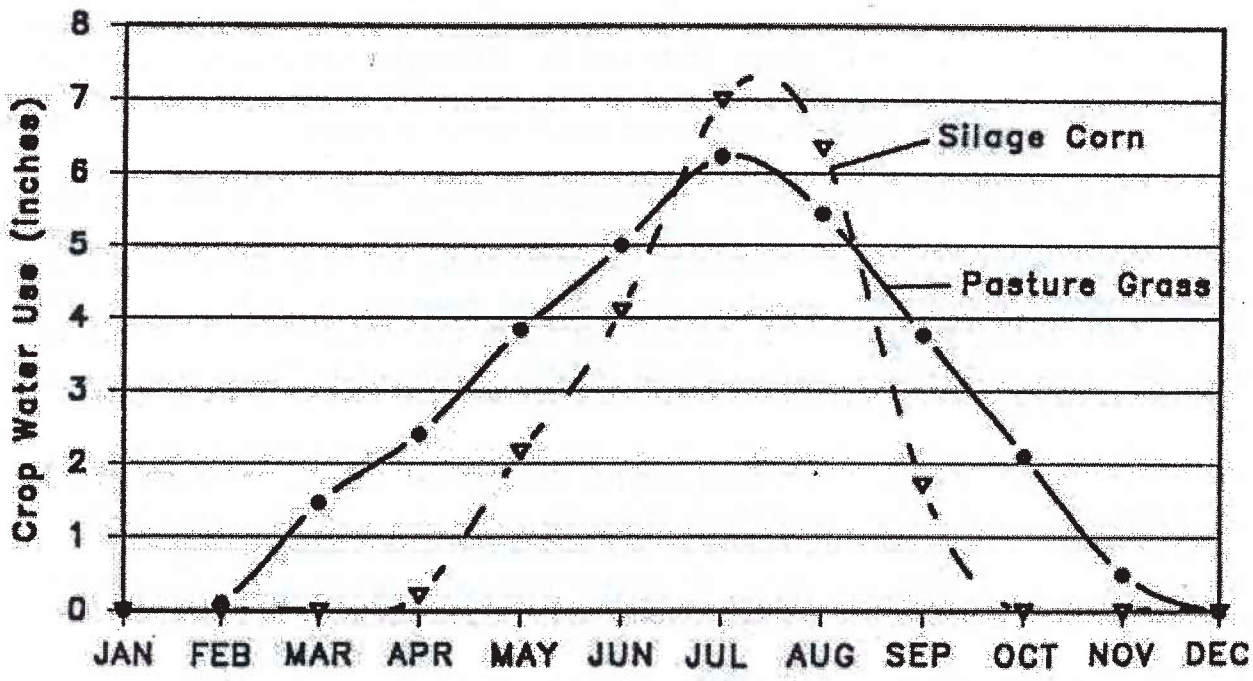

FIGURE 7-2
MONTHLY CROP WATER USE

GRANTS PASS WATER RESTORATION
PLANT FACILITIES PLAN
Grants Pass, Oregon
September 1991

CASCADE EARTH SCIENCES, Ltd.
PN 912024

Newberg sandy loam and Camas gravelly sandy loam adjacent to the riverbanks, with more finely textured, well-drained, moderately permeable Evans loam, Kerby loam, and Central Point sandy loam toward the interior of the property. The land at Everton Riffle is primarily Kerby loam and Evans loam. These are excellent soils for irrigation.

South of Grants Pass are large areas of somewhat poorly drained Clawson and Jerome sandy loam soils with moderately rapid permeability (map unit 3). In the Fruitdale area are well-drained soils with moderately slow permeability, dominated by Manita loam and Ruch gravelly silt loam with slopes of 2 to 12 percent (map unit 4). Although these soils are suitable for irrigation, they are more slowly permeable than the soils west of Grants Pass and would require more careful management to reduce ponding and runoff during irrigation.

The soils in the Merlin area are dominated by moderately well-drained, very slowly permeable Brockman cobbly clay loam with well-drained, moderately permeable Manita loam and well-drained Abegg gravelly loams with moderately slow permeability (map unit 5). These soils are on slopes ranging from 2 to 20 percent and require careful management to reduce runoff. The Takilma and Foehlin soils along Jumpoff Joe and Louse Creeks are mostly well-drained, moderately permeable Takilma cobbly loam and Foehlin gravelly loam. These soils are well-suited for irrigation.

In summary, the area's valley bottom and floodplain soils are well-suited for irrigation. Some management to control surface runoff would be required, especially in areas with more than a 5 percent slope. Because these soils occur in low-lying areas, groundwater is likely to rise to shallow depths in the winter and spring. Controlled irrigation would protect against potential groundwater impacts and ensure that groundwater levels do not rise in the local area.

Crop water use requirements vary with time, depending on temperature, humidity, crop type, and crop maturity (Figure 7-2). Most crops in the Grants Pass area require some form of irrigation during July and August when irrigation water needs are greatest. The actual length of irrigation time varies by crop from 9 months for pasture grass to less than 4 months for some annual crops (Kanalz, 1984).

Agricultural irrigation needs for the Grants Pass area are supplied by the Grants Pass Irrigation District (GPID), Fort Vannoy Irrigation District (FVID), and pumping from wells, creeks, and rivers in the area. Several of the larger landholders pump their own water directly from the Rogue or Applegate rivers. Irrigation practices are split between sprinkler and surface (flood) applications. The type of irrigation often depends on the water delivery system. Piped, pressurized delivery systems are necessary for sprinkler irrigation. Private pumped systems in the Grants Pass area are often sprinkler systems. GPID and FVID deliver by ditch systems. Their users generally practice flood irrigation since water is delivered by nonpressurized gravity flow. The planned removal of the Savage Rapids Dam will require the GPID to switch over to a pressurized delivery system which will pump water from the Rogue River to irrigated lands.

The agricultural community of the Grants Pass area consists of many small "hobby" or part-time farming operations. Only 12 of the 7,700 members of the GPID have holdings greater than 40 acres. Few large farms or fields are available.

The FVID consists of 700 acres comprised of at least six fields of 20 to 60 acres. Major land holdings in the FVID are the Wild River Farms apple orchard and Ken-Wal Farms. Although they are part of the FVID, some or all of these lands are irrigated privately with water pumped directly from the Rogue River.

Crops

The predominant mix of crops in the Grants Pass area is perennial pasture, alfalfa, and silage corn. Other crops such as orchard crops and flowers are grown in lesser amounts. The most desirable option for reclaimed water irrigation is hay or pasture crops because of the flexibility. Either flood or sprinkler irrigation can be used. Public access to pasture or hay fields generally is limited to the land owner, and the crop is not for human consumption. This minimizes health concerns and maximizes flexibility in the amount and type of wastewater treatment. Furthermore, the perennial pasture or hay crops consume water throughout a long growing season because of their well-established root systems. Assuming 75 percent of the water delivered to a farm is available to the crop after evaporative and delivery losses (75 percent irrigation efficiency), alfalfa and pasture grasses require respective applications of approximately 35 and 29 inches of water per growing season in the Grants Pass area (Table 7-3). Sprinkler irrigation typically is 75 percent efficient. This does not account for losses in the delivery system between the treatment plant and the irrigation site.

Table 7-3. Estimated Monthly Gross Irrigation Requirements for the Dominant Crops of the Grants Pass Area[a]

	Alfalfa, inches	Pasture, inches	Silage corn, inches	Orchard w/cover, inches
April	0.5	0.5	0.0	0.5
May	4.8	3.8	0.0	4.8
June	7.4	6.0	4.6	7.4
July	9.7	8.1	9.1	9.8
August	8.2	7.0	8.2	8.2
September	4.4	3.5	0.0	4.5
October	0.0	0.0	0.0	0.0
Total	35.0	28.9	21.9	35.2

[a] Assumes 75 percent irrigation efficiency.
Adapted from Oregon Engineering Handbook Irrigation Guide (Kanalz, 1984).

Use of reclaimed water for irrigating freeway landscape flora or golf courses is an option. Irrigation must be managed to minimize the potential of human contact with wastewater, but public access is restricted to these areas.

Woodlots

Hybrid poplar woodlots are used for wastewater irrigation. This alternative "crop" is fast gaining a reputation for high water uptake. In addition, when properly selected and sited, stands of poplar provide site screening and a high yield of fiber in the form of sawlogs and/or wood pulp or chips in 8- to 15-year harvest cycles. One analysis[1] shows net annual return ranging between -$7 to $205 per acre (varies with product prices and management strategy). Opportunities for marketing poplar fiber in the Grants Pass area have not been researched at this time.

Irrigation requirements vary considerably with site conditions and irrigation methods. Data specific to Southern Oregon is currently being gathered, but 42 inches of water per growing season is a rough estimate for the Grants Pass area. This uptake is higher than pasture grasses or even alfalfa, allowing the employment of fewer acres and a smaller irrigation distribution system. However, it must be noted that poplar stands require 3 years to mature into their maximum uptake rate, and harvesting activities can take tracts out of irrigation for as much as a full year between harvest cycles.

Poplars have been proposed on a small-scale project to provide some on-site screening at the WRP. They would be irrigated with final effluent. A total of approximately 2 acres is under consideration. This project may provide a good opportunity to learn more about site-specific irrigation requirements, local markets for poplar wood, and ease of maintenance of poplar woodlots. Until more data is available, the WRP should focus on other reuse options.

Wetlands

Reclaimed water has been successfully used to improve degraded natural wetlands. Most developing areas of the United States have lost significant portions of their functioning wetlands due to human activities such as damming rivers and dredging, draining, and filling marshes. Recent concerns about wetlands losses have caused the formation of countless public/private partnerships with the goal of protecting and enhancing remaining wetlands. While an assessment of local wetland resources is not within the scope of this Facilities Plan, it may become of interest to the city in the future as an alternative WRP effluent reuse strategy. However, current Department of Environmental Quality rules do not allow discharge of treated effluent into natural wetlands. However, current DEQ rules do not allow discharge of treated effluent into natural wetlands.

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[1] Heilman, P.E., R.F. Stettler, D.P. Hanley, R.W. Carkner, 1995. *High Yield Hybrid Poplar Plantations in the Pacific Northwest.* Pacific Northwest Extension Bulletin PNW356.

000624

PRELIMINARY EFFLUENT REUSE SYSTEM NEEDS

The primary objective of the effluent reuse system is to eliminate the need to discharge to the river when the river temperature standard could be exceeded or if a future nutrient discharge limit is imposed. This period corresponds with the dry-weather season, May 1 through October 31.

Five crop mixes for effluent reuse were developed from typical crop type and acreage patterns in the Grants Pass area (Table 7-4). Acreages and storage requirements were calculated for 100 percent usage on alfalfa or pasture (the predominant local crops) and mixes which also included such other crops as silage corn and orchard crops. Perennial crops, such as pasture grasses, alfalfa, and orchards with ground cover, provide the optimum conditions for reclaimed water irrigation. Long annual periods of water use are possible since these crops are well-established early in the season and continue growing during the fall. High nutrient uptake is due to removal of plant material by grazing or haying.

Table 7-4. Projected Crop Acreage and Reclaimed Water Storage Requirements

	Irrigated crops							
Crop mix	Alfalfa, percent	Pasture, percent	Corn (silage), percent	Orchard (w/cover), percent	1996 Acreage required, acres	1996 Storage required, acre-feet	2020 Acreage required, acres	2020 Storage required, acre-feet
1	100				870	630	1,500	1,060
2		100			1,050	640	1,900	1,100
3	50	50			960	640	1,700	1,080
4	30	55	15		1,030	730	1,800	1,200
5	13	75	10	2	1,050	700	1,900	1,200

Based on assumption of 4.5 mgd and 8.0 mgd in 1996 and 2020, respectively.

The irrigation areas and storage requirements listed in Table 7-4 were developed to provide a planning level assessment of the feasibility for effluent reuse. The next step would be to develop a detailed water balance showing seasonal water usage. This information would then be used to determine specific flows for sizing pumping stations and pipelines. The information in Table 7-4 is sufficiently accurate to develop cost estimates for evaluation and comparison of alternatives.

Acreage

Assuming an average reclaimed water flow of 4.5 mgd, the WRP's total output from May 1 to October 31 would be 2,541 acre-feet. The amount of crop acreage needed to use this volume of water depends on the crop or mix of crops to be irrigated. If grass pasture is the only crop irrigated, 1,050 acres would be required. Irrigation of only alfalfa, which needs more water,

would lower the requirement to 870 acres (Table 7-4). Other crop mixes involving silage corn and orchard (with cover) result in similar acreages. Inclusion of alfalfa in the crop mix reduces the estimated acreage required because of its higher water usage. The orchard acreage in the Grants Pass area is too small to have a significant effect on land requirements.

Acreage requirements will increase with reclaimed water output (Table 7-4). Projected requirements in the year 2020 for cropland will increase to 1,500 (100 percent alfalfa) or 1,900 (100 percent pasture) acres for 6 months of reclaimed water use.

Storage

As discussed above, the irrigation acreage requirement is dictated by the total annual amount of water available for irrigation. During the peak demand months of the irrigation season, the irrigation demand will exceed the effluent flow from the plant. Therefore, effluent storage will be necessary to match periods of high irrigation demand with the more uniform effluent flow. The amount of storage required is equivalent to 42 to 46 days of ADWF, depending on the crop mix and timing of reclaimed water flow. Projections indicate the current need is for 630 to 730 acre-feet of reclaimed water storage. This projection assumes an ADWF of 4.5 mgd from May 1 through October 31. In the year 2020, ADWF is expected to rise to 8.0 mgd with storage requirements at 1,060 to 1,200 acre-feet. This amount of storage equals an 100-acre, 15-feet-deep pond. These figures only account for reclaimed water storage. Rainfall estimates would have to be included in the final storage pond design. The storage lagoon also could act as a surge pond during times of high flows.

The irrigation season generally begins in late April or early May and continues through mid-September. To avoid discharging to the river, plant effluent would be directed to storage from mid-September through the end of October. Plant effluent held in storage during the winter would be available for irrigation during April. Around the beginning of May, plant effluent would be diverted from river discharge to the reuse system.

Reuse System Configuration

The effluent reuse system would incorporate three key elements: a transmission pipeline, storage reservoir, and a distribution system. The transmission system would consist of a pumping station at the wastewater treatment plant and pipeline leading to the storage reservoir. The reuse system would be laid out to serve the largest parcels of land suitable for irrigation. As shown on the previous figures, these areas lie west of the plant along the north side of the Rogue River. To reach sufficient land areas, the reuse system would have to extend beyond the Applegate River and also serve areas south of the Rogue River.

Storage would be provided by a single large reservoir or multiple smaller reservoirs. A single reservoir, requiring 100 acres of land, would be difficult to site. Multiple smaller reservoirs would be easier to site, more suitable for phased construction, may be more centrally located to the irrigation sites, and could make use of natural features to reduce construction costs. Small

reservoirs may also provide amenities for golf courses, parks, and industrial developments. Maintenance would, of course, be more costly for many small reservoirs than for a single large reservoir. Reservoir cost would be greatly affected if lining is required. This and other design questions would be addressed through predesign studies.

Because this report is not intended to be a reservoir siting study, we assumed, for the purposes of developing cost estimates (see Chapter 9), that a single reservoir would be constructed on the north side of the Rogue River near its confluence with the Applegate River.

The distribution system would deliver water from the storage reservoir to the irrigation sites. A low pressure distribution system would be suitable for flood irrigation. Farmers would then be responsible for boosting the pressure for spray irrigation. A high pressure distribution system, suitable for spray irrigation, would be attractive for potential users.

Implementation Issues

There are many implementation and management issues that would need to be addressed before undertaking a reuse program. Reservoir siting, piping route selection, and right-of-way acquisition are some of the more obvious issues. Other considerations include system ownership, financing, system management, and promotion.

System ownership and financing are somewhat related issues. Financing through the Bureau of Reclamation loan program would be an alternative to city financing. With existing irrigation sources becoming more scarce, the Bureau of Reclamation has been looking favorably at effluent reuse projects. Financing through the Bureau of Reclamation loan program could allow an irrigation district to construct and own the distribution system. The irrigation users would be responsible for repaying the loans. The city would only own and operate the transmission and storage systems.

Management of the reuse system is another consideration. A parcel of city-owned agricultural land would add flexibility in system operation. The water application rate on the city-owned parcel could be adjusted to account for excess or shortfall in water availability. In dry years, this would ensure that the irrigation customers have adequate water supplies. In wet years, the city would be assured that all of the plant effluent would be reused.

The above issues illustrate some of the complexities involved in developing an effluent reuse system. Strong leadership, thorough planning, and close management are critical for a successful effluent reuse program.

POTENTIAL USER INTEREST

Large land parcels currently under irrigation are best for establishing a reclaimed water use program. Control of the reclaimed water could be maximized and distribution simplified if a single large block of suitable land was available. Privately-owned land or city-owned land

would provide the most flexible system. Such irrigated parcels are uncommon in the Grants Pass area, but several areas have potential. Figure 7-3 shows the areas with most promise. These are lands zoned for agriculture in the Josephine County Comprehensive Plan. The figure also shows straight-line distances from the treatment plant to various parts of the county. Straight-line distances are shorter than either road travel or pipeline distances. A summary of potential reuse areas is as follows:

1. The most likely areas consist of agricultural properties west of the UGB. Several large parcels are located in this area, including an apple orchard (Wild River Farms) on Upper River Road with approximately 150 acres and the former "mint farm" on Riverbank Road with 400 to 500 irrigated acres. Most other parcels are approximately 5 acres in size, with the exception of several dairies west of Grants Pass with fields ranging from 30 to 60 acres. Several small orchards are south of Grants Pass in the Fruitdale area. Sufficient acreage may be difficult to find without complex distribution to many small farms and fields.

2. The GPID provides water for 7,700 users. Only 12 members have more than 40 acres of land. The large number of users and small parcels in the district make distribution and controlled use difficult under the current ditch delivery system. Furthermore, Level III effluent will not be suitable for many of the current crops grown. Lack of control on the final disposition of the water is another drawback. Unused water is returned to the Rogue River. Furthermore, discharge to the GPID system of canals and ditches is considered by DEQ rules as discharge into waters of the state. Therefore, as with discharge into small streams, beneficial uses would have to be protected.

3. The FVID has water rights from the Rogue River and Vannoy Creek. Water is pumped from these sources on an as-needed basis. The district contains 700 acres of land. The distribution system consists of 7 to 8 miles of ditches, and unused water is returned to the Rogue River and Vannoy Creek. Most of the irrigation is flood irrigation. The feasibility of using this system would be difficult due to the lack of parcels with significant acreage and the potential for reclaimed water discharge to the Rogue River and Vannoy Creek.

4. The Merlin area has the potential for using reclaimed water. Although only small amounts of cropped land exist in the Merlin area, several attempts have been made in the name of the Merlin Irrigation District to acquire water rights to irrigate 9000 acres. A proposed 18-hole golf course near the Josephine County Airport would require 300,000 to 400,000 gallons of water per day in addition to the reclaimed water available from its own wastewater treatment facility. Although the users would be 7 to 10 miles from the WRP, this area could represent an opportunity for beneficial reclaimed water use if sufficient parcels of land can be found. The possibility also exists for irrigation of the Interstate 5 right-of-way as part of a distribution system to the Merlin area.

Two key elements for user acceptance of reclaimed water are the availability of water and easy use. Removal of the Savage Rapids Dam will end local access to flood irrigation; a pumping system from the Rogue River is already planned. At this point in time, irrigators currently practicing flood irrigation may be more willing to accept reclaimed water supplied to fields in a pressurized pipeline. Although some revision in irrigation practices would be necessary, the longer irrigation season and consistent water supply may be attractive.

Treatment to Level IV standards, while more expensive, would significantly improve the ease of reuse. Public health concerns would be reduced, opening more sites to consideration (e.g., public sports fields). These advantages must be weighed against the cost of the higher level treatment.

SUMMARY

To use reclaimed water from the WRP during periods of river discharge restriction, sufficient cropland must be identified and transmission storage, and distribution facilities must be constructed. Irrigation areas would increase from 1,050 acres of pasture needed for 1996 flows to 1,900 acres by the year 2020. Land also will be needed for a storage reservoir. A storage volume of 650 acre-feet would be required to buffer against fluctuating flow rates and crop water use under the year 1996 conditions. Use of local irrigation districts is unlikely without modifications to current ditch distribution systems to prevent return flow to surface water bodies.

Current water quality is good. It would qualify as Level II water under OAR 340-55-015 if disinfection were more consistent. Level II water, however, is restricted by wide buffer zones around fields and limited crops to which it may be applied. Level III water would be the best option for the WRP since buffer zones are reduced or eliminated, and it can be achieved with only minor modifications to the current treatment processes. Level IV water may be cost-effective to produce, depending upon future summer discharge and irrigation restrictions.

The best land for irrigation with reclaimed water lies west of the UGB along the Rogue River and in the Merlin area to the north. Smaller areas of agricultural land lie to the south near Fruitdale and along the Applegate River. Soils in these areas are suitable for irrigation. Large parcels are the most desirable because water distribution is less complex and pipeline construction costs are lower. Large parcels are difficult to find near Grants Pass. West of Grants Pass are several large landholders within 5 to 6 miles of the WRP. In the Merlin area (7 to 10 miles north) is a proposed golf course, but sufficient additional agricultural land may be difficult to locate.

Effluent reuse with poplar trees has recently received much interest throughout Oregon. For example, Woodburn, Oregon, is one community that owns and irrigates a large plantation of poplar trees. Limited markets are one reason why poplar trees have been slow to gain widespread use. Currently, the Fort James paper mill in Wauna is the only mill in Oregon using poplar chips. Hauling distances over 50 miles severely limit any profit margin for private growers. However, because of their high water consumption, poplars are being carefully considered by many municipalities.

Depending on climate and planting practices, poplar trees may use 1- to 2-feet-per-year more irrigation water than forage crops. This translates to 25 to 40 percent less land area required for the reuse system. If poplar trees were used exclusively, the acreage requirement would be on the order of 600 to 750 acres at 1996 effluent flows and 1,100 to 1,400 acres for year 2020 flows.

In summary, the reclaimed water from the WRP currently is suitable for beneficial reuse by irrigation onto cropland. Land lying west, north, and south of the Grants Pass UGB could be used for this purpose. Land to the west is the most desirable since it is closest and contains some of the largest land parcels; however, adequate land in one location may be difficult to locate. The purchase or lease of sufficient land by the City of Grants Pass may need to be considered to ensure maximum control over water use and to simplify distribution.

It is clear that very large land areas would be needed for a reuse program sized to handle all summer wastewater flows. An alternative approach would be for the city to examine targets of opportunity where reuse may be feasible on a smaller scale. Examples could include producing Level IV effluent to irrigate nearby parks, provide supplemental flow into a portion of the GPID, or irrigate a demonstration poplar tree project on the plant site.

The key design data and cost estimate for an effluent reuse system are presented, along with other liquid treatment alternatives, in Chapter 9.

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CHAPTER 8

WASTEWATER COLLECTION SYSTEM

An evaluation of the collection system is necessary to identify sewer and pumping station capacities and limitations and to ascertain wastewater flow contributions associated with infiltration and inflow (I/I). Once deficiencies and capacities have been identified, effective rehabilitation, replacement, and expansion projects can be recommended.

EXISTING COLLECTION SYSTEM

The condition and capacity of the wastewater collection system greatly influences the peak flows experienced at the Grants Pass Water Restoration Plant (WRP). The collection system was most recently evaluated in the May 1983 *Sewage Collection System Master Plan* (James M. Montgomery Consulting Engineers). The following collection system description is based in part on the findings of the May 1983 plan.

History and Description

The existing collection system consists of approximately 110 miles of gravity sewers, one force main, approximately 2,000 feet long, and three pumping stations. A map of the collection system divided into 14 basins is shown in Figure 8-1. (This figure is located at the back of the report.)

Gravity Sewers. While records are not available, it is believed that sewers installed prior to 1927 are vitrified clay pipe with bituminous joints. In 1983, these lines accounted for about 17 percent of the total collection system and serve about 350 acres of the downtown area.

The sewers installed from 1927 to 1964 are mainly unreinforced concrete pipe with bell-and-spigot joints sealed with cement mortar. The cement mortar seal is inflexible and therefore cracks and fails if there is slight differential settlement of the pipe. In addition, the cement mortar deteriorates over time. Failed joints can allow groundwater to infiltrate into the pipes. Many of the sewers installed during this period were manufactured at a plant in Grants Pass. City staff have observed pipe deterioration during repair projects and report that the concrete pipe manufactured in Grants Pass has been more prone to structural failure than pipe obtained from other sources. The concrete lines installed during this period made up about 20 percent of the collection system in 1983.

Since 1964, most of the sewers installed in Grant Pass have been concrete with bell-and-spigot joints and rubber gaskets. Some asbestos cement and polyvinyl chloride (PVC) pipe was also

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used. The bell-and-spigot joints and rubber gaskets are flexible and relatively watertight. This type of construction accounted for more than 50 percent of the system in 1983.

Pumping Stations. The collection system includes three pumping stations, two of which are simply lift stations (Figure 8-1). Design data for all three pump stations are included in Table 8-1. The lift stations, Webster Lift Station 1 and Webster Lift Station 2, are both located on Webster Lane and serve the mobile home park in the western section of Basin A. Each station is equipped with two, vertical, nonclog, centrifugal pumps. The pumps in Station 1 are 7.5 hp pumps each with a capacity of 100 gallons per minute (gpm) at 23 feet total dynamic head (TDH). Station 2 pumps are 3 hp pumps, each with a capacity of 100 gpm at 10 feet TDH.

The third station, designated the Bridge Street Station, is located at the intersection of Bridge Street and Tami Court and discharges into a 4-inch-diameter, 1,973-foot-long PVC force main. The station is equipped with two submersible nonclog centrifugal pumps. Each of the 20 hp pumps has a capacity of 650 gpm at 75 feet TDH. Air injection is also provided for the force main to control sulfides. A 50 kW natural gas fueled engine generator is provided for standby power.

Table 8-1. Pumping Stations Design Data

	Webster Lift Station 1	Webster Lift Station 2	Bridge Street Pump Station
Location	All Sports Park	Webster Lane	Bridge Street and Tami Court
Type	Duplex	Duplex	Duplex
Pump Type	Self priming, vertical close coupled, nonclog, centrifugal	Self priming, vertical close coupled, nonclog, centrifugal	Submersible, nonclog, centrifugal
Motor Horsepower	7.5	3	20
Motor Type	Constant speed	Constant speed	Variable speed
Capacity, each, gpm	100	100	650
Head, ft	23	10	75

Each pumping station was inspected for evidence of sulfide corrosion. All wet wells and receiving manholes were found to be in good condition. Pump hour meters were reviewed to provide an indication of pump usage and remaining capacity.

During the high rainfall period of December 1996, the pumps at the Webster Lift Station 1 ran about 20 percent of the time and those at Webster Lift Station 2 ran about 15 percent of the time. During dry weather, the run times average about 15 to 10 percent for Webster stations 1 and 2,

respectively.

The run time meters at the Bridge Street pumping station are not functioning properly. However, the plant system staff monitored the station operation and determined that the pumps run about 20 percent of the time.

The Webster Lift Station 1 is designed not to overflow. In the event of a power outage, wastewater will flow through the lift station and on to the treatment plant. The Webster Lift Station 2 is only equipped to be served by a trailer-mounted standby generator. There is no overflow point so wastewater would back up within the area served. However, city staff believes that this station may also flow through at high flows. The Bridge Street pumping station is equipped with a natural gas fueled standby generator.

Maintenance Program. The maintenance program is staffed by two full-time people. In addition, the collection system supervisor, works half-time on the wastewater collection system and half-time on the water distribution system. Maintenance work consists of TV-inspection, emergency repair work, a line cleaning program, and a manhole solid lid replacement program.

The city budgets for 5 percent of the collection system to be TV-inspected each year. This includes inspection of all new collection system construction and a 1-year follow-up inspection. To date, the majority of lines 8 inches or larger have been inspected.

One quarter of the collection system is cleaned by city staff each year. In addition, select pipe reaches are part of a 6-month cleaning program due to chronic problems with grease and debris buildup.

To reduce inflow, the city began replacing manhole lids 3 years ago to eliminate those with holes. The program involves replacement of 200 lids each year and is to continue until complete replacement has been achieved.

Deficiencies

Wastewater collection systems can have several types of deficiencies:

- Capacity Limitations
- Structural failures
- Excessive I/I

Capacity Limitations. The best indication of inadequate capacity is sanitary sewer overflow (SSO) during peak storm events. During the storms experienced in December 1996 and January 1997, city collection system staff recorded all sewage overflows at manholes and estimated the overflow rate (Table 8-2). The locations of the problem manholes are noted on Figure 8-1. It should be noted that the period of December 1996 to January 1997 was marked with unseasonably high precipitation and substantial localized flooding throughout the Grants Pass area.

As can be seen in Table 8-2 and Figure 8-1, significant capacity limitations exist in Basins F and H. In order to help alleviate this problem and minimize overflows at manholes F69 and F70, collection system staff constructed a relief line that allows excess flow in Basin F to overflow into Basin E. The relief line is a 6-inch-diameter pipe that connects a cleanout in Basin F to a cleanout in Basin E. In March 1988, a 10-inch-diameter sewer was constructed to connect Basin F to Basin E.

Table 8-2. Summary of Wastewater Overflows

Manhole number	Overflow start date	Overflow start time	Overflow stop date	Overflow stop time	Estimated flow rate, gpm
F150	12/7/96	12:00	12/9/96	12:00	18
	12/30/96	19:00	12/31/96	6:00	50
	12/31/96	20:30	1/2/97	10:00	50
F94	12/7/96	12:00	12/9/96	12:00	15
	12/30/96	19:00	12/31/96	6:00	50
	12/31/96	20:30	1/2/97	16:00	50
F70	12/7/96	12:00	12/10/96	17:00	35
	12/29/96	10:00	1/4/97	18:00	35
F69	12/31/96	20:30	1/2/97	10:00	50
H139	12/7/96	12:00	12/9/96	12:00	8
H11	12/30/96	19:00	12/31/96	6:00	50
H6	1/1/97	9:00	1/2/97	10:00	50
H5	1/1/97	9:00	1/2/97	10:00	50
B52	12/7/96	12:00	12/9/96	12:00	5
N3	1/1/97	15:00	1/2/97	10:00	50

The 1983 plan listed a number of capacity deficiencies, determined by modeling the system using flow conditions existing in 1983. Peak wet weather flow in 1983 was estimated at 26 million gallons per day (mgd). These deficiencies are listed in Table 8-3 and are shown in Figure 8-1. Many of the overflow locations noted during December 1996 and January 1997 coincide with the undersized lines identified in the 1983 plan.

Table 8-3. System Capacity Deficiencies Identified in the 1983 Plan

Interceptor	Reach manholes	Length	Existing pipe size, inches
Pine	F91-F68	2,400	8-10-12
	F68-C23	1,450	12
	C23-C6	2,550	12
	C2-D22	350	12
	G32-C26	1,000	12
North	G1-D38	650	21
Greenwood	B20-D4	2,450	10-12
Seventh	H1-H7	1,100	12
	G9-H1	2,750	12-18
Mill	J41-I19-I4	7,900	8-12-15
	I4-D38	1,100	15

SSOs are receiving heightened attention in recent years with efforts by the Environmental Protection Agency (EPA) to develop national overflow standards. Current Department of Environmental Quality (DEQ) regulations, Oregon Administrative Rules (OAR) 340-41 (13-14), prohibit wet season overflows except during the 24-hour, 1-in-5-year storm event. Dry season overflows are prohibited except during the 24-hour, 1-in-10-year storm. Collection systems that have historically experienced SSO problems because of high I/I may be allowed until January 1, 2010, to comply with the regulation. However, DEQ must have accepted an interim plan outlining steps being taken to achieve compliance.

In addition to evaluating overflows, the capacity of the interceptors near the WRP site were calculated. Their capacities are summarized in Table 8-4. The interceptors near the plant site appear to have adequate capacity for current peak flows.

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Table 8-4. Interceptor Capacity Summary[a]

Upstream manhole	Downstream manhole	Pipe length, feet	Pipe diameter, inches	Slope, percent	Capacity,[b] mgd
D2	D1	150	30	2.99	45
D3	D2	80	24	0.40	9
D4	D3	140	24	0.39	9
D16	D2	280	30	0.40	17
D17	D16	249	30	0.40	17
A3	A1	500	10	0.28	0.75
A4	A3	560	10	0.28	0.75
M1	D40	560	30	0.27	14
M2	M1	50	30	0.34	15
N1	M2	340	18	0.38	4.2
N2	N1	260	18	0.12	2.4
N3	N2	293	18	0.12	2.4
N4	N3	251	18	0.14	2.5

[a] Capacity based on sewer invert elevations, lengths, and sizes reported in the May 1983 *Sewage Collection System Master Plan.*

[b] Assumes no pipeline surcharging except at D2 to D1.

Structural Failures. Sewer line defects such as holes, cracks, and corrosion can threaten the structural integrity of the pipe. In extreme cases, structural failure can result in the formation of sinkholes in streets and alleys. Grants Pass has experienced pipe failure. City staff report that 17 sinkholes due to sanitary sewer pipeline failures have been repaired since 1992. Figure 8-1 shows the location of the repaired sinkholes.

Defects that threaten the structural integrity of a sewer line are most commonly detected during TV-inspections. Grants Pass has a TV-inspection program that has identified a number of problem areas. Structural defects that have been noted include broken and missing pipe segments and heavy root intrusion.

Excessive I/I. I/I is extraneous wastewater flow in the collection system apart from that associated with residential, commercial, or industrial generation. Infiltration is defined as groundwater that enters the collection system from the surrounding soil via defects in the sewer pipe. Inflow is storm water that enters directly into the system through storm drain cross connections, holes in manhole covers, roof drains, and other direct surface to sanitary sewer connections.

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Problems associated with I/I include the use of collection system and treatment plant capacity that could otherwise be devoted to municipal and industrial growth. During extreme precipitation events, high levels of I/I can result in the occurrence of SSOs with adverse environmental and human health consequences. Finally, I/I results in increased wastewater treatment and collection costs due to higher capacity and energy demands.

The first step in evaluating I/I is to compare it with excessive criteria. I/I is defined as excessive when it is more expensive to convey and treat than it is to eliminate. In federally funded projects, the EPA has established guidelines for per capita wastewater flow rates above which the cost-effectiveness for I/I removal should be evaluated. The guideline for infiltration is 120 gallons per capita per day (gpcd) and is based on the highest 7-day-average flow during periods of seasonally high groundwater and no precipitation. The EPA guideline for inflow is 275 gcpd, and is based on peak flow rates during periods of seasonally high groundwater with associated precipitation. The EPA requires further I/I analysis if its I/I standards are exceeded. The study must quantify the amount of I/I in the collection system, identify corrective measures, and determine the cost-effectiveness of I/I removal over the cost of transporting and treating it.

The high peak flows experienced at the Grants Pass wastewater treatment plant indicate that the collection system has a significant I/I problem. Potentially excessive infiltration, flows greater than 120 gpcd, is indicated based on plant flow data for several periods in 1996 (Table 8-5).

Table 8-5. Analysis of Potentially Excessive Infiltration

Time period with no rain	Highest 7-day-average flow, mgd	Unit flow,[a] gpcd
Mar 12 - Mar 21, 1996	5.40	216
Apr 2 - Apr 8, 1996	5.46	218
Apr 25 - May 11, 1996	4.82	193
May 23 - May 31, 1996	4.64	186

[a] Based on current service area population of 25,000.

The collection system may also experience excessive inflow according to the EPA's definition. Based on a current service area population of 25,000 and a 1-in-5-year peak day treatment plant flow of 20 mgd, the per capita flow rate is 810 gallons per day. This is well above the EPA guideline of 275 gpcd.

In order to compare the infiltration rates seen in Grants Pass with those of other communities within Oregon, Table 8-6 contains unit dry weather flows for various communities. As can be seen, wastewater flows on a per capita basis is greater for Grants Pass than for any of the other towns listed. This is further evidence of high I/I rates in the Grants Pass collection system.

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Table 8-6. Per Capita Wastewater Flows for Selected Communities

City	Population served	Average dry weather flow, mgd	Unit dry weather flow, gpcd
Grants Pass	25,000	4.5	180
Medford[a]	77,475	13.2	170
Klamath Falls[b]	17,700	2.9	164
Gresham[c]	81,315	8.86	109
Brookings[d]	9,424	1.0	106
Florence[e]	6,401	0.68	106
Ashland[f]	17,725	1.72	97

[a] *Facilities Plan for the Water Quality Control Plant*, 1992.
[b] *Spring Street Sewage Treatment Plant Wastewater Facilities Plan*, 1993.
[c] *Gresham Wastewater Treatment Plant Facilities Plan*, 1996.
[d] *City of Brookings Wastewater Treatment Plant Facilities Plan Amendment-Draft*, 1997.
[e] *City of Florence Wastewater Facilities Plan—Draft*, 1997.
[f] *City of Ashland Wastewater Facilities Plan*, 1995.

Infiltration sources are most commonly identified through flow monitoring and TV-inspection of the collection system. During preparation of the 1983 plan, areas of high I/I were determined through flow monitoring of all collection system basins. In addition, the City of Grants Pass has TV-inspected multiple sewer reaches to assess the condition of the system. The most significant problems are summarized in Table 8-7.

Table 8-7. System Deficiencies Detected During TV-Inspection

Basin	Description
A2	Five severe leaks in new pipeline on Balsam Street[a]
B3	High infiltration in pipe on Western St. between Jordan and Bridge
C1	70 percent clay pipe. Root intrusion.
C2	90 percent clay pipe. Root intrusion. Part of pipeline missing.
H2	90 percent clay pipes. Root intrusion.
H3	Concrete pipes with mortar joints have high infiltration.
H4	66 percent clay pipes. Root intrusion.
H7	Concrete pipes with mortar joints have high infiltration.
J12	Pipe leaks on N Street

[a] Corrected in 1997.

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Inflow sources are typically discovered through smoke testing. Smoke testing involves blowing artificial smoke into the collection system with fans. While smoke is being injected, the area being tested is observed and locations where smoke is escaping from the collection system are noted. The identified locations are direct connections to the sanitary sewer that can transmit stormwater to the collection system. Grants Pass smoke tested its collection system in 1972 and again in 1995. Selected results of the 1995 test are summarized in Table 8-8 The city is currently working to correct these deficiencies.

COLLECTION SYSTEM I/I ANALYSIS

Peak per capita wastewater flow in Grants Pass is 810 gpcd based on a peak day flow of 20 mgd and a current service population of 25,000. This flow rate exceeds 275 gpcd, which is the level defined by EPA below which I/I is considered nonexcessive without a cost-effectiveness analysis of I/I removal. Because the Grants Pass per capita flow rates exceed this level, additional analysis of the cost-effectiveness of I/I removal was warranted.

Updated I/I Projections

The first step in the I/I analysis was to re-examine the 1983 *Sewage Collection System Master Plan* that contained an analysis of collection system I/I. The 1983 plan assumed 18 mgd of I/I was distributed throughout the collection system as determined through a series of instantaneous flow measurements at key manholes within the collection system. Table 8-9 shows a summary of the I/I distribution determined in 1983. As indicated in the table, current I/I levels were estimated and assigned to the various collection basins according to the same percent levels as contained in the 1983 plan. We essentially assumed that the highest I/I contributing basins in 1983 are still the highest contributors under the current flow conditions. This is a valid assumption as extensive rehabilitation work has not been done in the top identified basins.

Current I/I within the Grants Pass collection system is estimated to be 22 mgd. This was determined by subtracting the peak base wastewater flow rate from the current peak wet weather flow (PWWF) for the treatment plant. The peak base flow was determined by assuming a per capita flow of 120 gallons per day (gpd), a service population of 25,000, and a peaking factor of 2. PWWF for the plant flow is estimated at 28 mgd.

Table 8-9 contains a ranking of the collection system basins according to gross I/I contribution. However, when determining the cost-effectiveness of I/I removal, a more appropriate method of comparing the basins is based on the amount of I/I contributed per linear foot of sewer line. Table 8-10 contains a list of the basins ranked according to the amount of I/I contributed per linear foot of sewer line.

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Table 8-8. 1995 Smoke Testing Observations

Location	Description
504 NW Berry Lane	Roof drain connected to sewer.
1333 NE Ninth Street	Roof drain connected to sewer.
1061 Lee Roza Lane	Roof drain connected to sewer.
1095 Rogue River Highway	Roof drain connected to sewer.
917 SW J Street	Roof drain connected to sewer.
407 SE Eighth Street	Roof drain connected to sewer.
NW Savage Street at Gilbert Creek	Smoke detected on bridge.
101 NW Wrightwood Circle	Area drain connected to sewer.
402 NW Pleasant View Street	Smoke detected in ditch.
1013 NW Prospect Street	Smoke detected on ground under bridge.
SE Mill Street	Hole in sewer line.
722 NW 5th Street	Smoke detected in back yard.
1403 N.W. Lawnridge	Smoke at tree stump and crack in street
955-957-959 N.E. Ninth Street	Smoke in driveway out of sidewalk area
2175 N.W. Vine Street	Smoke from around cleanout and lean-to in back of house
702 N.E. A Street	Hole under pallet Ninth Street side of house
1774 N.E. Wharton Drive	Smoke from under back patio area
1019 N.E. Churchill	Smoke from under house pipe exposed
1029 Madrone	Smoke from under metal plate in driveway
459 N.E. Flint	Smoke out of heat pump
832 N.E. D Street	Smoke out of crack in driveway
1113 N.W. Bellevue Place	House removed, lateral not plugged
65 S.W. Western	Smoke from under sidewalk by front porch
32 S.W. Western	Hole in line under low growing pines in front yard
1601 S.W. Heather	Hole in lateral in front yard
2072 Rogue River Hwy.	Smoke from around foundation
732 S.E. J Street	Smoke from small building and cap missing from cleanout
275 Fruitdale	Old foundation, all plumbing pipes need caps
301 Canal Lane	Smoke coming from under house
205 S.W. Booth Street	Smoke from around power pole in alley
S.E. Tenth and I Street	Vacant lot. Broken lateral line.
S.E. Eighth Street	Vacant lot between J and K Street. Service not plugged.
976 S.E. N Street	Cap missing on cleanout*
Vacant lot next to 1340 N.W. Prospect	Broken cleanout in field
925 S.E. M Street	Cap missing on cleanout*
1075 S.E. Clarey Street	Need new cap on cleanout*
1635 S.E. Rogers Court	Need cap on cleanout in front yard*
1604 N.W. Crescent Drive	Broken cleanout cap*
424 N.W. Lawrence	Broken cap on cleanout*
1661 N.E. Terrace	Cap missing from cleanout*
755 N.E. Twelfth Street	Broken cleanout cap*
1045 S.W. K Street	Broken cleanout cap*

* Corrected in 1997-1998.

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Table 8-9. Estimated Collection System I/I Distribution

Basin	I/I 1983 (mgd)	Percent distribution	I/I 1997 (mgd)
G	5.7	32	7.0
F	2.2	12	2.7
J	1.9	11	2.4
C	1.8	10	2.2
H	1.1	6	1.4
K	1.0	5	1.2
B	0.8	4	1.0
I	0.8	4	1.0
M	0.7	4	0.8
L	0.5	3	0.7
E	0.5	3	0.6
N	0.5	3	0.6
D	0.4	2	0.5
A	0.2	1	0.2
Total	18.0		22.0

The majority of collection system I/I typically originates in faulty service laterals (WEF, 1994, Collection System Master Plan, 1983; Metcalf and Eddy, 1981). As a result, 20 percent of the estimated I/I was assumed to originate within the sewer main with the remaining 80 percent assumed to originate within the service laterals.

Potential I/I reduction was determined for two cases. The first case was for the replacement of the sewer main and all service laterals from the main to 5 feet inside the property line. This case was chosen because the current 6th and 7th Street sewer improvement project involved replacement of the main and all service laterals between the main and 5 feet inside the property line. Of the I/I originating within the sewer main, it was assumed that this method of rehabilitation would reduce the I/I by 50 percent. This results in only a 10 percent net reduction in I/I for a given length of sewer main. Small reductions in I/I are reasonable for this type of rehabilitation work because as leaks within the sewer main and service lateral connections are repaired groundwater levels tend to rise. Rising groundwater translates to greater infiltration rates through defects in the unrepaired sections of the service laterals. The city reports that groundwater levels rose by more than 4 feet following replacement of the sewer lines within the 6th and 7th Street project.

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Table 8-10. Current Estimated I/I and Associated Removal Costs

Basin	I/I (gpd/ft)	Sewer length (ft)	Number of laterals[a]	Number of manholes[b]	Potential I/I reduction[b]		Cost[g]	
					Mains[c] (mgd)	Mains and laterals[d] (mgd)	Mains[e] ($1,000)	Mains and laterals[f] ($1,000)
G	120	63,450	1,060	210	0.70	5.62	12,660	17,100
C	80	29,950	500	100	0.22	1.78	5,980	8,070
J	50	53,500	890	180	0.24	1.89	10,670	14,420
D	40	13,350	220	40	0.05	0.43	2,600	3,600
F	40	68,800	1,150	230	0.27	2.18	13,730	18,540
A	40	5,200	90	20	0.02	0.16	1,040	1,400
H	40	36,200	600	120	0.14	1.10	7,220	9,760
E	40	16,250	270	50	0.06	0.48	3,240	4,380
B	40	28,100	470	90	0.10	0.79	5,610	7,570
I	40	27,800	460	90	0.10	0.77	5,550	7,490
K	30	49,450	820	160	0.12	0.96	9,860	13,330
L	20	28,100	470	90	0.07	0.54	5,610	7,570
M	20	45,850	760	150	0.08	0.66	9,150	12,360
N	10	46,400	770	160	0.06	0.48	9,260	12,500
Total		512,400	8,540	1,710	2.23	17.83	102,220	138,090

Note: a Assumes one service lateral per 60 ft of sewer main and one manhole per 300 ft of sewer main.
b 20 percent of I/I is assumed to originate within the sewer main with the remaining 80 percent originating within the sewer laterals.
c Replacement of the sewer main and service laterals to within 5 ft of the property line assumed to eliminate 20 percent of the I/I originating in the sewer main.
d Replacement of the sewer main and entire service laterals including that between the property line and the structure served is assumed to eliminate 80 percent of the I/I originating in both the main and the service laterals.
e Cost includes replacement of sewer main, service laterals to within 5 ft of the property line, and 50 percent of all manholes encountered.
f Cost includes replacement of sewer main, entire service laterals, and 50 percent of all manholes encountered.
g Estimated costs:
Sewer main replacement: $80/ft
Lateral replacement: $70/ft - main to property line
$60/ft - property line to structure served
Manhole replacement: $2,500 each

The second repair method examined was replacement of the sewer main and the entire length of all service laterals encountered, including lengths falling inside private property lines. Replacement of the entire length of the service laterals will eliminate defects responsible for rainfall induced infiltration. It will also reduce or eliminate the potential for infiltration to shift

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from the sewer main into the shallow service laterals because of rising groundwater. As a result, it was assumed that replacement of both the sewer main and entire lengths of service laterals would result in a net reduction in I/I by 80 percent in the areas being rehabilitated. This very high I/I removal was selected as a conservative value to present service lateral rehabilitation in the most favorable light possible. This represents service lateral rehabilitation in the most economical and successful approach possible.

Structural Repair Projects

As will be shown in the cost-effectiveness analysis contained in Chapter 9, collection system rehabilitation is not cost-effective based solely on considerations of I/I reduction. As is commonly the case, it was determined that it is cheaper to convey and treat I/I than it is to remove it through rehabilitation of the collection system. However, the collection system has a finite life span and requires planned replacement to eliminate danger of structural failures. Because of their age and generally poor condition, replacement efforts should begin in areas of the collection system containing clay sewer lines. Table 8-11 contains a list of such projects, their associated costs, and potential I/I reduction.

Table 8-11. Estimated Costs and I/I Reduction for Structural Repairs

Project	Basin	Sewer length (ft)	Number of laterals	Number of manholes	Potential I/I reduction mains (gpd)	Cost[d,e] mains, (dollars)	Unit I/I Removal Cost ($/gpd)
Pine Street[a]	C	4,500	75	15	33,400	929,200	28
2nd Street[b]	F	3,000	50	10	11,900	619,500	52
Western Avenue[c]	B	2,500	40	9	10,000	507,000	51
Other clay	C,D,E,F,G,H	90,450	1,508	302	583,300	17,885,000	31

Note:
[a] Pine Street interceptor is considered to be from manhole D44 to C26 and from C26 to F-1.
[b] 2nd Street interceptor is considered to be from manhole from F80 to F105.
[c] Western Avenue interceptor is considered from manhole B32 to B60.
[d] Includes replacement of service laterals from main to 5 ft inside the property line.
[e] Estimated costs:

Sewer main replacement:	$80/ft
Lateral replacement:	$70/ft - main to property line
	$60/ft - property line to structure served
Manhole replacement:	$2,500 each

The Pine Street and 2nd Street projects are recommended based on the large number of structural failures reported by the city in recent years. These structural failures have resulted in numerous sink holes within Pine Street that are a liability concern for the city. In addition to structural concerns, these lines were identified as having capacity deficiencies in the 1983

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Sewage Collection System Master Plan. Subsequent to the preparation of the 1983 plan, there have been several SSOs within this section of the collection system. Collection system modifications constructed in 1997 appear to have corrected the SSO in this area.

The Western Avenue project is an area where severely deteriorated concrete pipe was observed. TV inspection identified high infiltration and severe root intrusion. This line also feeds into a capacity limited segment identified in the 1983 Master Plan.

The potential I/I reduction shown in Table 8-11 is based on replacement of only the sewer mains and the sections of the service laterals from the main to 5 feet inside the private property line. As was previously discussed, replacement of these lines is not cost-effective on the basis of I/I removal but instead are necessitated due to the structural condition of the pipe and potential capacity deficiencies within the sections recommended for replacement.

In sections already requiring replacement because of structural or capacity concerns, replacing entire service laterals instead of only a portion of the laterals becomes cost-effective in terms of I/I removal. Replacing private service laterals in their entirety can result in a reduction in I/I by as much as 80 percent compared to less than 10 percent for repair of the main and service connections alone. Replacing service laterals in conjunction with a planned structural repair project is potentially cost-effective as a method of I/I reduction. Table 8-12 shows estimated costs and potential I/I reduction associated with lateral repair for the structural projects identified in Table 8-11. Comparison of the unit I/I removal costs in Tables 8-11 and 8-12 illustrates the fact that replacement of laterals provides a significantly more economical means of I/I reduction than does replacement of gravity mains alone. A complete I/I removal cost-effectiveness analysis will be presented in Chapter 9.

Table 8-12. Estimated Costs and I/I Reduction for Lateral Repair

Project	Basin	Sewer length (ft)	Number of laterals	Potential I/I reduction laterals (gpd)	Cost[c,d] laterals (dollars)	Unit I/I removal cost ($/gpd)
Pine Street[a]	C	4,500	75	233,700	315,000	1.3
2nd Street[b]	F	3,000	50	83,200	210,000	2.5
Western Avenue[c]	B	2,500	40	53,800	168,000	3.1
Other clay	C,D,E,F,G,H	90,450	1,508	4,082,900	6,331,500	1.6

Note:
- [a] Pine Street interceptor is considered to be from manhole D44 to C26 and from C26 to F1
- [b] 2nd Street interceptor is considered to be from manhole F80 to F105.
- [c] Western Avenue interceptor is considered to be from manhole B32 to B60.
- [d] Cost is for replacement of service lateral from 5 ft inside the property line to the structure served.
- [e] Estimated costs: Lateral replacement: $60/ft - property line to structure served

COLLECTION SYSTEM MASTER PLAN

The large number of sewer line failures and overflows experienced in recent years illustrates the need for the development and implementation of an on-going sanitary sewer improvement program. The 1983 plan contained recommendations for such a program. However, changes have occurred within the collection system that warrant updating of the master plan.

1983 Sewage Collection System Master Plan

The 1983 *Sewage Collection System Master Plan* presented a staged collection system improvement program. The report presented system capacity deficiencies identified by actual overflow events and through modeling the collection system. Subsequent to 1983, the city has acted on several of the report's recommendations. For example, the system has been completely smoke-tested and a 6th and 7th Street interceptor project was completed in 1998.

Master Plan Recommendations. Recommendations included in the 1983 plan were divided into three phases based on their priority. Phase I, to be completed by 1987, was to address existing 1982 capacity deficiencies identified by overflows and system modeling. Phase II, 1987 to 1992, was to correct deficiencies projected for the 5-year interval following 1987. Finally, Phase III recommendations were to address projected capacity deficiencies at full system development in the year 2002.

The 1983 plan included smoke testing, pipeline replacement, rehabilitation projects to reduce I/I, and installation of parallel pipelines. Table 8-13 presents the phased recommendations.

The 1983 plan recommended 28 individual projects for the maintenance and improvement of the Grants Pass collection system. To date, two of these projects have been done. The first was the system wide smoke testing performed in 1995. Smoke testing identified multiple sources of inflow, many of which have been corrected.

The second project undertaken by the city was the 6th and 7th Street interceptor replacement. This project was a Phase II recommendation which entailed replacement of the line between manholes G26 and G98 on 6th Street. Approximately 4,500 feet of sewer main was installed, replacing the existing lines. The project also included repair of service laterals to reduce system I/I. All active non-PVC laterals were replaced from the main sewer to 5 feet inside the property line. Fifty percent of the estimated cost was paid by the property owner with the city making up the balance of the actual cost. It is estimated that 120 service laterals were uncovered of which 75 were active. Abandoned service laterals were disconnected.

Of special interest during the 6th and 7th Street project has been the discovery of several stormwater catch basins connected to the sanitary sewer. One of the connections received runoff from approximately 1 acre of impervious area. This may have caused an inflow of 261 gpm (0.38 mgd) into the system, assuming a 1-hour storm with a 1-in-5-year reoccurrence interval for Grants Pass. The city corrected this particular connection by disconnecting the stormwater catch basin.

Table 8-13. Staged Improvement Program

Description	Basin
Phase 1	
City-wide smoke test program	All
Smoke test repair program	All
15-inch replacement line—Second Street (System 101)	F
18-inch replacement line—"F" and Booth Streets (System 102)	F/C
18-inch replacement line—Pine Street and Rogue River Drive (System 103)	C
15-inch and 30-inch replacement line—Rogue River Drive and Lee Lane (System 104)	C
30-inch replacement line—South Seventh Street (System 202)	D
12-inch and 15-inch replacement line—Bridge Street to Greenwood Avenue (System 502)	B
15-inch replacement line—"A" Street (System 801)	H
21-inch replacement line—North Seventh Street (System 802)	G
Rehabilitation—Basins J1, J2, J3 (System 1501)	J
Rehabilitation—Basin J4 (System 1502)	J
Rehabilitation—Basins I-1, I-2, I-3 (System 901)	I
Rehabilitation—Basin N-3 (System 402)	N
Plug west outlet of manhole J41	J
Phase 2	
15-inch replacement line—Laurel Street and Rogue River Avenue (System 106)	C
12-inch replacement line—Western Avenue and Bridge Street (System 501)	B
21-inch replacement line—South Seventh Street (System 802)	G
12-inch parallel line—McLearn Drive (System 1501)	J
10-inch parallel line—Webster Lane (System 1602)	A
8-inch parallel line—"B" and "C" Streets (MH F15 - MH F17)	F
8-inch parallel line—Hillside Street (MH F33 - MH F35)	F
10-inch and 15-inch replacement line—Prospect Avenue (MH F91 - MH F105)	F
10-inch and 12-inch replacement line—Sixth Street (MH G26 - MH G98)	G
10-inch and 12-inch replacement line—Hefly Street (MH H7 - MH H49)	H
Rehabilitation Program (Multiple Sub-Basins)	-
Phase 3	
Rehabilitation, Basin N-2 (System 402)	N
10-inch parallel line—Rogue River Drive (System 1501)	J
Rehabilitation Program (Multiple Sub-Basins)	-

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Three Skunk Creek crossings and one irrigation canal crossing were also replaced. In each case, the old 8-inch clay sewer was deteriorated and had settled to form a belly. It has long been speculated that creek crossings could represent a significant inflow source. Prior to the pipe replacement, flow in the old sewer was observed to be about 1/4 full upstream of the creek crossing and about 1/3 full downstream.

Upon completion of the 6th and 7th Street sewer project, it has been noted that the groundwater level has risen an estimated 4 feet. Prior to construction, the groundwater level was at or below the old sewer line. The increased groundwater level may be contributing to greater infiltration in other areas of the system.

Planned Actions. The next project contained in the 1983 plan that the city would like to pursue is replacement of the Pine Street interceptor. This project was included in Phase I of the staged improvement program. In recent years, structural failures in the Pine Street interceptor have resulted in six sinkholes that have required immediate repair. Because of its poor structural condition, the 1983 plan recommended high priority be given to the replacement of this interceptor. The city may also assess the condition of 6-inch local mains and private service laterals that tie into the interceptor to determine the need for additional work within the basin.

The city plans to take additional steps not included in the 1983 plan for the improvement of its collection system. The first of these is the investigation of the 6th Street interceptor in the area north of the current replacement project. Stormwater system maps will be consulted to identify potential cross connections. Potential connections may be dye tested to verify actual cross connection with the sanitary sewer system. TV-inspection will be performed to identify acute problem areas and to determine the overall condition of the interceptor and laterals.

A second step the city wishes to take that was not included in the 1983 plan is to purchase a flow meter and a push-type TV camera to bolster its ability to assess system conditions. The flow meter will be used to identify problem areas that contribute high I/I flows to the system. Purchase of the TV camera will allow inspection of private service laterals. Information gained through inspections will assist the city in determining the overall condition of the collection system and will be used to evaluate the need for lateral replacement.

Updated Collection System Master Plan

Due to the continuing deterioration of the collection system, an updated Master Plan may be warranted. Basic goals of a collection system program are the protection of human health, the protection of the environment, and the protection of the public's investment in the existing collection system. The following project objectives have been developed to ensure that the basic goals of the collection system improvement program are attained.

- Eliminate structural failures
- Eliminate sanitary sewer overflows
- Economical wastewater treatment

- Develop planning tools
- Establish private service lateral policy
- Determine budget requirements

Eliminate Structural Failures. The first objective of a collection system improvement program would be the elimination of structural failures. Sanitary sewer systems are commonly assigned service lives of 50 to 100 years (WEF, 1994). As a system's design life is reached, failures can be expected to occur because of pipe deterioration from corrosion, differential settling, and loading. An improvement program could be developed such that repair of structurally threatened system components is a planned maintenance activity rather than an emergency response to a sinkhole caused by a failed pipe. Sinkholes in city streets also present a public safety and city liability risk.

The 1983 Collection System Plan estimated that 26 percent of the collection system is 50 years old or older, having been installed between 1900 and 1950. Of this amount, 85,200 feet of clay pipe was installed between 1900 and 1930 and has been responsible for the majority of sinkhole formation. Concrete sewers constructed since 1930 have also shown serious deterioration. There is some evidence that the older concrete pipe may have been poor quality and is rapidly deteriorating. Repair effort would focus on the system's oldest and therefore most structurally threatened pipe line.

Following repair of all components currently threatened with structural failure, an on-going maintenance program should be developed. Resources could be allocated for the replacement of approximately 1 percent of the total system each year. This assumes a useful service life of 100 years and will result in planned repair of pipe as it reaches its serviceable life. Thus, Grants Pass should rehabilitate approximately 1.1 miles of its 110 miles of sewer line each year. Repair priorities can be determined from existing system knowledge and a continuing program of system investigation and evaluation.

A continuing program of yearly system repair would protect the public's investment in the existing collection system. Structural failures impact the performance of the system as a whole, not just the individual pipe involved in a failure.

Eliminate Sanitary Sewer Overflows. Sanitary sewer overflows threaten public health and environmental quality because of the potential for exposure to untreated wastewater. To limit this potential, DEQ has mandated that SSOs are not permitted except during extreme precipitation events, OAR 340-41-120 (13-14). Wet season overflows are not permitted except during the 1-in-5-year storm, and dry weather overflows are only allowed during the 1-in-10-year event.

SSO elimination can be achieved in one of two ways. The first is by reducing total peak wastewater flows associated with precipitation events. The second method is system design resulting in increased total capacity.

Peak flows during precipitation events are predominantly due to I/I entering the collection system. The I/I uses up available system capacity resulting in the occurrence of an overflow. 80 percent of the systems total peak flow is from I/I. Thus, actions taken to eliminate or reduce the amount of I/I will result in the decreased likelihood of overflows.

The second method of SSO elimination is through system design. Collection system capacity can simply be designed to allow for high peak flow rates associated with precipitation. Because I/I can never be totally eliminated from collection systems, additional capacity must always be included to accommodate the entry of stormwater and groundwater into the system. However, at some point it becomes more economical to begin reducing I/I levels, because of the costs associated with conveying and treating the excess water.

Economical Wastewater Treatment. The conveyance and treatment of I/I flows results in increased capital spending on additional treatment plant and collection system capacity above base domestic and commercial generation. I/I also results in greater energy costs associated with treatment of high peak flows.

An effective collection system program will weigh the cost-effectiveness of I/I removal with that of continued conveyance and treatment. Only a certain percentage of I/I can be cost-effectively eliminated. A collection system program must evaluate this relationship to ensure economical treatment of wastewater.

Develop Planning Tools. In anticipation of continued municipal growth and development, planning tools are a necessary component of an effective collection system program. A collection system model is one such tool. Models enable the study of collection system impacts resulting from projected growth and associated increases in wastewater generation. Capacity deficiencies can be identified and eliminated. This allows accommodation of future growth while providing protection from sanitary sewer overflows.

Additional benefits of collection system modeling may be development of GIS (Geographic Information System) data. GIS data is useful in coordinating planning efforts for projected municipal growth. Potential applications are infrastructure design, land use planning, and zoning.

The collection system was originally modeled during preparation of the 1983 plan. An updated effort could be made including use of extensive flow monitoring data and current population projections. The 1983 model projected conditions up to the year 2002 and assumed a build-out population of 35,350. Population projections and capacity demands have changed and would be reflected in an updated system model. Current projections for the wastewater treatment system are for the year 2020 with a service population of 44,500. Because recommendations of the 1983 plan were based on lower flows, reassessment of the system is necessary to correctly size lines for future flows.

Establish Private Service Lateral Policy. The majority of collection system I/I can originate in faulty service laterals (WEF, 1994, Collection System Master Plan, 1983; Metcalf and Eddy, 1981). One element of I/I is rainfall- induced infiltration. Rainfall-induced infiltration typically

enters collection systems through service laterals because of their shallow depth and results in immediate system response to precipitation events. This behavior is characteristic of the Grants Pass system. Figure 8-2 shows total system flow during a dry season storm event that occurred in October 1997. The 0.95-inch rainfall event began at 12:00 p.m. and the WRP received a rapid increase in flow within 3 hours, peaking only 6 hours after the rain began.

Assuming that I/I reduction is one objective of the collection system improvement program, steps would be taken to repair leaky service laterals. Issues that could be addressed include the city's legal authority to mandate lateral repair, liability issues associated with work on private property, and assigning payment responsibility for the repairs.

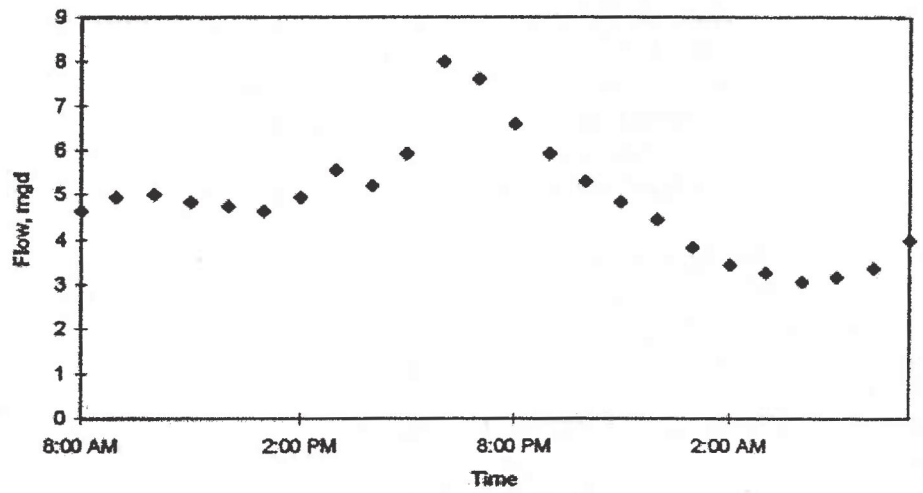
Approaches taken in Oregon to ensure service lateral repair include the recent 6th and 7th Street interceptor project in Grants Pass. In this project, all private service laterals were replaced from the main to a point 5 feet within the property line. The property owner bore 50 percent of the estimated cost, with the remainder being the responsibility of the city. The cities of Springfield and Eugene replace all service laterals from the main connection to the property line bearing the full cost of the work during rehabilitation projects. Salem goes one step further bearing the full cost and responsibility of total lateral replacement during mainline rehabilitation work.

Lateral repair may occur during concerted basin rehabilitation efforts or in a more piecemeal manner. One such approach is to require testing and repair of service laterals prior to property sale. This approach will not result in immediate basin I/I reduction but does provide a valuable long-term tool for the maintenance of collection system service laterals.

Determine Budget Requirements. The final objective of the collection system improvement program would be to determine resource allocation required to maintain a viable wastewater collection system. Collection systems have finite serviceable lives. In light of this, yearly spending is required for the replacement of collection system components. Annual investment in component replacement will result in the elimination of structural failures and the need for unplanned emergency repairs.

Because of the age of the Grants Pass collection system, much of the system has reached its serviceable life. This is evidenced by the structural failures that have occurred in recent years. During the first years of an improvement program, additional resources would be required to catch up with the rate of system deterioration. Following an initial catch up period, an on-going repair program could be initiated and appropriately funded to maintain the serviceability of the system into the future.

Yearly investment in the repair of system mains and interceptors forms the minimum effort required to maintain a functioning collection system. As previously discussed, this will have little net effect on the reduction of system I/I. Additional spending would be required to achieve real reduction in I/I. The collection system improvement program could include budget recommendations for cost-effective removal of I/I from the system.



Note: October 1997 event, prior to stormwater catch basin repair

Figure 8-2. System Response to Rainfall Event

CHAPTER 9

LIQUID STREAM TREATMENT ALTERNATIVES

A Facilities Plan (FP) for Grants Pass WRP completed in January 1999 proposed one alternative for the liquid stream treatment. The upgrades designed were based on a peak flow of 49.8 mgd using a conventional method of treatment. This future flow rate projection was one reason why a Value Engineering (VE) Workshop[1] was held in June 1999. The projections were recalculated and other treatment alternatives were conceptualized. Therefore, two of these alternatives presented in this chapter are from this VE Workshop. The third alternative is the FP concept, but the component construction has been adjusted to treat the peak flow rate of 37.8 mgd. These alternatives were then evaluated in order to select the best method for the liquid stream treatment for the Grants Pass WRP.

UPGRADES COMMON TO THE ALTERNATIVES

All the alternatives have several component upgrades in common. At the headworks, an additional mechanical bar screen would be installed with a 23.5-mgd capacity. This provides redundancy and a maximum capacity of 47 mgd, which is well beyond the projected future flows. An outfall diffuser would be added to improve and reduce ammonia toxicity into the Rogue River. Several miscellaneous plant improvements, which existed in the FP, have also been included in the VE alternatives. These improvements include laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion.

Miscellaneous Plant Improvements. Other miscellaneous plant improvements include:

- **Plant Equipment Audit.** A full analysis of the existing component conditions would be conducted. This audit would analyze the remaining life span of each component and develop an operations and maintenance schedule for the 20-year planning period.

- **Additional Plant Landscaping.** Currently a substantial amount of landscaping at the treatment plant has occurred to promote a good-neighbor environment. However, to continue this effort, additional landscaping would be necessary.

- **Public Involvement/Education Program.** This program would assist in raising the public's knowledge and understanding of the issues that must be considered in decisions regarding the City's sewerage system. Better understanding by the public of the complexity of the issues and the true implications of decisions, plus the variables that must be faced in plant operations would go a long way to minimizing public opposition when future events cause impacts on the plant neighbors and the public in general. This could include open houses, cooperation with schools, and social events to bring the public to a greater sense of understanding and ownership.

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[1] The VE Liquid Stream Workshop is in Appendix H.

ALTERNATIVE ONE – CONVENTIONAL EXPANSION

Alternative One was the proposed alternative from the FP and provides a conventional method of treatment. Figure 9-1 presents the flow schematic for this expansion and Table 9-1 summarizes the existing components and future additions proposed.

The four existing pumps would be replaced with three 19 mgd capacity pumps to meet a firm capacity of 38 mgd. The existing clarifiers and aeration basins would all undergo rehabilitation. One additional rectangular primary clarifier, two aeration basins, and two 115-foot diameter secondary clarifiers would be constructed to meet the projected flow.

SIMILAR UPGRADES TO ALTERNATIVES TWO AND THREE

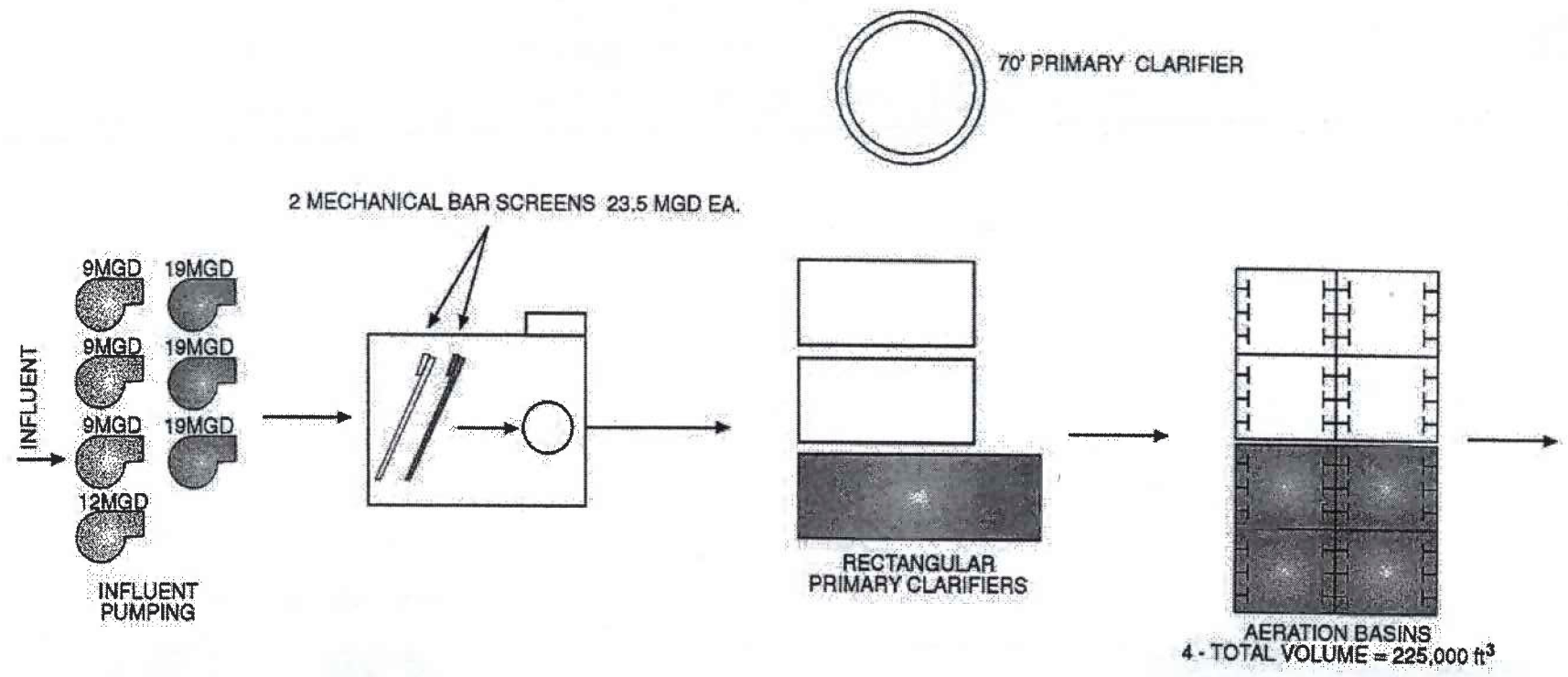
Alternatives Two and Three have several similar upgrades to the Grants Pass WRP (see Table 9-2). At the headworks, it would be necessary to install an additional pump to meet the projected future capacity of 38 mgd. An evaluation would also be necessary to determine if the current influent pumps are in proper working condition. This examination would determine if one additional pump is sufficient, or if all of the pumps need to be replaced. Because the headworks are a primary source for odor, it would be necessary to install odor containment at the influent pump station and the mechanical screening area.

Ballasted Sedimentation. These sedimentation tanks are 30 to 40 times smaller than conventional primary basins and can remove up to 60 percent BOD_5, 90 percent TSS, and 90 percent phosphorus. Ballasted sedimentation is a process where chemical addition (alum or ferric chloride) and microsand are mixed in a flash-mixing tank. Solids and BOD_5 are adsorbed onto the microsand into a heavy floc that is settled in a clarification tank and removed. The clarified water then exits the treated water outlet. The heavy floc is pumped to a hydrocyclone where the sand particles are separated from the floc. The sludge would be pumped to the digester and the microsand recycled back through the process.

Both Alternatives Two and Three present the treatment alternative of Ballasted Sedimentation. The existing gravity thickener would be converted to the ballasted sedimentation basin. This basin would serve as peak overflow treatment for flows greater than 13.5 mgd. If conversion of this basin is unfeasible, a new ballasted sedimentation basin would be constructed on site. The treated effluent would then be recombined with the main treatment train prior to entering the UV-disinfection channel.

Primary Clarification/Gravity Thickener. The existing 70-foot-circular primary clarifier would be converted to a combination primary clarifier/gravity thickener. All of the sludge from the rectangular sedimentation basins would be pumped to this one unit to be thickened before transfer to the digester. This conversion would satisfy the need for primary solids thickening well beyond the 20-year design period.

Ultraviolet Disinfection. Based on the flows projected during the VE study, additional UV capacity is not anticipated during the 20-year planning period.



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Components Being Replaced

Components Being Converted

New Structures

Existing Structures

Figure 9-1
Alternative 1

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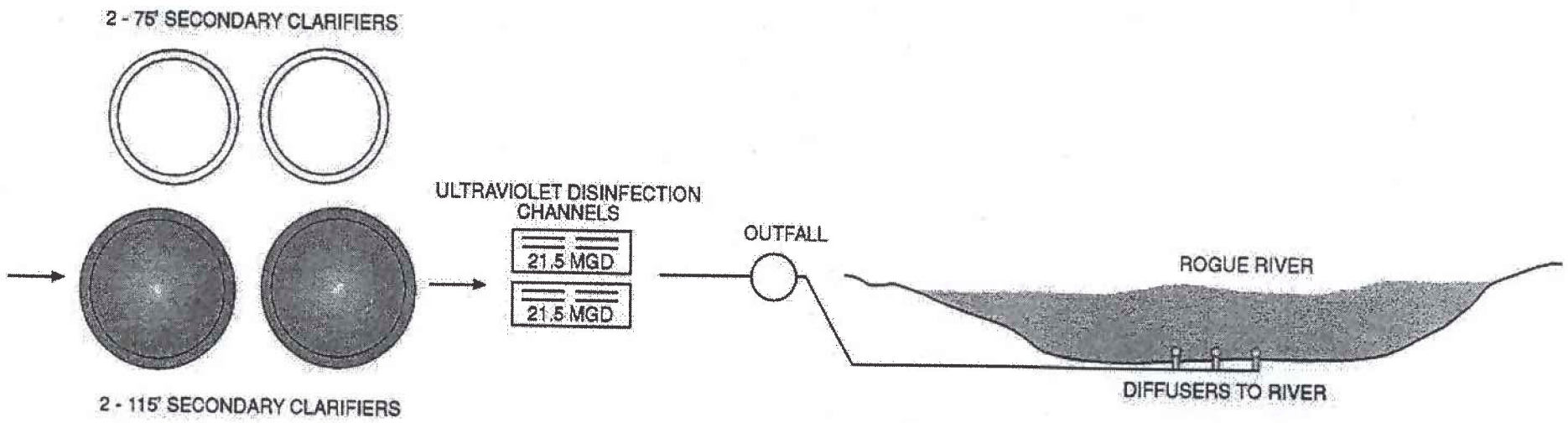

Components Being Replaced
Components Being Converted
New Structures
Existing Structures

Figure 9-1 cont.
Alternative 1

Table 9-1. Proposed Improvements for Alternative One – Conventional Expansion

UNIT PROCESS	YEAR 1999	YEAR 2020
INFLUENT PUMPING		
Type: Variable speed centrifugal		
Number of Pumps	3	-
Capacity each, mgd	9	-
Number of pumps	1	4
Capacity each, mgd	12	18
Total firm capacity, mgd	27	54
MECHANICAL BAR SCREENS		
Number	1	2
Capacity, mgd	23.5	47
PRIMARY SEDIMENTATION BASINS		
Circular		
Number	1	1
Diameter, feet	70	70
Capacity at PWWF, mgd	10.6	10.6
Rectangular		
Number	2	2
Length, feet	108	108
Width, feet	21.5	21.5
Capacity at PWWF, mgd	8.2	8.2
Total primary capacity at PWWF, mgd	27	27
Number		1
Length, feet		150
Width, feet		21.5
Capacity at PWWF, mgd		11.3
Total primary capacity at PWWF, mgd		38.3
GRIT REMOVAL (Degrit Primary Sludge)		
Cyclones, number	2	3
Classifiers, number	1	2
AERATION BASINS		
Number	2	4
Total Volume, 1,000 cf	112.5	225

Table 9-1. Proposed Improvements for Alternative One – Conventional Expansion (cont.)

UNIT PROCESS	YEAR	
	1999	2020
SECONDARY CLARIFICATION		
Number	2	2
Diameter, feet	75	75
Depth, feet	12.3	12.3
Number	-	2
Diameter, feet	-	115
Depth, feet	-	18
UV DISINFECTION		
Type: Medium pressure UV		
Number of channels	2	2
Modules per channel	2	2
Total number of lamps	192	192
PWWF hydraulic capacity, mgd	43	43
MISCELLANEOUS PLANT IMPROVEMENTS		
Laboratory Upgrades		X
Operations building repairs and modifications		X
SCADA System Expansion		X
Yard Piping		X

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Table 9-2. Upgrades Common to Alternative Two – Ballasted Sedimentation and Alternative Three – The Zenon Process

UNIT PROCESS	YEAR 1999	YEAR 2020
INFLUENT PUMPING		
Type: Variable speed centrifugal		
Number of pumps	3	3
Capacity each, mgd	9	13
Number of pumps	1	1
Capacity each, mgd	12	13
Number of pumps	4	
Total firm capacity, mgd	27	39
MECHANICAL BAR SCREENS		
Number	1	2
Capacity, mgd	23.5	47
BALLASTED SEDIMENTATION		
Number		1
Capacity, mgd		24
PRIMARY CLARIFICATION/GRAVITY THICKENER		
Number	1*	1*
Diameter, feet	70	70
Capacity at PWWF, mgd	10.6	10.6
UV DISINFECTION		
Type: Medium pressure UV		
Number of channels	2	2
Modules per channel	2	2
Total number of lamps	192	192
PWWF hydraulic capacity, mgd	43	43
MISCELLANEOUS PLANT IMPROVEMENTS		
Laboratory Upgrades		X
Operations building repairs and modifications		X
SCADA System Expansion		X
Plant Equipment Audit		X
Plant Landscaping		X
Public Education Program		X
Yard Piping		X

* Existing circular primary clarifier converted to combination primary clarifier/gravity thickener.

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ALTERNATIVE TWO – BALLASTED SEDIMENTATION

Alternative Two is the preferred alternative from the VE Workshop. This alternative treats the monthly maximum wet weather flow of 13.5 mgd through the existing primary treatment and conveys the peak flow to ballasted sedimentation. This alternative eliminates the need for additional primary sedimentation basins during this 20-year planning period to meet the future flow demand. After treatment through the ballasted sedimentation tank, the wastewater would be combined with the flow before entering the UV channel for disinfection. Figure 9-2 shows the flow schematic of this alternative. Table 9-3 presents the preliminary sizing of this alternative.

Table 9-3. Proposed Improvements for Alternative Two – Ballasted Sedimentation

	YEAR	
UNIT PROCESS	1999	2020
AERATION BASIN		
Aeration basin fine bubble		X
Aeration basin selector		X
Blowers and DO control		X
Motorized gates		X
SECONDARY CLARIFICATION		
Number	2	2
Diameter, feet	75	75
Depth, feet	12.3	12.3
Number	-	2
Diameter, feet	-	90
Depth, feet	-	14

Aeration Basin. Currently, there are two aeration basins, each segmented into three bays, operating in a plug flow mode. These basins are capable of handling the future flows of 13.5 mgd as long as the secondary clarifier capacity is available. To improve the settling performance in the secondary clarifiers, a bioselector installed in the aeration basin would provide an anoxic treatment zone to control filamentous bacteria. The smallest bay would be segmented into two zones and aerated with coarse bubble diffusers. The rest of the existing aeration basins would be modified by the addition of fine bubble diffusers, dissolved oxygen control, and motorized gates. Other than providing an anoxic zone and fine bubble diffusers, no other aeration basin modifications are needed during the 20-year planning period.

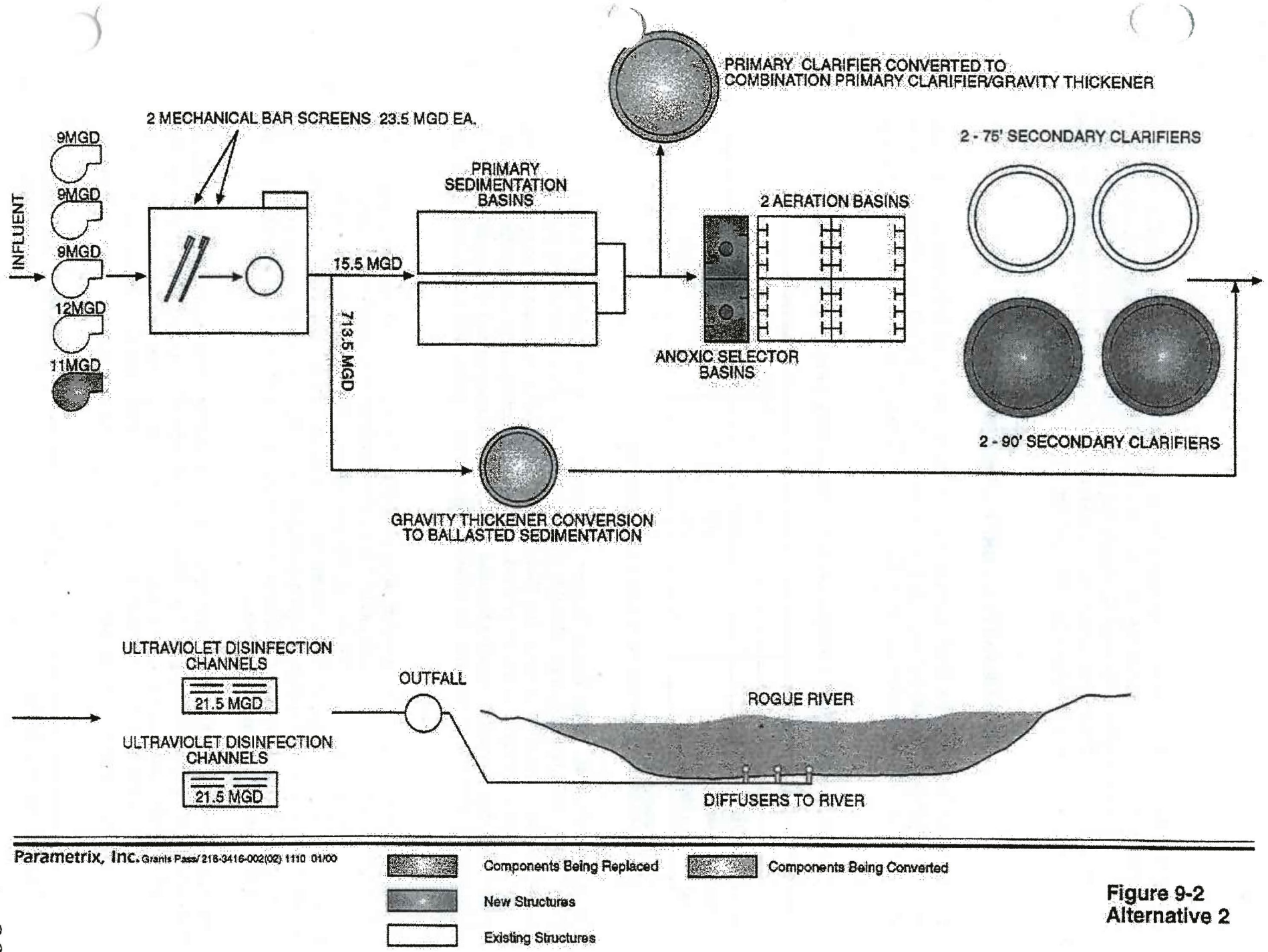

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Figure 9-2
Alternative 2

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Secondary Liquid Treatment. Rehabilitation of the existing secondary clarifiers would be necessary to correct short-circuiting and flow distribution problems. Proposed modifications include weir adjustments, additional skimmers, baffle upgrades, and balancing of flow between clarifiers. To meet future flow demands, two additional 90-foot secondary clarifiers would be constructed, one in year 2010 and the other in year 2020.

ALTERNATIVE THREE – ZENON PROCESS

As in Alternative Two, the third alternative incorporates the use of ballasted sedimentation. However, instead of traditional secondary treatment, the final clarifiers are replaced with the Zenon process. The flow schematic is in Figure 9-3, and Table 9-4 lists the preliminary sizing.

Table 9-4. Proposed Improvements for Alternative Three – Zenon Process

	YEAR	
UNIT PROCESS	1999*	2020**
Design Flow, mgd	13.5	13.5
Total Volume, 1,000 cf	120.5	120.5

* Existing aeration basins.
** The Zenon membranes would be placed in the existing aeration basins.

The Zenon Process. The Zenon Process is a suspended growth biological reactor plus a microfiltration membrane system. For this alternative, the membranes would be placed into the existing aeration basins to serve as the biological reactor. Microorganisms are present which consume organic wastes as a source of food. The waste is converted to carbon dioxide, water, and chemical intermediates and extra biomass. This membrane separation allows elevated levels of biomass to degrade or remove the soluble form of the organic pollutants from the waste stream.

Zenon is a promising process; however, it is still being developed. Some problems that affect the process is that the loading rate (gal/ft^2 * h) of the membrane is unpredictable, which affects the overall performance of the system. Because the loading rate is time-related, it is necessary to adjust the membranes (by addition or removal) by trial-and-error. To prevent trial-and-error design, the process can be designed with a conservative loading rate, which greatly increases the start-up cost. The membranes have also experienced problems with clogging, which can lead to higher operation and maintenance costs.

Despite trying to adjust the loading rate for maximum treatment, the Zenon process has several positive aspects. Because of a high BOD, TSS, and microorganism removal rate, the effluent can be used for water recharge or water reuse. The membrane process performs treatment at a much higher organic loading rate, therefore the size of the tanks can be reduced significantly. Secondary clarification is not necessary because the Zenon process achieves tertiary effluent.

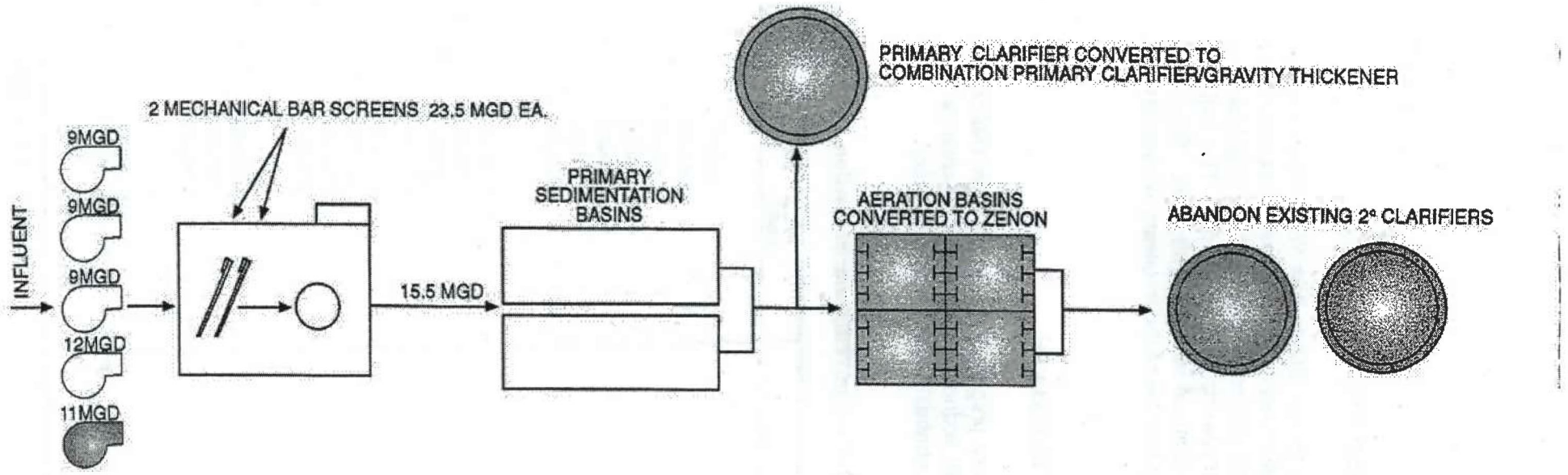

Figure 9-3
Alternative 3

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OPERATIONS AND MAINTENANCE COSTS

The current operations and maintenance (O&M) costs for the Grants Pass WRP would increase with any alternative selection. The conventional expansion and the ballasted sedimentation would have a similar increase but in different measures. The conventional expansion would have an increase in energy usage because of additional components being on line. The ballasted sedimentation would increase O&M costs because of chemical addition to the tank. The Zenon process would have the greatest increase to O&M costs because of the frequency of cleaning and replacing the membranes.

PRELIMINARY COST ESTIMATE

For each alternative, a preliminary cost estimate has been prepared and presented in Tables 9-5, 9-6, and 9-7 for the conventional expansion, ballasted sedimentation, and the Zenon process alternatives, respectively. Costs include engineering, administration, and contingency.

Table 9-5. Capital Costs for Alternative One – Conventional Expansion

ITEM	COST
PRELIMINARY AND PRIMARY TREATMENT	
Influent Pumping	$650,000
Raw Sewage Pipeline	$400,000
Mechanical Bar Screen No. 2	$230,000
New Rectangular Primary Clarifier	$1,080,000
Grit Cyclones	$83,000
Rehabilitate Existing Primary Clarifiers	$90,000
Yard Piping	$270,000
SECONDARY TREATMENT	
Aeration Basin Modifications	$530,000
Aeration Basin Expansion	$1,100,000
Blowers and DO Control	$870,000
New Secondary Clarifiers	$3,300,000
Rehabilitate Existing Clarifiers	$770,000
Flow Distribution Structure	$310,000
Return Sludge Pumping	$340,000
Yard Piping	$770,000
OTHER PLANT IMPROVEMENTS	
Outfall diffuser	$540,000
Lab Improvements	$100,000
Operation Building Repairs and Office	$130,000
SCADA System Expansion	$670,000
TOTAL	$12,233,000

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Table 9-6. Preliminary Costs for Alternative Two – Ballasted Sedimentation

ITEM	COST
PRELIMINARY AND PRIMARY TREATMENT	
Influent Pumping	$560,000
Raw Sewage Pipeline	$240,000
Mechanical Bar Screen No. 2	$230,000
Yard Piping	$270,000
Screening Odor Control	$390,000
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000
Modify Existing Primary to Combination Clarifier/Thickener	$610,000
SECONDARY TREATMENT	
Aeration Basin Modifications	$790,000
Blowers and DO Control	$870,000
New Secondary Clarifiers	$2,400,000
Rehabilitate Existing Clarifiers	$770,000
Yard Piping	$770,000
Motorized Gates	$340,000
OTHER PLANT IMPROVEMENTS	
Outfall Diffuser	$540,000
Lab Improvements	$100,000
Operation Building Repairs and Office	$130,000
SCADA System Expansion	$670,000
Equipment Replacements from Audit Results	$250,000
Plant Landscaping	$100,000
Public Education Program	$50,000
TOTAL	$13,590,000

Table 9-7. Capital Costs for Alternative Three – Zenon Process

ITEM	COST
PRELIMINARY AND PRIMARY TREATMENT	
Influent Pumping	$560,000
Raw Sewage Pipeline	$240,000
Mechanical Bar Screen No. 2	$230,000
Yard Piping	$270,000
Screening Odor Control	$390,000
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000
Modify Existing Primary to Combination Clarifier/Thickener	$610,000
SECONDARY TREATMENT	
Zenon	$13,880,000
Aeration Basin Modifications	$790,000
Blowers and DO Control	$870,000
Yard Piping	$770,000
OTHER PLANT IMPROVEMENTS	
Outfall Diffuser	$540,000
Lab Improvements	$100,000
Operation Building Repairs and Office	$130,000
SCADA System Expansion	$670,000
Equipment Replacement from Audit Results	$250,000
Plant Landscaping	$100,000
Public Education Program	$50,000
TOTAL	$23,960,000

Table 9-8. Capital Costs for Collection System Improvements

Pine Street	$1,050,00
2nd Street	$700,000
Western Avenue	$580,000
Master Plan	$170,000
TOTAL	$2,500,000

PREFERRED ALTERNATIVE

The estimated cost of each of the alternatives, as shown in Tables 9-5 through 9-7, was compared. Table 9-9 summarizes the capital costs of the alternatives. To further this comparison, the advantages and disadvantages of these alternatives were summarized in Table 9-10.

Table 9-9. Comparison of Alternatives

ALTERNATIVE	CAPITAL COST (Millions)
One – Conventional Expansion	$12.23
Two – Ballasted Sedimentation	$13.59
Three – Zenon Process	$23.96

Table 9-10. Summary of Alternatives

ADVANTAGES	DISADVANTAGES
ALTERNATIVE ONE - Conventional Expansion	
Staff familiar with process.	Requires major construction on site.
Meets water quality standards.	Requires greatest use of land.
Uses existing processes.	
Lowest cost alternative.	
ALTERNATIVE TWO - Ballasted Sedimentation	
Moderate cost alternative.	Staff not familiar with the Ballasted Sedimentation Process - training would be required.
Provides maximum utilization of structures.	Moderately high operation and maintenance cost.
Meets water quality standards.	
ALTERNATIVE THREE - Zenon Process	
Produces extremely high effluent quality.	Staff not familiar with the Ballasted Sedimentation Process or Zenon Process - training would be required.
Provides maximum utilization of structures.	Highest cost alternative.
No need for secondary clarifiers.	Highest operation and maintenance cost.
Least amount of construction.	Cost not justified unless water is for reuse.

Selection of Preferred Alternative. Alternative Two is the preferred alternative for the Grants Pass WRP liquid treatment stream. This selection has been based on the following evaluation:

- The difference in capital cost between preferred Alternative 2 – Ballasted Sedimentation and Alternative 1 – Conventional Expansion (lowest cost alternative) is only 10 percent. This is well within the level of accuracy of the overall estimates. Therefore, the cost of both alternatives are essentially the same and the cost differential cannot be used to distinguish between them.

- Alternative 2 – Ballasted Sedimentation has the following major advantages over Alternative 1 – Conventional Expansion:

  - Ballasted Sedimentation requires significantly less site construction than conventional expansion.

  - Ballasted Sedimentation is not subject to the "biological floc wash-out" problem normally associated with peak flow events and the conventional expansion.

  - Ballasted Sedimentation, once constructed, offers an opportunity to use the sedimentation process during non-peak flow events. The sedimentation tanks could be used to polish final secondary effluent using chemically assisted settling (tertiary treatment).

  - Ballasted Sedimentation can be constructed on site with no possible impacts to plant operation since it is a separate process.

  - Ballasted Sedimentation maximizes use of the existing treatment structures at the plant site.

Estimated Performance of Ballasted Sedimentation. When Ballasted Sedimentation is used for split flow treatment, the combined plant effluent must meet secondary treatment criteria for effluent concentration. Mass balance calculations of the combined effluent concentrations are presented in Table 9-11 for the year 2020 design flows. Plant data from 1996-2000 was used to estimate influent wastewater characteristics and the performance of the existing conventional treatment during high flow events.

The calculations summarized in Table 9-11 indicate that the final effluent concentrations during split flow treatment will meet secondary treatment guidelines. The conclusion from this analysis is that the Ballasted Sedimentation process operating in parallel with the upgraded conventional treatment train will consistently meet secondary effluent criteria during storm events.

Table 9-11. Ballasted Sedimentation Mass Balance Calculations for Year 2020

Description	Average	Best	Comments
2020 Peak Wet Weather Flow – mgd	38.0	38.0	From Table 4-9
2020 Peak Day Flow	27.5	27.5	From Table 4-9
2020 Max Month Wet Weather Flow – mgd	13.5	13.5	From Table 4-9
Maximum Flow to Ballasted Sedimentation – mgd	24.5	24.5	Assume all flow above 2020 maximum month wet weather flow of 13.5 mgd to BS
Peak Day Flow to Ballasted Sedimentation – mgd	14.0	14.0	Assume all flow above 2020 maximum month wet weather flow of 13.5 mgd to BS
Plant Influent BOD – mg/l Plant Influent BOD – lb/day	72 16,513	48 11,009	Based on 1996-2000 data
Plant Influent TSS – mg/l Plant Influent TSS – lb/day	94 21,559	49 11,238	Based on 1996-2000 data
Removals in Ballasted Sedimentation			
BOD Removal	67%	73%	Based on Actiflo Results at Tacoma
TSS Removal	85%	94%	Based on Actiflo Results at Tacoma
Ballasted Sedimentation Effluent BOD – mg/l	23.8	13.0	
Ballasted Sedimentation Effluent BOD – lb/day	2,774	1,513	
Ballasted Sedimentation Effluent TSS – mg/l	14.1	2.9	
Ballasted Sedimentation Effluent TSS – lb/day	1,646	343	
Removals in Conventional Treatment			
BOD Removal	81%	88%	Based on 1996-2000 data
TSS Removal	80%	94%	Based on 1996-200 data
Secondary Effluent BOD – mg/l Secondary Effluent BOD – lb/day	13.7 1,540	5.8 649	
Secondary Effluent TSS – mg/l Secondary Effluent TSS – lb/day	18.8 2,117	2.9 331	
Total Effluent BOD – lb/day Total Effluent TSS – lb/day	4,314 3,763	2,162 674	
Total Effluent BOD – mg/l Total Effluent TSS – mg/l	19 16	9 3	
Total Effluent BOD Removal	74%	81%	
Total Effluent TSS Removal	83%	94%	

Notes: Does not account for selector and planned improvements in aeration and clarifiers.

CHAPTER 10

BIOSOLIDS HANDLING AND DISPOSAL ALTERNATIVES

A reliable biosolids management program is a critical element in the operation of a wastewater treatment plant. For the Grants Pass WRP, several alternatives were analyzed during the VE Workshop[1] to determine the best program. Three alternatives were selected from the VE Workshop based on economics and feasibility. The fourth alternative is from the FP.

SLUDGE QUANTITIES AND CHARACTERISTICS

To ensure proper design of solids treatment components, future biosolids quantities were calculated using typical removal efficiencies and yield coefficients based on BOD and TSS. The BOD and TSS are proportional to population increase, therefore representing the increase in biosolids production. The average and maximum month primary and secondary biosolids production is in Table 10-1.

Because of the lack of heavy industry in the Grants Pass WRP service area, the solids have low concentrations of heavy metals. Therefore, if solids are treated to meet Class A criteria, the criteria for "Exceptional Quality" (EQ) could be easily met. An EQ rating would allow the end product of solids processing to be sold to the public without any application requirements.

Table 10-1. Biosolids Production (lbs/day)

YEAR	1999	2010	2020
Average Primary	5,100	6,300	7,600
Max. Month Primary	7,000	8,600	10,400
Average Secondary	3,700	4,500	5,500
Max. Month Secondary	5,700	6,500	7,900
TOTAL AVERAGE	8,800	10,900	13,200
TOTAL MAX MONTH	12,700	15,100	18,300

[1] The Biosolids Handling Workshop Document is in Appendix H.

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CURRENT BIOSOLIDS HANDLING AND DISPOSAL PLAN

Currently, the Grants Pass WRP land applies Class B biosolids to farmland near the plant, weather permitting. When wet weather does occur, the biosolids are dewatered at the plant and hauled to Merlin Landfill.

To produce Class B biosolids, several steps occur. The flow schematic for the current process is in Figure 10-1. The primary clarifier solids are thickened in the gravity thickener and the gravity belt thickener dewaters the secondary clarifier waste activated sludge. The primary and secondary sludge are then combined and sent to the 50-foot-diameter anaerobic digester. The digester has a capacity of 59,000 cubic feet and an average detention time of 28 days. The biosolids are digested to 2 to 4 percent solids and then hauled in two 3,500-gallon tanker trucks to be land applied. A short-term holding tank (referred to as the No. 2 digester) can store biosolids for 4 to 6 days, if necessary. These biosolids from the storage tank are either land-applied or dewatered by a belt filter press to 16 to 18 percent solids. Dewatered biosolids are loaded into a dumpster and hauled to the Merlin Landfill.

ALTERNATIVE ONE - MERLIN LANDFILL CO-COMPOST FACILITY

Alternative One, the preferred alternative from the VE Workshop, would co-compost digested primary and raw secondary biosolids. Figure 10-2 presents the flow diagram of this alternative. This alternative would consist of the existing digester, a new dewatering device (the existing belt filter press would remain on site for standby emergency use), and a co-composting facility. The elimination of the Redwood co-composting facility leaves a regional need for co-compost. Therefore, this alternative would provide the public the product. If the Class A co-compost received an EQ rating, it would have no application restrictions.

Under this solid waste handling system, the existing digesters would be used to treat only primary clarifier solids. The secondary clarifier solids would be combined with the digested solids, dewatered in a new component, and trucked to the new co-composting facility located near the center of the 316-acre Merlin Landfill. The new facility would co-compost green wastes (i.e., yard debris, animal manure, and wood waste) from the general public, with the biosolids from the Grants Pass WRP using the extended aerated state pile method of co-composting.

To determine the dewatering device, the City would pilot-test various devices, such as a centrifuge or another belt filter press. Equipment would be tested for solids capacity and handling, operator ease and equipment complexity, energy usage, polymer usage, and overall performance issues. The device must produce at least 15 percent solids or greater. Based on the results of the tests, the City would choose the most feasible equipment for the Grants Pass WRP.

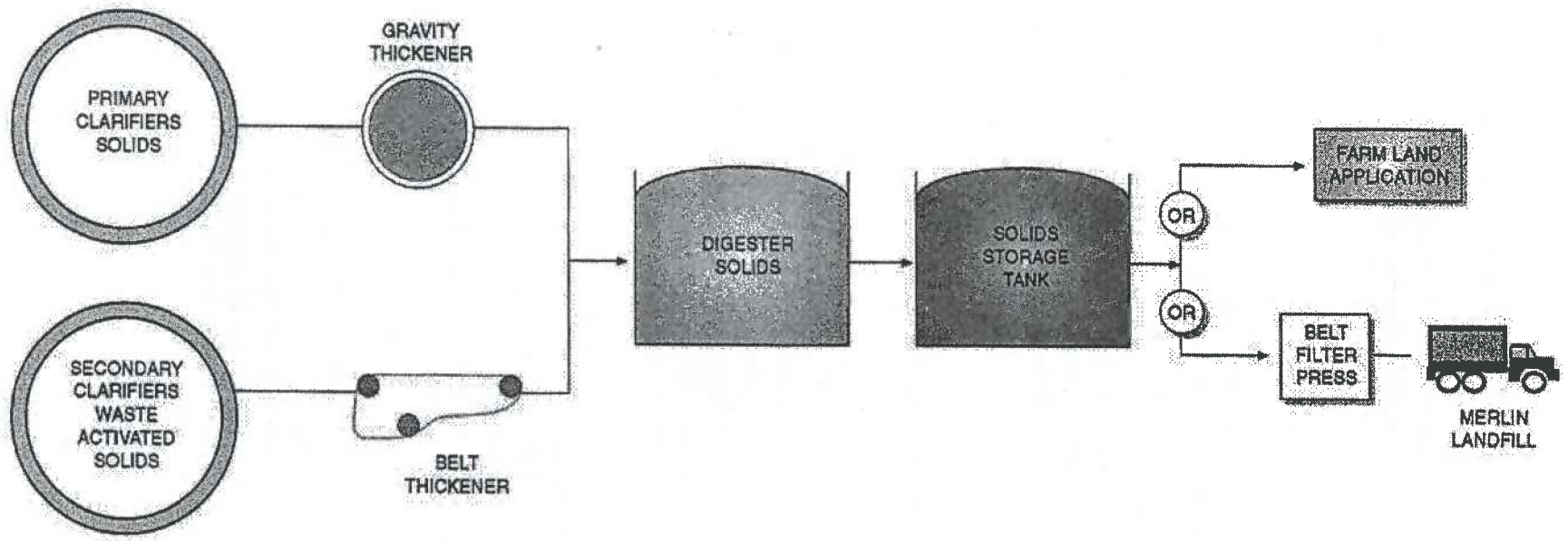

Figure 10-1
Current Sludge Program

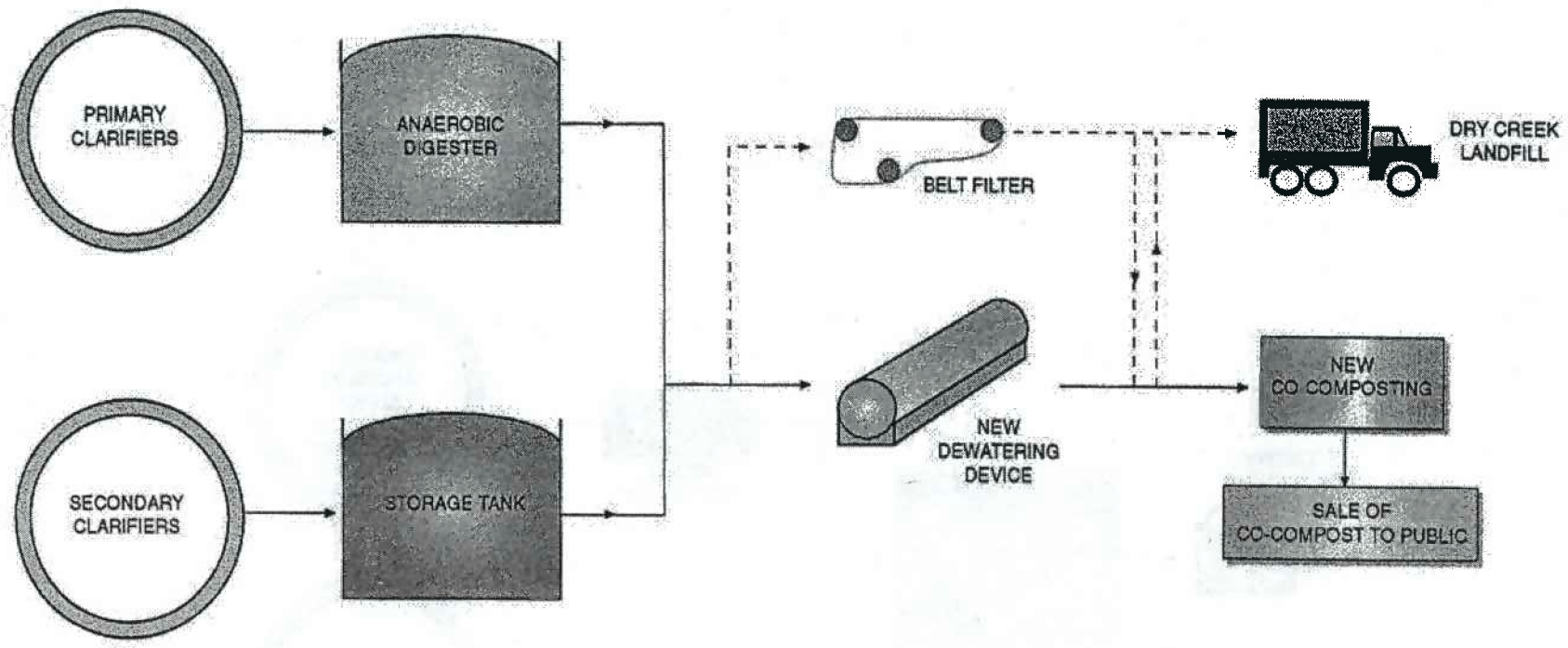

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Figure 10-2
Alternative 1

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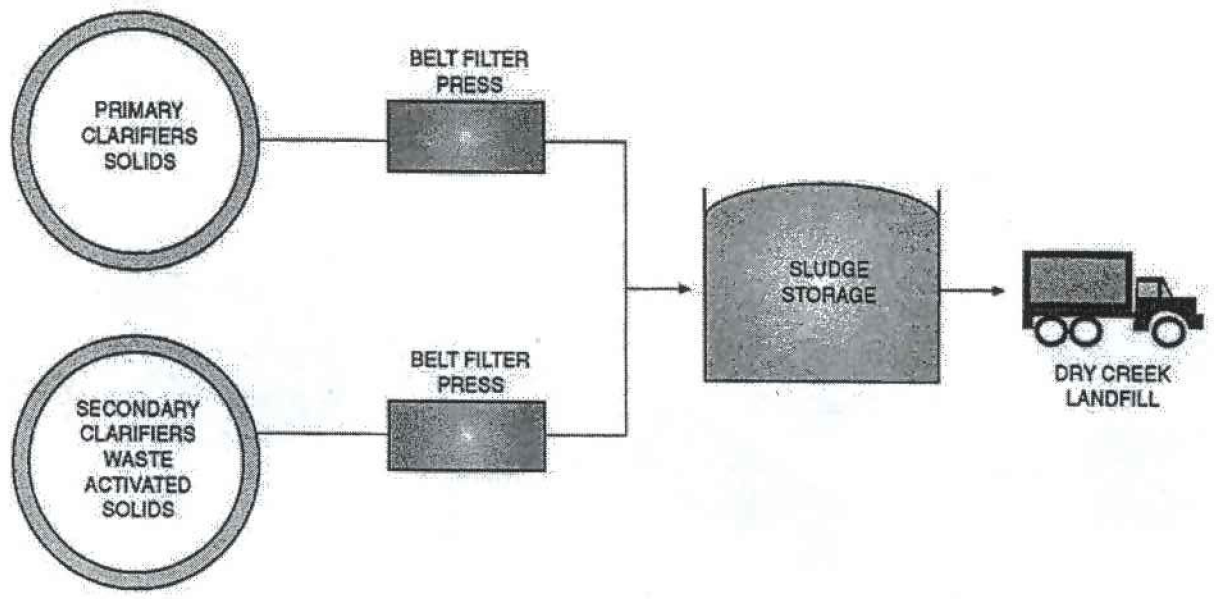

Figure 10-3
Alternative 2

ALTERNATIVE TWO - DRY CREEK LANDFILL

Alternative Two is to haul dewatered biosolids to the Dry Creek Landfill. Figure 10-3 presents the flow schematic for this alternative. Both raw primary and secondary biosolids would be dewatered at the plant to 15 percent and hauled approximately 35 miles in trucks. A new belt filter press would be installed in the existing chlorine building that would also house the existing press. Additional hauling equipment would be required for transporting the biosolids to the landfill.

By hauling the dewatered raw biosolids to the landfill, the grit removal, gravity thickener, gravity belt thickener, anaerobic digester, tank trucks, and land irrigation equipment could be eliminated.

A major disadvantage of this alternative would be if future regulatory requirements restrict the disposal of biosolids in the landfill. Therefore, this alternative could never be a sole source for biosolids disposal. However, it could be used as a temporary backup for the co-composting alternative in the event of an equipment failure or inclement weather.

ALTERNATIVE THREE - LAND APPLYING CLASS B BIOSOLIDS

Alternative Three is a continuation of the current biosolids management program of long hauling Class B biosolids and land applying. See Figure 10-1 for the flow schematic of this alternative. However, because of the projected increase in biosolids, the land application would be expanded. The City would contract with large landowners in Eastern Oregon to apply these Class B biosolids to farmland. Biosolids would be dewatered to 15 percent, hauled to the site in large tractor/trailers, and applied with a manure spreader. During the winter, the biosolids would be stored near the application site.

Additional components would be necessary to meet the demands of the increased biosolids production. Several of the existing components would be duplicated, which include the gravity thickener, gravity belt, anaerobic digester, and belt filter press. The existing anaerobic digester would also need to be rehabilitated.

A new covered storage building would be built near or at the application site to hold three months worth of biosolids. A site for the storage building has not been selected.

Another consideration is producing Class A biosolids using co-composting. A pilot scale co-composting facility could occur on site, located in the future biosolids storage building, utilizing low-cost aeration equipment. As discussed in Alternative One, the benefits of producing Class A biosolids would be the potential to produce "EQ" solids which could be distributed to the public.

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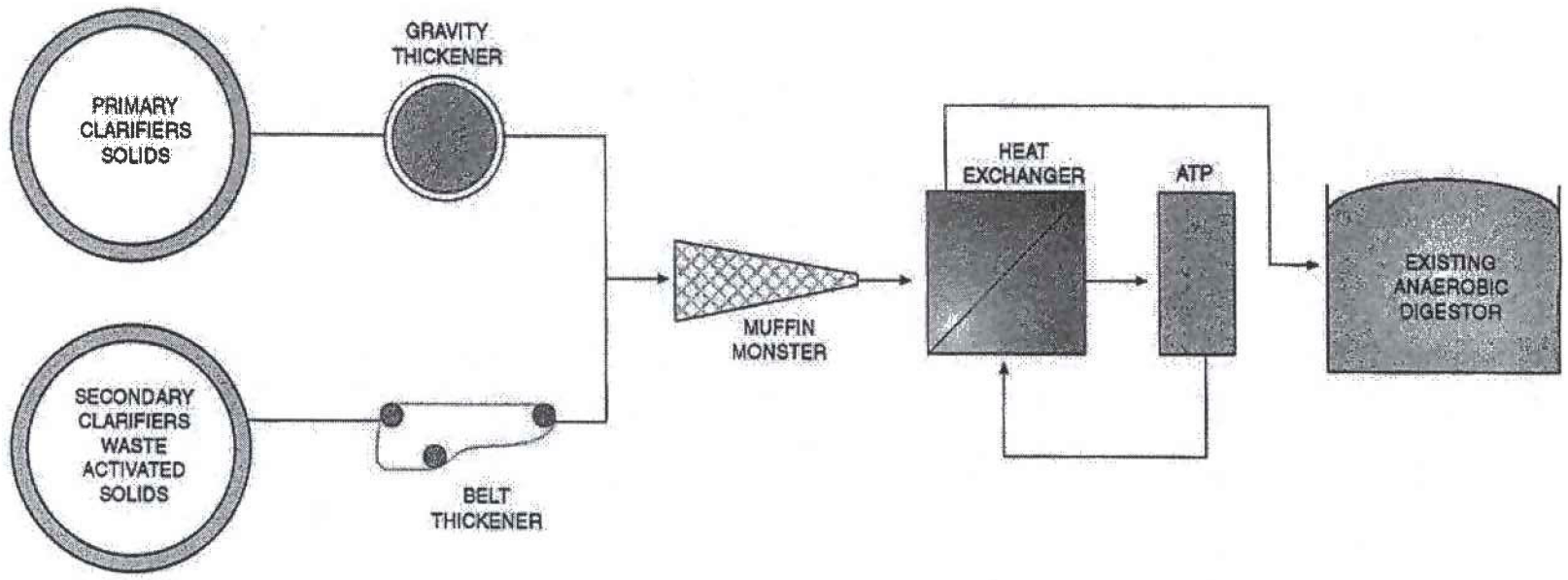

Figure 10-4
Alternative 4

ALTERNATIVE FOUR – AEROBIC THERMOPHILIC PRETREATMENT (ATP)

In lieu of another anaerobic digester, this alternative would install an Aerobic Thermophilic Pretreatment (ATP) component to accommodate the future load increase. Figure 10-4 presents the flow schematic for this alternative. A Muffin Monster would be placed prior to the ATP to reduce the size of plastics in the biosolids.

ATP is a two-stage digestion process that reduces pathogens and conditions the biosolids prior to anaerobic digestion. The thermophilic process has a detention time of about 24 hours, thus reducing the vessel size and associated footprint.

Replacing a new anaerobic digester has several advantages, which include:

- The elimination of heating the digester.
- The reduction in footprint, which is a reduced capital cost.
- The ATP produces biosolids listed as "Exceptional Quality," therefore the end product can be sold to the public.

Despite these advantages, the ATP does have high operating costs, which weighs heavily against the selection of its use.

ANTICIPATED COST ESTIMATE FOR BIOSOLIDS ALTERNATIVES

For each alternative, a preliminary cost estimate was developed. Tables 10-2 through 10-5 list Alternatives One through Four, respectively. Although the Dry Creek Landfill alternative is the least expensive method, the risk of regulatory change could require re-evaluation of this alternative in the future. Therefore, this alternative has been eliminated. While land applying Class B biosolids offers staff familiarity with the components, this alternative has the greatest cost and was eliminated. The ATP has a lower capital cost; however, because of operational complexity and high operating costs, this alternative was eliminated.

The co-composting facility at Merlin Landfill presents a moderate capital cost and potential for a profitable end product that would be sold to the public.

Table 10-2. Capital Cost for Alternative One – Merlin Landfill Co-compost Facility

Project Item	Cost
Existing Digester Rehabilitation	$740,000
Dewatering Device	$1,000,000
Co-composting Facility	$1,850,000
Subtotal	$3,590,000

Table 10-3. Capital Cost for Alternative Two – Dry Creek Landfill

Project Item	Cost
New Belt Press installed in existing building and move existing Belt Press	$811,200
Hauling Equipment	$552,500
Subtotal	$1,363,700

Table 10-4. Capital Cost for Alternative Three – Land Applying Class B Biosolids

Project Item	Cost
Belt Thickener	$960,000
New Anaerobic Digester	$1,800,000
Existing Digester Rehabilitation	$740,000
Dewatering BFP	$2,730,000
Land	$370,000
Off-Site Improvements	$550,000
Dewatered Sludge Storage Building	$2,950,000
Biosolids Handling Equipment	$550,000
Co-compost Pilot	$370,000
Subtotal	$11,020,000

Table 10-5. Capital Cost for Alternative Four – Aerobic Thermophilic Pretreatment

Project Item	Cost
Aerobic Thermophilic Pretreatment	$2,340,000
Existing Digester Rehabilitation	$740,000
Muffin Monster	$78,000
Subtotal	$3,158,000

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PREFERRED ALTERNATIVE

To choose the best alternative for the Grants Pass WRP, both feasibility and cost were analyzed. Table 10-6 compares the advantages and disadvantages of each alternative.

Table 10-6. Summary of Alternatives

ADVANTAGES	DISADVANTAGES
ALTERNATIVE ONE – Merlin Landfill Co-compost Facility	
Moderate operating cost.	Possible landuse conflicts at landfill.
Provides co-compost to community.	Moderately high initial capital cost.
Limits primary biosolids health risk.	
ALTERNATIVE TWO – Dry Creek Landfill	
Already utilized by plant during winter.	Long-term permitting may be an issue from DEQ.
Least capital cost option.	Tipping fee for public may increase.
Low operating cost.	City retains responsibility for possible environmental impacts at landfill.
No immediate permitting issues.	
Easy to implement.	
ALTERNATIVE THREE – Land Applying Class B Biosolids	
Biosolids used as resource.	Second digester required at plant.
Extensive land available.	Contract with property owners uncertain.
Permitting likely easy.	City retains responsibility for possible environmental impacts.
Climate reduces environmental concerns.	Hauling difficulties in bad weather.
ALTERNATIVE FOUR – Aerobic Thermophilic Pretreatment (ATP)	
Class A product with "Exceptional Quality" possible.	Public education necessary on Class A biosolids reuse.
Fertilizer available to community in lieu of co-compost.	Market uncertain for final product.
Permitting issues moderate.	High operating cost.
Moderate capital cost.	

The preferred alternative for Grants Pass is the Merlin Landfill Co-compost Facility (Alternative One) that co-composts digested primary and raw secondary biosolids. This method presents reasonable cost and provides co-compost material to the public.

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CHAPTER 11

PREFERRED ALTERNATIVE

This chapter presents the preferred alternatives for both the liquid and solid treatment streams. As presented in Chapter 9, the Ballasted Sedimentation alternative (Alternative Two) was the preferred liquid stream alternative. The biosolids handling and disposal alternatives were presented in Chapter 10, with the Merlin Landfill Co-Compost Facility (Alternative One) being the preferred alternative. Figure 11-1 presents the proposed layout of the upgrade of the Grants Pass WRP.

LIQUID STREAM PREFERRED ALTERNATIVE

Headworks

At the headworks, an additional mechanical bar screen would be installed with a 23.5 mgd capacity. This would provide redundancy and a maximum capacity of 47 mgd.

Also at the headworks, it would be necessary to install an additional pump to meet the firm capacity of 38 mgd. Additionally, an evaluation would be necessary to determine if the current pumps are in proper working condition and if any need to be replaced.

Due to the plant location and because the headworks are the primary source for odor, it would be necessary to install odor containment at the influent pump station and the mechanical screening area.

Primary Clarification/Gravity Thickener

The existing 70-foot circular primary clarifier would be converted to a combination primary clarifier/gravity thickener. All of the sludge from the rectangular clarifiers would be pumped to this one unit to be thickened before transfer to the digester. This combination offers easier control of sludge pumping because it is from only one unit, rather than several units. This conversion would satisfy the need for primary solids thickening well beyond the 20-year design period.

Aeration Basin

Currently, there are two aeration basins, each segmented into three bays, operating in a plug flow mode. These basins are capable of handling the future flows of 13.5 mgd as long as the secondary clarifier capacity is available. To improve the settling performance in the secondary

clarifiers and provide filamentous bacteria control, the addition of a bioselector in the basin is suggested. The smallest bay would be segmented into two equal zones and aerated with coarse bubble diffusers. This segmented basin would provide the anoxic treatment zone to control the bacteria. The rest of the existing aeration basin would be modified by the addition of fine bubble diffusers, dissolved oxygen control, and motorized gates.

Secondary Clarifiers

The two 75-foot diameter by 13-foot side water depth (SWD) clarifiers would need to be rehabilitated to correct short-circuiting and flow distribution problems. Proposed modifications include weir adjustments, additional skimmers, baffle upgrades, and balancing of flow between clarifiers. Each of the existing clarifiers has a flow rating of 10.6 mgd, which is sufficient for current flows. However, construction of two additional clarifiers would be necessary to meet future flows, one in the year 2010 and the other in the year 2020. These 90-foot diameter by 14-foot SWD clarifiers would add an additional capacity of 22 to 25 mgd to the plant. These new clarifiers would be constructed next to the existing clarifiers.

Ultraviolet Disinfection

Since the completion of the second UV channel, the plant capacity for disinfection is 43 mgd. Therefore, no additional UV channel construction during the 20-year planning period is necessary.

Outfall

An outfall diffuser would be added to the existing outfall to improve mixing and reduce ammonia toxicity in the Rogue River.

Miscellaneous Plant Improvements

To update the Grants Pass WRP, several miscellaneous upgrades have been included. These improvements include a plant equipment audit, additional plant landscaping, the development of a public education program, laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion. The yard piping would also be upgraded as necessary.

To evaluate any infiltration and inflow problems, a Sewer System Master Plan will be conducted in 2000-01. This document would provide the necessary information for collection system corrections.

BIOSOLIDS HANDLING AND DISPOSAL PREFERRED ALTERNATIVE

Alternative One, the preferred alternative from the VE Workshop, would co-compost digested primary and raw secondary biosolids. This alternative consists of rehabilitating the existing digester, a new dewatering device, and a co-composting facility. This new biosolids handling system increases the capacity to effectively manage biosolids of the Grants Pass WRP.

Under the preferred biosolids alternative, the existing digesters would be rehabilitated and used to treat only primary clarifier solids. Secondary clarifier solids would be combined with the primary solids from the digester, dewatered in a new device, and trucked to the new co-composting facility at the Merlin Landfill. The existing belt dewatering system would remain on site for standby use.

The City would pilot-test a dewatering device, such as a centrifuge or another belt filter press. Based on these results, a dewatering device would be selected by the City.

Co-Composting

The proposed co-composting facility would be located at the Merlin Landfill property, near the center of the 316-acre parcel, and would produce Class A biosolids for sale to the public. Figure 11-2 is a conceptual diagram of the proposed co-composting operation.

The new facility would co-compost green wastes (i.e., yard debris, animal manure, and wood waste) from the general public with the biosolids from the Grants Pass WRP. The facility would also co-compost dewatered septage (70% solids) from Clearwater Co-op. Biosolids from the Grants Pass WRP would be approximately 15% solids when delivered to the co-composting operation. The green waste and other bulking material would be added to the biosolids at an approximate ratio of 3.5 parts green waste to 1 part biosolids (on a volumetric basis).

The co-composting facility would utilize the Extended Aerated Static Pile (EASP) method of co-composting. This method uses perforated pipes installed beneath the co-compost pile and connected to electric blowers that can either force air into the co-compost pile (positive aeration) or draw air through the pile (negative aeration).

As the co-compost pile is constructed, a 12-inch-thick (minimum) cover of finished co-compost would be placed over the new feedstock materials. The cover serves four purposes, including 1) a blanket to ensure that all of the materials reach desired temperatures, 2) a moisture barrier to maintain the desired moisture content within the pile, 3) a biofilter to remove offensive odors, and 4) a vector barrier to control flies, birds, and rodents.

The aeration system is designed to periodically deliver fresh air into the co-compost pile, with the goal of maintaining an oxygen level of at least 8 percent throughout the pile (i.e., aerobic conditions). Adjusting the rate of aeration is the principle means of controlling pile temperatures and mitigating potentially offensive odors. Oxygen within the pile stimulates microbial activity,

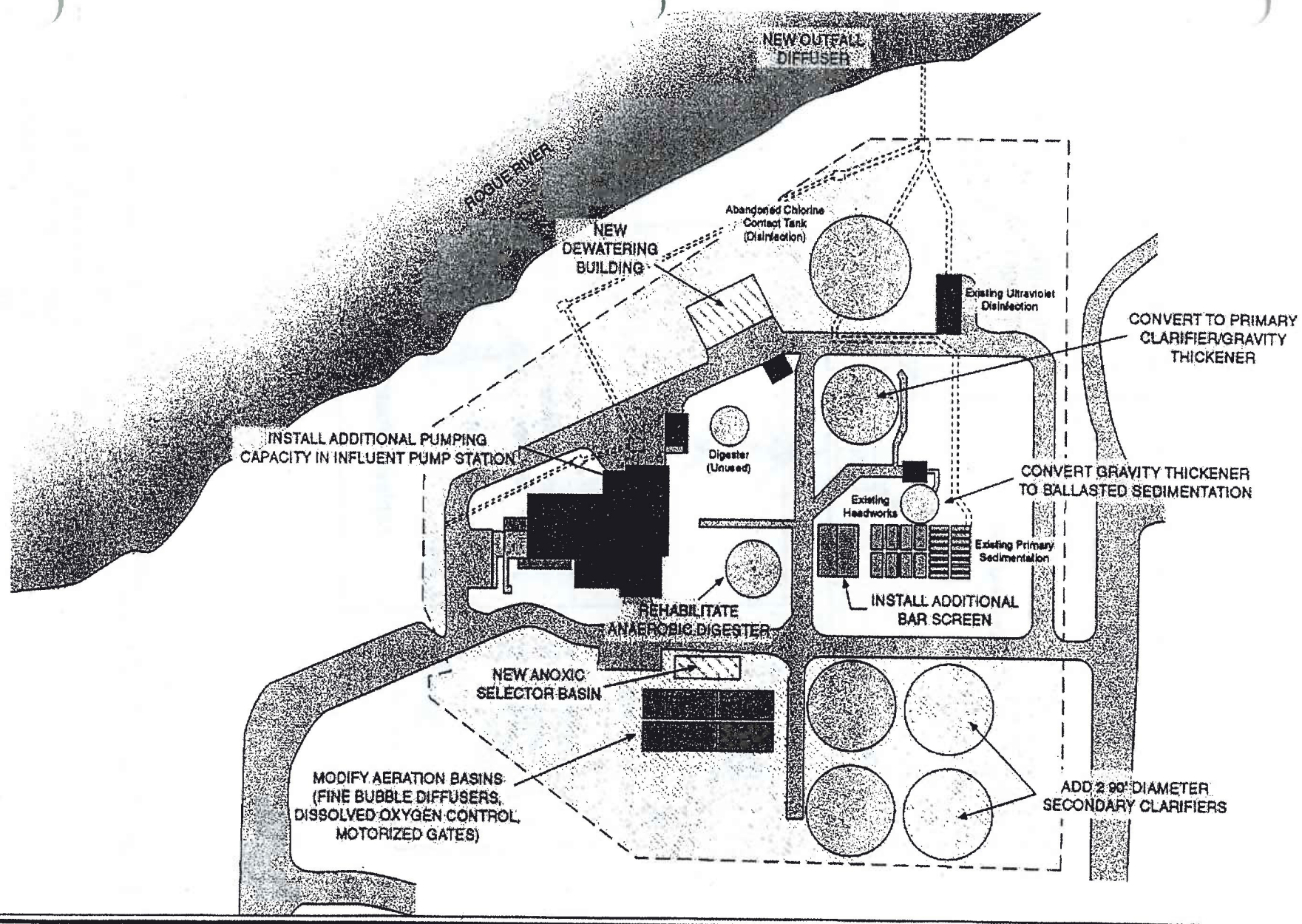

Figure 11-1
Preferred Alternative
Site Plan

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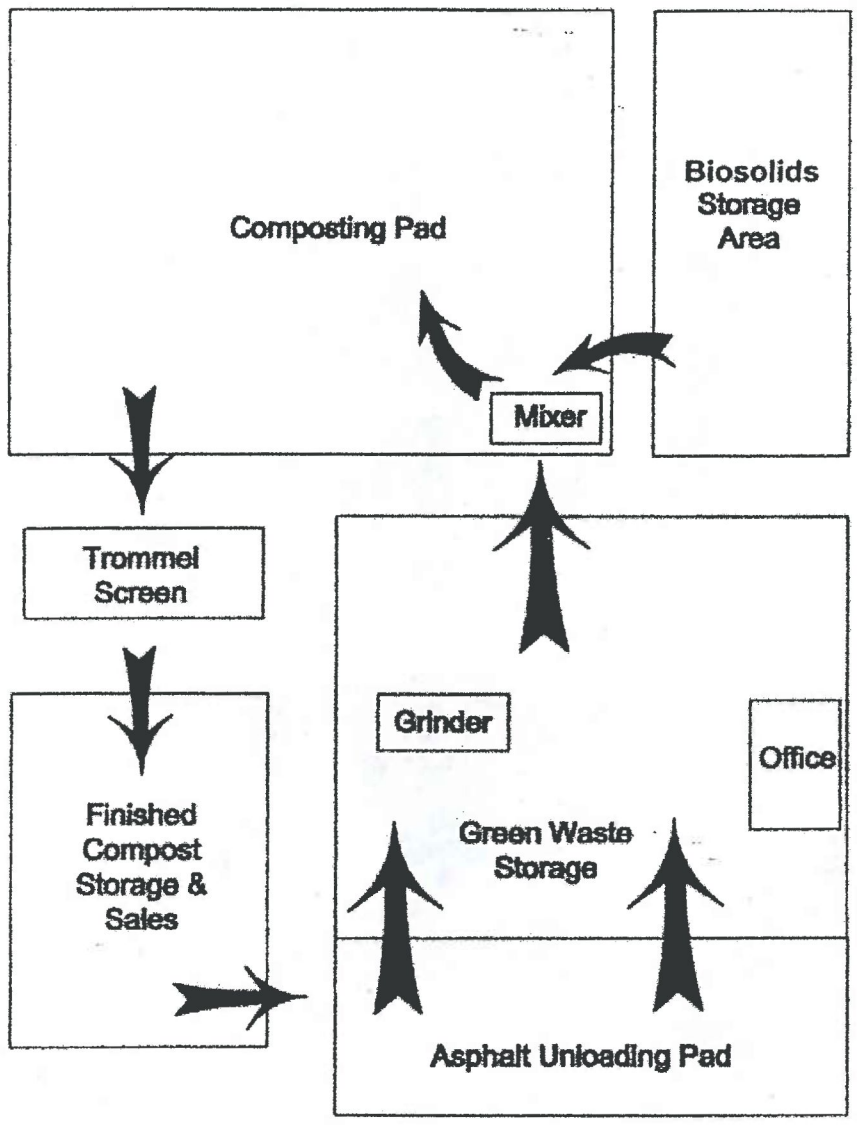

Parametrix, Inc. City of Grants Pass Facilities Plan/216-3416-002(02) 01/00 (S)

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Figure 11-2
City of Grants Pass
Merlin Landfill
Co-Composting Facility
Site Layout

optimizes digestion of the compostable materials (i.e., volatile solids), and produces a superior compost product.

The active phase of co-composting (the period during which most of the process heat and potentially offensive odors are generated) lasts between 21 and 30 days. The subsequent curing phase of co-composting (the period during which the product becomes stable and marketable), lasts an additional 45 to 60 days. The curing phase can be accomplished either in the original co-compost pile, with adjusted airflow, or in a separate stockpile. The screening process can take place either before or following product curing.

CONSTRUCTION SCHEDULE

For the proposed liquid stream upgrade, construction would occur in three phases. Below is a breakdown of when these modifications would occur. The biosolids handling and disposal plan would go into effect immediately.

Year 2000

- Install odor containment and control at the influent pump station and mechanical screening areas of the plant.
- Add an anoxic selector basin to the aeration basin system.
- Modify the existing aeration basin with fine bubble diffusers, and add dissolved oxygen control and motorized gates. Provide new aeration blowers.
- Rehabilitate the existing secondary clarifiers to correct short circuiting and flow distribution problems.
- Add a single new secondary clarifier.
- Begin laboratory upgrades and improvements.
- Begin operations building repairs and modifications.
- Begin instrumentation and control system expansion.
- Conduct a plant equipment audit.
- Continue to install plant landscaping.
- Develop and implement a public education program.

Year 2005

- Install additional influent pumping capacity.
- Add a second mechanical bar screen.

- Convert the existing circular primary clarifier to a combination primary clarifier/gravity thickener.
- Modify the existing gravity thickener to a ballasted sedimentation tank to treat peak flow.

Year 2010

- Add a second new secondary clarifier.

CONSTRUCTION COST ESTIMATE

The preliminary cost estimates for the liquid and solids treatment alternatives and the collection system improvements are presented in Table 11-1. Costs are presented in 1999-2000 dollar amounts and include engineering, administration, and contingency.

Table 11-1. City of Grants Pass - Water Restoration Plant Capital Improvement Plan*

		Implementation Schedule		
Project Item	Probable	2001-2004	2005-2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping	$560,000		$560,000	
Raw Sewage Pipeline to Accommodate Ballasted Sedimentation	$240,000		$240,000	
Headworks Odor Control	$390,000	$390,000		
Mechanical Bar Screen No. 2	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000		$3,510,000	
Modify Existing Primary to Combination Clarifier/	$610,000		$610,000	
Yard Piping	$270,000		$270,000	
SECONDARY TREATMENT				
Aeration Basin Fine Bubble	$530,000	$530,000		
Aeration Basin Selector	$260,000	$260,000		
Blowers and DO Control	$870,000	$870,000		
Rehabilitate Existing Clarifiers	$770,000	$770,000		
New Secondary Clarifiers	$2,400,000	$1,200,000		$1,200,000
Yard Piping	$770,000	$400,000		$370,000
Motorized Gates	$340,000	$340,000		
FINAL TREATMENT				
Outfall Diffusers	$540,000	$540,000		

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Table 11-1. City of Grants Pass - Water Restoration Plant Capital Improvement Plan* (cont.)

Project Item	Probable	Implementation Schedule 2001-2004	2005-2006	2010-2011
OTHER PLANT IMPROVEMENTS				
Lab Improvements	$100,000	$100,000		
Operation Building Repairs and Office	$130,000	$130,000		
SCADA System Expansion	$670,000	$250,000	$250,000	$170,000
Equipment Improvements from Audit Results	$250,000	$100,000	$75,000	$75,000
Plant Landscaping	$100,000	$40,000	$30,000	$30,000
Public Education Program	$50,000	$20,000	$20,000	$10,000
SOLIDS THICKENING AND DIGESTION				
Rehabilitate Existing Digester	$740,000	$740,000		
Dewatering Centrifuge	$1,000,000	$1,000,000		
SOLIDS HANDLING OFF-SITE				
Co-composting Facility	$1,850,000	$1,850,000		
COLLECTION SYSTEM				
Pine Street	$1,050,000	$1,050,000		
2nd Street	$700,000	$700,000		
Western Avenue	$580,000	$580,000		
Master Plan	$170,000	$170,000		
TOTALS	$19,680,000	$12,030,000	$5,795,000	$1,855,000

* 1999-2000 dollar amounts.

ENVIRONMENTAL REVIEW

Introduction

The purpose of this environmental review is to summarize the potential adverse impacts associated with the No Action Alternative and the two build alternatives evaluated for improvements to the Grants Pass WRP and the disposal of biosolids. The Grants Pass Water Restoration Plant is owned and operated by the City of Grants Pass. This section includes a characterization of the natural and human elements in the study area, economic considerations, and a summary of public participation. Natural and human elements considered in this chapter include land uses and zoning, historic and cultural resources, wetlands, floodplains, agricultural lands, wild and scenic rivers, fish and wildlife, threatened and endangered species, and other unique or sensitive environmental resources. The No Action Alternative and the two build alternatives are defined below.

No Action Alternative

Grants Pass Water Restoration Plant

The No Action Alternative is defined as continuing operation of the existing Grants Pass WRP without any upgrades to the facility. Under this alternative, new connections to the collection system sewered by the plant would be prohibited to avoid the plant exceeding the maximum permitted capacity. The collection system sewered by the plant serves the Grants Pass Urban Growth Area. Additionally, a National Pollution Discharge Elimination System (NPDES) permit would be issued in 2001, which would likely subject the plant to future violations regarding needed outfall modifications and possibly more stringent effluent standards. Therefore, this alternative is not considered feasible and has been eliminated from further consideration.

Biosolids Disposal

Currently, biosolids from the Grants Pass WRP are land applied from approximately April to October. From approximately November to March, the biosolids are disposed of at the Merlin Landfill. However, the Merlin Landfill is closing in October 2000. Once this occurs, the RSSSD proposes to dispose of biosolids at the Dry Creek Landfill, approximately 35 miles to the southwest, during the rainy season from November through March. During the months of April through October, disposal of biosolids would continue to be by land application. Specific sites for land application of the biosolids have not been identified. Finding acceptable sites to land apply the biosolids is becoming more difficult, and the hauling distances to these sites from the Grants Pass WRP continue to increase.

Preferred Alternative

Grants Pass Water Restoration Plant

The Preferred Alternative proposes modifications and improvements to the existing Grants Pass WRP. This alternative was selected during a Value Engineering Workshop held in June 1999. For the liquid stream primary treatment, the desire is to install odor containment and control at the influent pump station and mechanical screening areas of the plant. An additional influent pump would be installed to increase capacity. The existing circular primary clarifier would be converted to a combination primary clarifier/gravity thickener, while the existing gravity thickener would be modified to a ballasted sedimentation tank to treat peak flow.

For the liquid stream secondary treatment, the existing aeration basin would be modified to include fine bubble diffusers, dissolved oxygen control, and motorized gates. An anoxic selector would also be added prior to the aeration basin for better filamentous bacteria control. Rehabilitation of the existing secondary clarifiers is necessary to correct short-circuiting and flow distribution problems. To meet future flow demands, two additional secondary clarifiers would be constructed. For the final treatment, the only required plant improvement would be to install an outfall diffuser. Other miscellaneous plant improvements include laboratory upgrades and improvements, operations building repairs and modifications, and instrumentation and control system expansion.

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Biosolids Disposal
The Preferred Alternative for biosolids handling and disposal would be to rehabilitate the existing digester and install a new dewatering centrifuge. A new co-composting facility would also be constructed off-site at the Merlin Landfill for biosolids disposal.

Second Preferred Alternative

Grants Pass Water Restoration Plant
The Second Preferred Alternative proposes more extensive modifications and improvements to the Grants Pass WRP. This alternative is the recommended alternative identified in a previous Facilities Plan for the Grants Pass WRP (January 1999). For the liquid stream primary treatment, greater capacity pumps would replace the existing pumps and an additional bar screen would improve the headworks to meet future flow demands. Two additional rectangular primary sedimentation basins would be constructed and the existing primary circular basin would be rehabilitated. The aeration basin, the blowers, and aeration equipment would be upgraded and a second aeration basin would be constructed. The existing secondary clarifiers would be rehabilitated and three additional secondary clarifiers would be constructed. One additional ultraviolet disinfection channel would be constructed to meet the anticipated peak hydraulic capacity. An outfall diffuser would also be installed to improve mixing and reduce ammonia.

Biosolids Disposal
The Second Preferred Alternative for the biosolids handling and disposal would be to construct a second gravity thickener and a second belt thickener to meet future waste activated sludge loading. The existing digester would be rehabilitated and a second anaerobic digester would be constructed. A permanent dewatering facility would be constructed at the present temporary trailer mounted belt filter press and would be relocated in the building. In addition, a new press would be added.

This alternative involves the disposal of biosolids at the Dry Creek Landfill during the rainy season from November through March. The biosolids would be transported from the Grants Pass WRP to the Dry Creek Landfill facility, which is located approximately 35 miles to the southwest. During the months of April through October, disposal of biosolids would be by land application although the specific sites have not been identified.

The disposal of biosolids by land application is regulated by the Department of Environmental Quality (DEQ). DEQ regulates land use application of biosolids through the existing NPDES permit for the City of Grants Pass. Because the biosolids often contain pathogens, heavy metals, nitrogen, phosphorous, and ammonia, there is a potential for significant adverse impacts to human health and natural resources. Runoff from land use application would be considered a non-point pollution source to nearby streams and wetlands. The DEQ has a 100-foot buffer requirement from land use application areas to water bodies and a 50-foot buffer to surrounding properties. The odor from the land use application is also a concern to the surrounding area.

Zoning and Land Uses

Grants Pass Water Restoration Plant
The Grants Pass WRP is located on a parcel that is zoned as moderate density residential with an allowable 6,000 sf lot, density (R-1-6). The existing site is occupied by the Grants Pass WRP, and the remaining parcel is primarily open undeveloped land. The land has been relatively disturbed by the construction of the plant facilities and the various grading, leveling, and filling that has occurred. In the northeastern corner of the property, there is an effluent experimental treatment test area that has been planted with poplar trees and is artificially irrigated. The Grants Pass WRP is bordered on the south by the Rogue River; to the north by single family detached residences zoned R-1-6; to the east by open space and by single family detached residences, which are zoned R-1-6; and to the west by single family residences and the All Sports Park which are zoned low density residential with an allowable 8,000 sf lot, density (R-1-8).

The existing Grants Pass WRP is considered a minor public facility which is a permitted use in the R-1-6 zoning district. The proposed improvements in the Preferred Alternative are also permitted uses and would require a development permit review by a Type III Procedure. The Type III Procedure requires review at a public hearing before the Hearings Officer or the Planning Commission.

Biosolids Disposal
In the Preferred Alternative, disposal of biosolids at a co-composting facility located on a 5-acre parcel within the existing 100-acre Merlin Landfill is proposed. The 100-acre Merlin Landfill is an active municipal solid waste landfill. Although closure of the Merlin Landfill is expected in October 2000, the co-composting facility would remain in operation. Land uses surrounding the landfill are primarily agriculture. This parcel is zoned Woodland Resource (WR). Construction of the co-composting facility in this zone would require a Conditional Use Permit (CUP).

Although construction of the co-composting facility may cause temporary traffic impacts on adjacent roads, no adverse impacts to land uses are anticipated as a result of the Preferred Alternative. If the Preferred Alternative is chosen, than the City of Grants Pass will prepare a CUP to build the co-composting facility at the Merlin Landfill.

Disposing of the biosolids at either Dry Creek Landfill or by land application in the Second Preferred Alternative would require transporting the biosolids to locations that are further in distance than the Preferred Alternative. Increasing the hauling distance of the biosolids would possibly increase traffic on some roadways. This increase is not expected to be significant, and no adverse impacts to land uses are anticipated as a result of the Second Preferred Alternative.

Historic and Cultural Resources

A pedestrian survey conducted on January 7, 2000, at both the Grants Pass WRP and the proposed off-site co-composting facility at the Merlin Landfill did not reveal evidence of either prehistoric or historic archaeological resources.

Based on the results of the surveys, no additional archaeological research is recommended at either location. However, there is always the possibility that ground disturbance during construction activities may expose buried cultural material or human burials that were not detected during the survey. If such an event should occur, Oregon State law (ORS 97.740, 97.760, 358.905, 390.235, and 358.955) and various federal laws and regulations, which may be applicable to this project, will require that work in the vicinity of such finds be suspended. The State Historic Preservation Office (SHPO) and the appropriate tribes should be notified, and a qualified archaeologist would be called in to evaluate the discovery and recommend subsequent courses of action in consultation with the tribes and SHPO.

There are no anticipated adverse impacts to historic and cultural resources for the Preferred Alternative or the Second Preferred Alternative.

Economic Considerations

The Preferred Alternative would be financed by a combination of funding mechanisms, including loans and cash. This $17.5 million project would be constructed when Clean Water State Revolving Fund (CWSRF) loan monies become available. Sewer rates would not increase as a result of these upgrades; therefore, there would be no adverse economic impact to service area users. If rates do increase, it would be a result of inflation.

The City of Grants Pass is fully funding the co-composting facility which is proposed to be located at the Merlin Landfill. The public would contribute monies towards the co-composting facility by paying a yard waste drop-off fee and a fee to purchase the compost.

The construction costs associated with the Preferred Alternative are $12 million less than the costs associated with the Second Preferred Alternative.

Wetlands

Grants Pass Water Restoration Plant
There were no wetlands identified within the existing Grants Pass WRP property. Upgrades to the existing facilities and the construction of new structures proposed in the Preferred Alternative and in the Second Preferred Alternative would have no impact on wetlands, and no permitting would be required.

The Rogue River is a designated "Water of the US," which is considered a jurisdictional wetland. The placement of an outfall diffuser structure proposed in the Preferred or Second Preferred Alternative in the river would require environmental permits from the Oregon Division of State Lands (ODSL) and the Army Corps of Engineers (Corps).

Biosolids Disposal

The Preferred Alternative would potentially impact wetlands within the 5-acre portion of the proposed co-composting facility. At the time of the site visit in January 2000, a seepage slope that drains into an ephemeral stream in the center of the 5-acre site was identified and there was no apparent flow into the stream. The seepage slope appeared to collect the surface water drainage from the surrounding area and flow down gradient until eventually flowing into the stream. There was evidence of wetland characteristics including hydrophytic vegetation, hydrology, and hydric soils in the seepage slope and the ephemeral stream. The three wetland criteria have been met and would likely be considered jurisdictional. Any adverse impacts to wetlands would require permits from ODSL and Corps. A wetland delineation to identify the exact boundary and extent of the wetlands within this 5-acre portion would be recommended.

Under the Second Preferred Alternative, disposal of biosolids in an already permitted landfill facility would have no adverse affect on wetland or stream systems. The land application of biosolids could have a significant impact on wetland and stream systems if the run-off contained pathogens, heavy metals, nitrogen, phosphorous, and ammonia.

Floodplains

Grants Pass Water Restoration Plant

The Grants Pass WRP is located in Zone AE, which is considered to be an area of special flood hazard (for the 100-year flood)[1]. The base flood elevation for the Grants Pass WRP parcel is approximately 910 feet. The proposed improvements to the plant will comply with the local, state, and federal base flood elevation height requirements.

The Preferred Alternative proposes to install an outfall diffuser for the liquid stream final treatment. The outfall diffuser would discharge into the floodway area of Zone AE (Rogue River). The average daily flow rate from the Grants Pass WRP in 1997 was 5.10 mgd, and the average flow of the Rogue River at the Grants Pass station is 2,135 mgd. In other words, the Grants Pass WRP's daily flow rate is .002% of the Rogue River's flow; therefore, addition of the proposed improvements to the existing facility should not alter or affect flooding in this area.

The Second Preferred Alternative also proposes to install an outfall diffuser for the liquid stream final treatment. As with the Preferred Alternative, addition of an outfall diffuser should not alter or affect flooding in this area.

[1] Federal Emergency Agency (FEMA) Flood Insurance Rate Map (FIRM) community panel #410108 003 C Panel 3 of 4, September 27, 1991.

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Biosolids Disposal
The proposed co-composting facility in the Preferred Alternative is not located within a flood zone; therefore, there are no anticipated adverse impacts to floodplains[2]. The Second Preferred Alternative does not propose to dispose of biosolids in floodplains; therefore, there are no anticipated adverse impacts to floodplains.

Agricultural Lands

Grants Pass Water Restoration Plant
The Grants Pass WRP improvements proposed in both the Preferred Alternative and Second Preferred Alternative are to an existing facility that is zoned as moderate density residential (R-1-6). No agricultural activities are conducted on the site; therefore, no disruption of agricultural activities or displacements of agricultural land will occur as part of this project.

Biosolids Disposal
The proposed co-composting facility in the Preferred Alternative would be constructed on a 5-acre parcel within the existing 100-acre Merlin Landfill, which is zoned Woodland Resource. No agricultural activities are conducted on the site; therefore, no disruption of agricultural activities or displacements of agricultural land should occur as part of this project.

The Second Alternative proposes to dispose of the biosolids at an existing landfill and land applying the biosolids. Disposal of biosolids at the existing landfill would not disrupt any agricultural activities. Land application of biosolids is not anticipated to have adverse impacts to agricultural lands.

Wild and Scenic Rivers

Grants Pass Water Restoration Plant
A section of the Rogue River has been designated as a wild and scenic river under the 1968 Wild and Scenic Rivers Act. The wild and scenic river designation begins at the confluence of the Applegate River and extends approximately 84 miles to the Lobster Creek Bridge. Because the project is not located within the wild and scenic area of the Rogue River, neither the Preferred Alternative or the Second Preferred Alternative would have impacts to the area designated as a wild and scenic river.

Biosolids Disposal
The Preferred Alternative proposes biosolids disposal at the proposed co-composting facility at the Merlin Landfill. There are no designated wild and scenic rivers within the vicinity of the Merlin Landfill; therefore, the Preferred Alternative would have no adverse impacts to any designated wild and scenic rivers.

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[2] Federal Emergency Agency (FEMA) Flood Insurance Rate Map (FIRM) community panel #415590 0226 Panel 226 of 500, September 27, 1991.

The Second Preferred Alternative proposes biosolids disposal through land application from April through October, and landfilling from November through March. Land application sites have not been identified. Landfill disposal would occur at Merlin Landfill until October 2000, when it closes, and then would occur at the Dry Creek Landfill. The Dry Creek Landfill is not located near a wild and scenic river, nor would sites that are selected for land application be located near a wild and scenic river; therefore, the Second Preferred Alternative is not anticipated to have adverse impacts to wild and scenic rivers.

Fish and Wildlife

Correspondence with the Oregon Natural Heritage Program (ONHP), U.S. Fish and Wildlife Service (FWS), and National Marine Fisheries Service (NMFS) was initiated to identify any federally listed, proposed, or candidate threatened or endangered species that may potentially occur in the project area. Agency correspondence indicated that the southern Oregon/northern California coastal population of coho salmon *(Oncorhynchus kisutch)*, a federal threatened species; the Klamath Mountains province population of steelhead *(O. mykiss)*, a federal candidate species; and the southern Oregon coast fall-run population of chinook salmon *(O. tshawytscha)*, a federal proposed threatened species, may be present in the proposed action area. If required at a later time, potential impacts to listed species identified in the agency correspondence would be addressed in a Biological Assessment (BA), which would be conducted after final design is underway.

Fish – Grants Pass Water Restoration Plant

Potential impacts to fish species from both the Preferred Alternative and the Second Preferred Alternative are essentially the same. Both alternatives include upgrades to the secondary treatment process, which should enable the plant to meet basin-wide discharge standards for biochemical oxygen demand (BOD) and suspended solids (SS). Since the Rogue River is water quality limited, enhancing water quality at the Grants Pass WRP outfall, under either alternative, will benefit fish species in the Rogue River.

Both alternatives also propose to add an outfall diffuser in the Rogue River. The addition of a 24-inch-diameter outfall diffuser to the Grants Pass WRP will improve habitat for fish species by improving effluent mixing, eliminating the existing shore-hugging effluent plume, and addressing the potential problems to fish caused by ammonia toxicity within the mixing zone. The construction and placement of the outfall diffuser in the streambed at the existing outfall may result in short-term direct impacts on salmonid species unless properly mitigated, as described below. These potential impacts may include disturbance of spawning adult salmon at the project site, disturbance of eggs and alevins due to redd (shallow depression in the stream gravel dug by the female salmon before spawning) disruption during construction, excessive sedimentation of redds from stream bed disturbance during outfall construction, and disturbance and displacement of rearing juveniles from available habitat near the project site during outfall construction.

Fish impacts can be mitigated through timing the in-water construction to occur when no salmonids are spawning, incubating, or rearing; however, the loss of the stream bed and fish habitat where the outfall would sit cannot be mitigated. The on-shore construction site can be mitigated by the use of silt fencing, settling pond, off-site staging area, and spill control plan.

All in-water work shall be performed during the ODFW preferred in-water work period for this reach of the Rogue River (June 15 to August 31). Any in-water work outside of this time frame is prohibited unless written permission to change or extend the in-water work period is received from the ODFW Rogue River district fisheries biologist, the National Marine Fisheries Service, and an ODOT biologist. In-water work is defined as "any work done in, on, under, or within the active channel (2-year floodplain)."

Disturbance of spawning, incubating, or rearing endangered fish species would likely be considered a "take" under the Endangered Species Act. It could be a significant adverse impact (i.e., "may affect, and is likely to adversely affect" determination) if not properly mitigated. If mitigated, it would probably decrease to a "may affect, but is not likely to adversely affect" BA determination.

Fish – Biosolids Disposal

Correspondence with the Oregon Natural Heritage Program (ONHP), U.S. Fish and Wildlife Service (FWS), and National Marine Fisheries Service (NMFS) was initiated to identify any federally listed, proposed, or candidate threatened or endangered species that may potentially occur in the project area. Agency correspondence indicated that the southern Oregon/northern California coastal population of coho salmon *(Oncorhynchus kisutch)*, a federal threatened species; the Klamath Mountains province population of steelhead *(O. mykiss)*, a federal candidate species; and the southern Oregon coast fall-run population of chinook salmon *(O. tshawytscha)*, a federal proposed threatened species, may occur in the proposed action area. If required at a later time, potential impacts to listed species identified in the agency correspondence would be addressed in a Biological Assessment (BA), which would be conducted after a biosolids handling and disposal alternative is chosen and final design is underway. At present, no BA is required for the biosolids handling and disposal portions of the project.

The Preferred Alternative includes a new co-composting facility to be constructed off-site at the Merlin Landfill for biosolids disposal. The wet weather storage and co-composting of biosolids from the Grants Pass WRP would continue the beneficial reuse of biosolids and reduce the reliance on landfilling. In general, the co-composting of biosolids should benefit fish species and their habitat in the Rogue River Basin.

At the Merlin Landfill, there are two surface water drainages near the proposed site: 1) an ephemeral stream (with two stormwater detention ponds) which flows in the winter and spring across the west side of the landfill, and 2) Louse Creek, a perennial stream that bisects the valley approximately 1,700 feet north of the landfill. There is no fish use of the ephemeral stream, although it empties to Louse Creek. The reach of Louse Creek downstream of the site does not appear to contain much suitable salmonid rearing habitat, but is believed to be used by summer and winter steelhead and coho salmon.

The co-composting facility, specifically the co-composting pads and buildings, would create additional impervious surface area which, if untreated for stormwater discharge, could result in increased peak run-off and erosion from the site. This additional impervious surface would consist of 2,200 sf for the access road and utilities, 10,200 sf for an asphalt-paved maneuvering area, 33,000 sf for the concrete co-composting pad and roof, and 10,000 sf for the canopy roof and concrete pad. However, stormwater drainage and sediment from the proposed site are planned to be diverted into the existing stormwater detention ponds for treatment. No impacts to fish resources in Louse Creek are expected from stormwater discharge or sediment from the site, assuming a suitable stormwater drainage plan is developed and implemented.

Groundwater infiltration from the Merlin Landfill into Louse Creek has been thoroughly studied and documented as part of the Ecological Screening Assessment Report prepared by EMCON (1999). Due to the effects of natural attenuation and the groundwater treatment system, coho salmon in Louse Creek are not expected to be exposed to contaminants in the groundwater leachate from the Merlin Landfill at concentrations sufficiently high to pose potential adverse effects. Likewise, no risk to fish from any groundwater leachate from the proposed biosolids co-co-composting facility is expected.

In the Second Preferred Alternative, disposal of biosolids in an already permitted landfill facility would have no adverse affect on plant species. However, impacts to resources would be greater than under the Preferred Alternative, since land application of biosolids potentially increases the amount of nutrients entering the Rogue River basin. The land application of biosolids could have a significant impact on fish and wildlife if the run-off contained pathogens, heavy metals, nitrogen, phosphorous, and ammonia.

Wildlife – Grants Pass Water Restoration Plant
Correspondence with the Oregon Natural Heritage Program (ONHP), U.S. Fish and Wildlife Service (FWS), and National Marine Fisheries Service (NMFS) was initiated to identify any federally listed, proposed, or candidate threatened or endangered species that may potentially occur in the project area. Agency correspondence indicated that bald eagles *(Haliaeetus leucocephalus)*, a threatened species, may occur in the vicinity. If required at a later time, potential impacts to listed species identified in the agency correspondence would be addressed in a Biological Assessment (BA), which would be conducted after a Grants Pass WRP alternative is chosen and a final design is underway. At present, no BA is required for treatment portions of the project.

A FWS representative, John Thiebes, was contacted about the potential of nesting sites and critical fish and wildlife habitat within the project area. He reviewed available FWS maps and did not identify any nesting sites or critical fish and wildlife habitat.

Potential impacts to bald eagles from both the Preferred Alternative and the Second Preferred Alternative are essentially the same. Both alternatives include upgrades to the Grants Pass WRP to increase handling capacity, and should enable the plant to meet basin-wide discharge standards for biochemical oxygen demand (BOD) and suspended solids (SS). All construction necessary for the upgrade would occur within the present boundary of the treatment plant, with the exception of constructing a new outfall on the Rogue River shoreline.

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Agency correspondence does not indicate nesting, or roosting sites for bald eagles within 2 miles of the project area; however, they may use the area while foraging along the Rogue River. They frequently use prominent perches that provide open views, and prefer snags along shorelines and riverbanks (Stalmaster and Newman 1979). Several prominent trees are present along the Rogue River shoreline in the vicinity of the treatment plant, and could potentially be used by foraging eagles.

Potential project impacts include disturbance of eagles, or their prey, during the construction phase of the project. However, these potential impacts are considered insignificant for the following reasons: 1) the majority of the construction would be away from the shoreline and limited to the present facility where there is already a large amount of human activity, and 2) prey species in the vicinity are just as likely to move to other locations where they would still be readily available to foraging eagles. In addition, the area surrounding the Grants Pass WRP has a high level of human activity, which includes a sports park, fairgrounds, and medium income residences. It is likely that the construction would result in a beneficial effect to bald eagles, since the project would result in an improvement of the habitat for eagle prey by improving water quality.

Based on preliminary review by Parametrix biologists, impacts to listed wildlife species are not anticipated.

Wildlife – Biosolids Disposal

Correspondence with the Oregon Natural Heritage Program (ONHP), U.S. Fish and Wildlife Service (FWS), and National Marine Fisheries Service (NMFS) was initiated to identify any federally listed, proposed, or candidate threatened or endangered species that may potentially occur in the project area. Agency correspondence indicated that bald eagles *(Haliaeetus leucocephalus)*, a threatened species, may occur in the vicinity of the co-composting facility. If required at a later time, potential impacts to listed species identified in the agency correspondence would be addressed in a Biological Assessment (BA).

The Preferred Alternative includes a new co-composting facility to be constructed off-site at the Merlin Landfill for biosolids disposal. The wet weather storage and co-composting of biosolids from the Grants Pass WRP would continue the beneficial reuse of biosolids and reduce the reliance on landfilling. In general, the co-composting of biosolids should benefit fish and wildlife species and their habitat in the Rogue River Basin.

Agency correspondence does not indicate nesting, or roosting sites for bald eagles, within 2 miles of the project area; however, they may use the area while foraging. Bald eagles are more commonly found along shorelines of rivers and large bodies of water where they forage for their typical prey of waterfowl and fish (Stalmaster and Newman 1979). The landfill site is not located near any water features that are considered suitable foraging habitat for foraging eagles. Louse Creek is approximately 0.25 miles north of the site, but is a relatively low-flow creek with a maximum flow of 54.2 cubic feet per second and is not considered suitable foraging habitat for eagles (Emcon 1999).

In addition, the proposed site does not provide any habitat preferred by bald eagles. Dominant overstory species at the site are Pacific madrone *(Arbutus menziesii)* and Oregon white oak (Quercus garryana), which provide a relatively low canopy compared to Douglas-fir-dominated stands that are preferred by bald eagles (Anthony and Isaacs 1989). Since the site does not offer any suitable habitat for bald eagles, it is unlikely they would use the area and, therefore, they would not be impacted by construction or use of the facility.

In the Second Preferred Alternative, disposal of biosolids in an already permitted landfill facility would have no adverse affect on plant species. However, impacts to resources would be greater than under the Preferred Alternative, since land application of biosolids potentially increases the amount of nutrients entering the Rogue River basin. The land application of biosolids could have a significant impact on fish and wildlife if the run-off contained pathogens, heavy metals, nitrogen, phosphorous, and ammonia.

Plant Species – Grants Pass Water Restoration Plant
According to the OHNP and FWS, there were no documented occurrences of a listed plant species within the Grants Pass WRP property; therefore, there are no adverse impacts resulting from the Preferred Alternative and the Second Preferred Alternative.

Plant Species – Biosolids Disposal
The Preferred Alternative may impact a federally listed plant species. The ONHP and FWS identified one documented occurrence of a listed plant species within the Merlin Landfill site. The Gentner's Fritillaria *(Fritillaria Gentneri)* plant species is a federally listed endangered plant, and two were identified in May 1982 within a one-mile radius of the proposed project. The 5-acre portion of the Merlin Landfill site is relatively undisturbed mature oak and madrone forest, which is a potentially suitable habitat for the Gentner's Fritillaria species. A threatened or endangered species survey to identify the potential presence of this species is recommended. A survey would need to be completed for this species during the flowering period, which is from April 15 through June 15. If identified, one mitigation measure would be to relocate the plant species.

In the Second Preferred Alternative, disposal of biosolids in an already permitted landfill facility would have no adverse affect on plant species. However, impacts to resources would be greater than under the Preferred Alternative, since land application of biosolids potentially increases the amount of nutrients entering the Rogue River basin. The land application of biosolids could have a significant impact on plant species if the run-off contained pathogens, heavy metals, nitrogen, phosphorous, and ammonia.

Other Unique or Sensitive Environmental Resources

No other unique or sensitive environmental resources were identified for either alternative.

Construction Techniques, Best Management Practices, and Mitigation of Selected Natural Resource Concerns

In-Water Construction BMPs

To best avoid direct impacts to spawning, incubating, and rearing salmonids, all in-water construction for the new outfall should be performed during the ODFW preferred in-water work period for this section of the Rogue River (June 15 to August 31). Fish passage should be provided for both adult and juvenile forms of all salmonid species throughout construction and placement of the outfall, per ORS 498.268 and 509.605.

Because it is toxic to fish, any fresh concrete that is in the plastic state used for outfall construction should be contained within tight forms or otherwise isolated from the Rogue River for 24 hours or more. Anchoring of the outfall to the streambed should be conducted in a manner that causes the least disturbance of the streambed in terms of total material removed and amount of stream sediment dislodged into the water column. Any turbidity caused by outfall construction should not exceed DEQ water quality standards for the Rogue River. Any riprap used should be placed "in the dry" wherever possible, or from the top of the bank. Riprap should be clean and non-erodible upland rock. If feasible, the use of a temporary work platform on the bank would help to minimize stream bank, channel, and riparian zone construction impacts.

As an avoidance measure, if feasible, no heavy equipment should be permitted within the active channel of the river. However, it is recognized that the placement and anchoring of the outfall would likely require the use of heavy equipment in the channel. A staging area for heavy equipment, petroleum products and hydraulic fluids, and construction materials should be designated in an upland site out of the existing 100-year floodplain. All heavy equipment that is used for in-water work should be cleaned of external grease, oil, and dirt before entering the water.

The contractor should develop an appropriate Spill Prevention and Countermeasure or Pollution Control Plan (PCP).

Erosion Control BMPs

An Erosion Control Plan (ECP) should be prepared and implemented by the contractor, which ensures that the project does not exceed DEQ turbidity standards during outfall construction. It is assumed that the project will require a National Pollution Discharge Elimination System (NPDES) Storm Discharge Permit that would include erosion control, stormwater discharge, and construction practices. On the bank, erosion control seeding and mulching, and placement of erosion control blankets and jute mats should be completed; and disturbed areas stabilized as soon as possible following exposure, but not later than 7 days after exposure.

The ECP should provide for periodic inspection and maintenance of temporary erosion control devices.

Fisheries Mitigation
Since the Rogue River is spawning habitat limited for salmonid species, potential fisheries mitigation efforts may include the strategic placement of instream structures, such as boulders in areas where pool habitat is lacking. This would promote the recruitment of spawning cobbles and gravels. Migration efforts would require coordination with ODFW biologists as to the timing, location, and type of instream mitigation. However, upstream of the outfall might be a suitable location.

Alternatively, the project budget may include a specific line item for future fisheries mitigation efforts to be identified by ODFW biologists, including locations out of the main stem Rogue River but within the Rogue River basin.

Tree/Vegetation Removal Mitigation
There are currently no tree ordinances or replacement requirements for Josephine County (telephone conversation with Josephine County Planning Department representative, Dale Kellenbeck, January 31, 2000). The co-composting facility is located outside of the City of Grants Pass city limits and the Urban Growth Boundary; therefore, the City of Grants Pass Development Codes are not applicable.

A discussion with FWS representative, John Thiebes, indicated they would not require tree replacement because the project area is not within 50 feet of a fish bearing stream and is not considered critical fish and wildlife habitat (telephone conversation with FWS representative, John Thiebes, January 21, 2000.)

Public Involvement

Project Press Release and District-wide Project Update
A project press release was sent to the local newspaper and radio stations, and a District-wide project update was sent to ratepayers in the District. The purpose of the press release/newsletter was to announce a public meeting on February 3, 2000, to discuss the amended plan and the future of the Grants Pass WRP.

Newspaper Advertisement
An advertisement was placed in the *Daily Courier* announcing the February 3, 2000, meeting's purpose, location, date, and time.

Public Information Meeting on Facilities Plan Update
A public meeting was held on Thursday, February 3, 2000, regarding the Grants Pass Water Restoration Plant Facilities Plan Update. A newsletter was sent to all ratepayers (approximately 9,000). In addition, a news announcement appeared in the *Daily Courier* on Wednesday, February 2, 2000, and two local radio stations announced the meeting.

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Six people attended the public meeting. A questionnaire was distributed among the attendees for feedback. Two surveys were returned, and they are located in Appendix I. Primary concerns and questions voiced by the public included the following issues:

- Citizens were concerned with justification for the costs and if rates would rise as a result of the upgrades.

- One citizen asked if a green waste transfer station had been considered. The citizen was concerned that the new co-composting facility would not receive an adequate volume of green waste due to its location. The individual suggested including a transfer station in the costs of the upgrade.

- One citizen lives close to the plant and mentioned a concern of odor from the plant.

- Several citizens commented that they were not given adequate notice to attend the meeting.

To respond to citizen concerns regarding the lack of adequate notice regarding the meeting, the City prepared a mailer apologizing for the short meeting notice.

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Anthony, R.G. and F.B. Isaacs. 1989. Characteristics of Bald Eagle Nest Sites in Oregon. Journal of Wildlife Management 53:148-159. Oregon Natural Heritage Program, Correspondence from Terry Campos to Michele Eccleston, December 22, 1999.

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Stalmaster, M.V. and J.R. Newman. 1979. *Perch-site Preferences of Wintering Bald Eagles in Northwest Washington*. J. of Wildlife Manage. 43:221-224.

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CHAPTER 12

FINANCING PLAN

Various funding alternatives exist for the City of Grants Pass to implement the Capital Improvement Plan (CIP). The purpose of this section is to determine the best option for financing.

As part of the work in developing a financing plan, a computer model was created to assist the City in future modifications to both the CIP and operation/maintenance funding. This model is the basis for the conclusions presented herein.

Capital Costs

The CIP includes three major elements: Treatment Plant (liquid treatment), Biosolids, and Collection System. The proposed phasing plan is shown in Table 12-1.

Table 12-1. Capital Improvement Plan
(Year 2000 dollars)

	Year 2001-2004	Year 2005-2006	Year 2010-2011
Treatment Plant	$5,940,000	$5,795,000	$1,855,000
Biosolids	$3,590,000		
Collection System	$2,500,000		
TOTAL	$12,030,000	$5,795,000	$1,855,000

In addition to capital costs, the City must be able to fund the ongoing cost of operating and adequately maintaining its sewer utility. A breakdown of these operation and maintenance costs is shown in Table 12-2.

Table 12-2. Operation and Maintenance Expenses
(Year 2000 dollars)

Wastewater Collection Services	$333,488
Wastewater Treatment Services	$887,513
Customer Services	$167,910
General Program Operations	$361,393
TOTAL	$1,750,304

Current Funding

The City of Grants Pass has the following rate and fee structure:

- Monthly User Charge ($)
- System Development Charge (SDC)

The City of Grants Pass has two main revenue sources: monthly user charges and system development charges. The monthly user charge is collected from all system customers. In addition, those outside the city limits pay an additional surcharge.

All new connections to the system pay a system development charge. This charge is designed so new customers pay their share of wastewater collection and treatment costs. Currently the SDC averages $1,000 per new connection.

Capital Funding Mechanisms

To fund the CIP, the City will need to consider other funding mechanisms. These other funding mechanisms include the following:

- Revenue Bonds
- Low interest loans - State Revolving Fund (SRF)
- Grants

A revenue bond is a very common tool to fund capital improvements. Rates are determined by market conditions and currently are around 5 to 6 percent. Due to the significant financing cost and associated coverage, this funding method is normally a last resort if other funds are not available.

Grants used to be the only way to finance wastewater treatment improvements. Many of the upgrades to secondary treatment across the United States were funded by grants, sometimes up to 75 percent of the cost. Wastewater construction grants, while still available, are insufficient compared with the current demand.

One of the best governmental programs available is the State Revolving Fund loan program. This fund, seeded by money from the Environmental Protection Agency (EPA), is giving state governments, including Oregon, funds to loan to municipalities for treatment and collection system improvements. These loans, administered by the Oregon Department of Environmental Quality Wastewater Finance Office, are "low interest" with current rates at less than 4 percent. Unlike bonds, normally these loans have a smaller reserve amount and no coverage requirements. Due to federal funding, cities like Grants Pass seeking SRF funding must comply with EPA requirements for a Facility Plan, thus this plan has been developed according to said requirements.

It is projected that the City of Grants Pass will secure $4M in SRF loans in the 2001-2002 fiscal years. Based on the cash flow described below, this money will allow the City to proceed with Phase I of the CIP.

Projected Cash Flow

The major components of both revenue and expenses in a simplified view of the wastewater utility are:

- Revenue: Monthly Sewer Service Charges, SDCs, Loan Proceeds, and Interest
- Expenses: Operation and Maintenance, Capital Expenditures, Debt Service, and Reserve Amounts

In reality, these revenues and expenses are tracked in separate accounts; however, for the purposes of this simplified analysis, all revenue and expenses will be considered as one account.

Table 12-3 projects the annual cash flow for the Grants Pass wastewater utility for the period of the CIP. With the funding from the SRF, and the current cash on hand (based on the City's construction fund only), it appears that the City will be able to fully fund the CIP. In addition, the City may be able to fund an ongoing sewer rehabilitation program. This amount, shown in the cash flow during non-CIP funded years, is $250,000. It will have to be determined if this amount is sufficient to address the long-term rehabilitation needs of the City.

Table 12-3. Simplified Cash Flow Analysis

	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011
Beginning Balance	$6,800,000	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	$1,401,552	$931,076
Rate Revenue	$2,650,000	$2,689,750	$2,730,096	$2,771,048	$2,812,613	$2,854,803	$2,897,625	$2,941,089	$2,985,205	$3,029,983	$3,075,433	$3,121,565
SDC Revenue		$100,000	$101,500	$103,023	$104,568	$106,136	$107,728	$109,344	$110,984	$112,649	$114,339	$116,054
O&M Cost	$1,750,000	$1,802,500	$1,856,575	$1,912,272	$1,969,640	$2,028,730	$2,089,592	$2,152,279	$2,216,848	$2,283,353	$2,351,854	$2,422,409
Loan Funding		$2,000,000	$2,000,000			$2,000,000						
Debt Service		$145,435	$290,871	$290,871	$290,871	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306
Reserve Account		$150,000	$150,000			$150,000						
Capital Cost		$6,195,400	$6,195,400	$100,000*	$100,000*	$2,547,000	$2,547,000	$100,000*	$100,000*	$100,000*	$921,000	$921,000
Investment Income		$290,000	$243,728	$111,106	$56,396	$82,298	$92,184	$50,301	$19,036	$34,521	$48,912	$46,653
Ending Balance	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	$1,401,552	$931,076	$435,633

* Balances reflect both Capital and Operations Accounts

Conclusion

The financial situation of the City's wastewater utility is good. Given the ability to secure loans from the SRF program, and the sound fiscal management of the utility, the City will be able to fully fund the CIP as outlined in this Facilities Plan.

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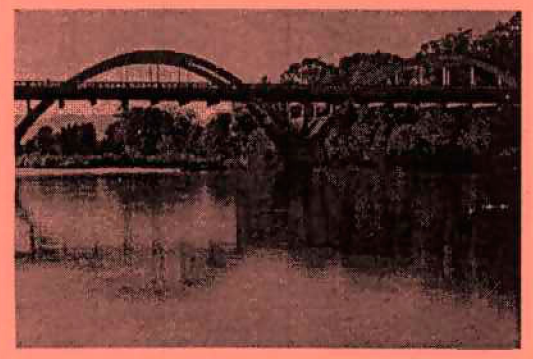

Collection System Master Plan

Prepared for:

City of Grants Pass

EXHIBIT 8 to UAPC Staff Report 6-7-09

Parametrix

September 2004

City of Grants Pass
Collection System
Master Plan

Prepared for

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September 2004
Project No. 276-3416-025 (03/0350)

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CERTIFICATE OF ENGINEER

The technical material and data contained in this document were prepared under the supervision and direction of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.



Prepared by Steven C. Gilbert, P.E.

Checked by Mike Sailor, P.E.

Approved by Ken Black, P.E.

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C Sanitary Sewer Lateral Replacement Policy
D Pump Station Drawings
E GIS Database Used in Maintenance and Reliability Analysis
F Detailed Structural and Maintenance Analysis

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KEY TERMS

cfs	cubic feet per second
CIP	Capital Improvement Plan
City	City of Grants Pass
CMOM	Capacity, Management, Operation and Maintenance
CWA	Clean Water Act
EPA	Environmental Protection Agency
fps	feet per second
GIS	geographic information system
gpm	gallons per minute
I/I	infiltration and inflow
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
O&M	operation and maintenance
OAR	Oregon Administrative Rules
ODEQ	Oregon Department of Environmental Quality
Plan	Collection System Master Plan
RSSSD	Redwood Sanitary Sewer Service District
SDWA	Safe Drinking Water Act
SSOs	sanitary sewer overflows
UGB	urban growth boundary
USDA	United States Department of Agriculture
USGS	United States Geological Survey
WRP	water restoration plant

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EXECUTIVE SUMMARY

INTRODUCTION

The present and future needs of wastewater collection systems are assessed through the preparation of a Collection System Master Plan. The plan evaluates the existing system and the operation and maintenance of the system. Using growth forecasts for the service area, the plan identifies system deficiencies and improvements to address the needs within a designated planning period.

The last Collection System Master Plan completed by the City of Grants Pass (City) was in 1983. Since that time, the City has experienced continued growth and has expanded the City's collection system to include the Redwood Sanitary Sewer Service District (RSSSD) and its collection system west of the city. A new plan is warranted, and the Oregon Department of Environmental Quality also requires that a new plan be completed by the City.

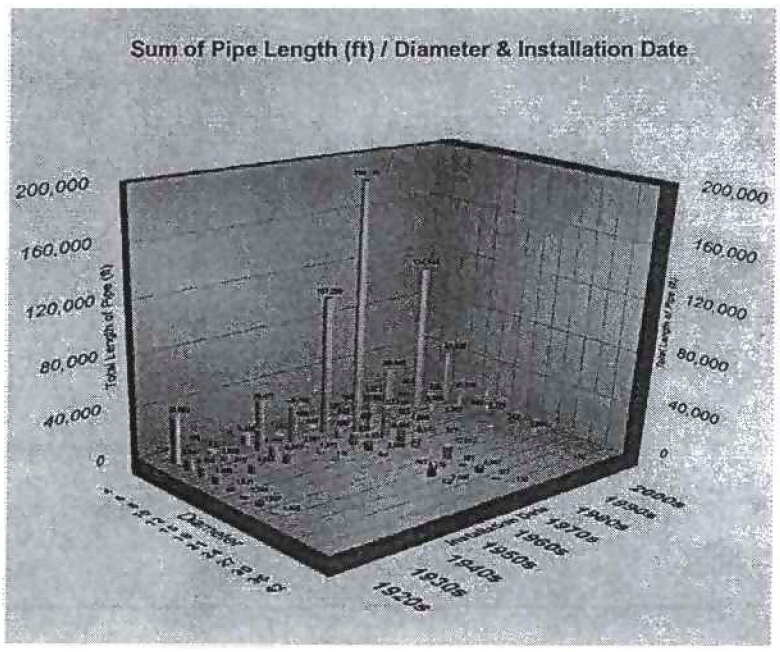
SERVICE AREA

The service area covered in this plan follows the Urban Growth Boundary (UGB) defined in the 1982 *Grants Pass Comprehensive Plan for Community Development*, which includes the city and the unincorporated Harbeck-Fruitdale area. The service area also includes the RSSSD area.

The Grants Pass area has experienced steady population growth since the 1920s. Most recently, the area has grown at about 1.5 percent annually. Current population within the city limits in 2003 is estimated at 22,444. Adding the Harbeck-Fruitdale area and the RSSSD area yields a total existing service area population of 32,778 in 2003. Also, commercial/industrial flows need to be included, and this component is assumed to be equivalent to 35 percent of the total residential service population.

EXISTING COLLECTION SYSTEM

The existing collection system is comprised of 23 separate drainage basins and a total of about 0.8 million feet of sewer pipelines ranging in size from 4 inches to 42 inches in diameter as shown in the figure to the right. Approximately one-half of these sewers are at least 30 years old, with most of the old pipes clustered in the downtown area. From the late 1920s through the mid-1960s, most sewers were made locally. These concrete pipelines were constructed from substandard materials and have poor long-term performance.



There are numerous diversion structures in the existing system that shunt flow from over-capacity pipelines to adjacent pipelines. These diversions were installed to

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prevent sewer overflows during peak rain events. Even with these diversions, however, the system has had 10 overflows since 1997, with the most recent in December 2000. The most common cause of these overflows was pipe obstructions.

Since 1986, the City has been compiling sewer condition and maintenance data into a central electronic database. These data offer a historical record of a number of structural conditions and maintenance issues for individual pipeline segments. These data have been used in a maintenance and reliability analysis of the existing collection system.

There are three existing lift/pump stations in the collection system. These include the Webster No. 1 and No. 2 lift stations and the Bridge Street pump station. In addition, a future lift station will likely be required at the east edge of the Urban Growth Boundary (UGB) and the Rogue River. There are also two new pump stations in the Redwood area, the Redwood and Darneille Pump Stations.

HYDRAULIC ANALYSIS

To evaluate the ability of the existing collection system to meet future capacity needs, a hydraulic analysis was conducted. Using HYDRA, a computer model of the major sewers serving the service area was developed and calibrated. To calibrate the model, a 2-month flow monitoring program was conducted in December 2002 to February 2003. Six flow monitors were installed at six manholes that best represent flow from major drainage basins with known deficiencies and anticipated growth. Collected flow and rainfall data were used in the calibration process.

The hydraulic model of the collection system was then modified by increasing service area population to the year 2025, a 20-year projection, and year 2060, a projection assuming all available land in the service area was at build-out condition. The year 2060 projection was used to size future collection system pipelines. The total equivalent population in the service area in 2025 was about 65,000 and about 130,000 in 2060.

In all, six models were created for the analysis; three models were based on populations from the years 2003, 2025, and 2060, with the existing diversions retained, and three corresponding models with diversions removed. Using a threshold of 2 feet of allowable surcharge for any pipeline during a 5-year, 24-hour rainfall, collection system hydraulic deficiencies were identified.

MAINTENANCE AND RELIABILITY ANALYSIS

To evaluate the structural condition of the existing collection system and identify future repair/replacement needs, a maintenance and reliability analysis was conducted. Structural deficiencies were identified using available sewer condition and maintenance data and linking these data to a graphical representation of the collection system. This analysis included extensive discussions with the City's sewer maintenance staff to confirm known areas that require routine maintenance and areas where other structural deficiencies had been observed.

In addition, a condition assessment of all lift and pump stations was conducted. No needed improvements to these stations were identified.

RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS/CAPITAL IMPROVEMENT PLAN

Based on the collection system hydraulic analysis conducted, five capital improvement projects were identified that need to be completed in the next 20 years to maintain adequate conveyance system capacity. These five projects are shown in Figure ES-1.

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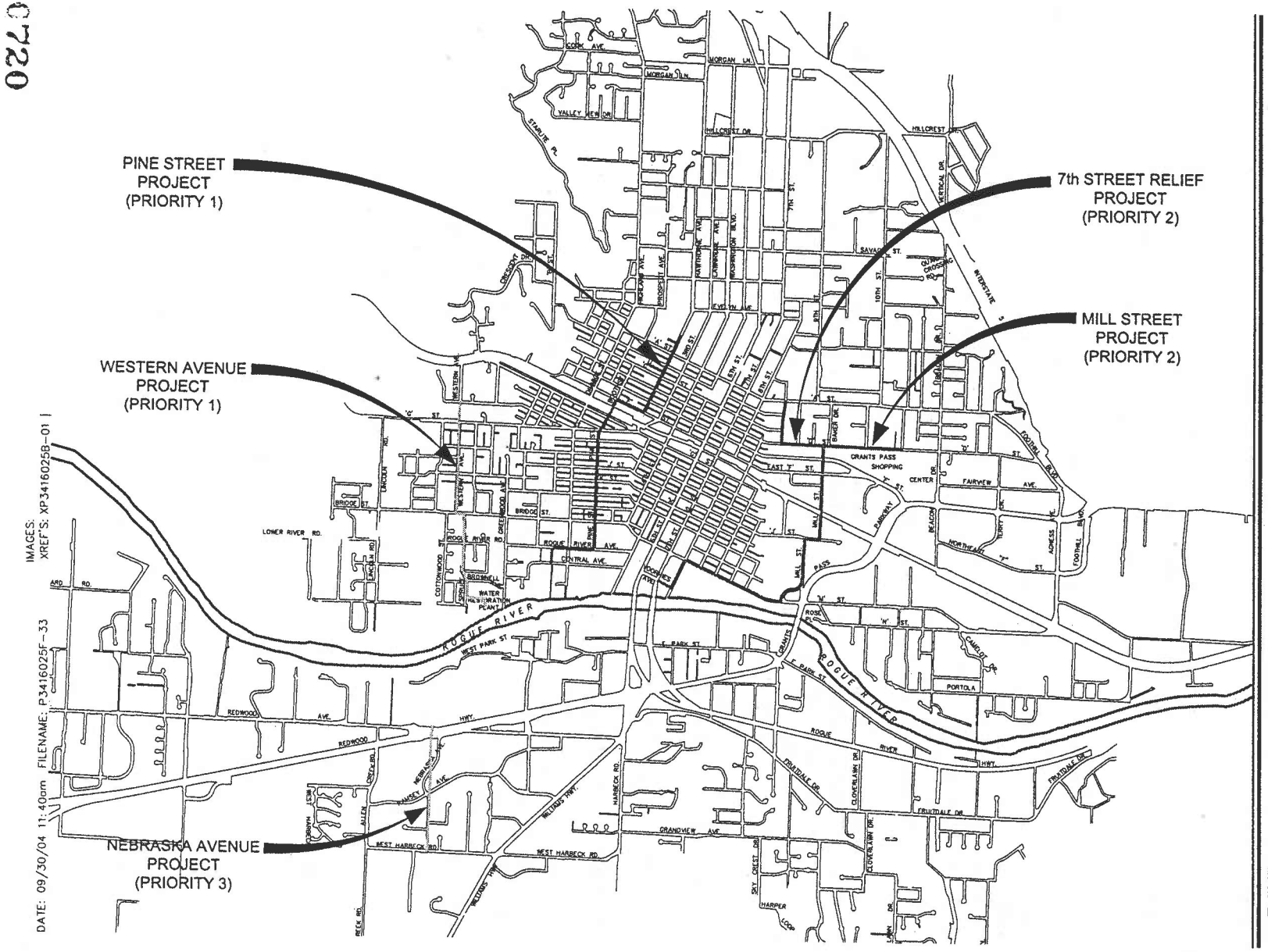

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CITY OF GRANTS PASS
COLLECTION SYSTEM
MASTER PLAN

Parametrix

Figure ES-1
Key Map of Proposed
Pipeline Improvement Projects

The Pine Street sewer has already experienced surcharging during rainfall events and needs to be replaced with a larger diameter pipeline as soon as possible. The Western Avenue sewer also has experienced surcharging and similarly needs to be a larger diameter pipe to handle future flows on the west side of the UGB due to growth. The proposed new Mill Street sewer is needed to provide conveyance capacity for growth in the Mill Street area and also to avoid having to improve the existing dual siphons crossing the Rogue River and the existing sewer paralleling the Rogue River on the south side. The Mill Street sewer will divert flow to available conveyance capacity on the north side of the river.

The 7th Street relief sewer is needed to divert flow away from the existing 7th Street sewer and avoid having to replace this pipeline with a larger diameter pipe. This project diverts flow to the future new, larger-diameter Mill Street sewer. In the future, the Nebraska Avenue sewer needs to be replaced with a larger-diameter pipeline to accommodate growth south of the Redwood Highway.

Based on the maintenance and reliability analysis conducted on the existing collection system, six structural repair areas were identified and are shown in Figure ES-2. In each of these areas, there are numerous very old small-diameter (6-inch) pipelines that exhibit extensive structural defects and severe root encroachment. Numerous services calls have been made on these pipelines and the pipelines require very frequent, difficult cleaning. Rather than attempt to identify individual pipelines within each of these areas that require repair/replacement, these areas first need more detailed investigations and inspections, after which specific repair/replacement programs can then be identified and implemented. Steps to be followed would include further testing and inspections, evaluation of alternative repair/replacement technologies and strategies, and cost evaluations. Upon completion of these steps, a final repair/replacement program for each area can then be developed and implemented.

CAPITAL IMPROVEMENT PLAN

Using appropriate unit prices for pipeline construction and other project cost components, the total project cost for both the five pipeline improvement projects (Figure ES-1) and the six structural repair areas (Figure ES-2) have been estimated. For the six structural repair areas, the cost was based on complete replacement of one-half of the existing sewers with a minimum pipe diameter of 8 inches per standard sewer design criteria. In addition, using the prioritization of the improvements identified in the hydraulic and maintenance and reliability analysis, a recommended Capital Improvements Plan has been developed and is shown in Table ES-1.

Table ES-1. Recommended Capital Improvement Plan Grants Pass Collection System

Project	Schedule	Cost ($1M)
Pine Street Sewer	2004–2006	$2.55
Western Avenue Sewer	2006–2009	$1.46
Pine Street Structural Repair	2009–2011	$1.11
5th Street Structural Repair	2010–2012	$2.13
7th Street Structural Repair	2013–2015	$1.46
Lawnridge-Washington Structural Repair		$1.64
Mill Street Sewer	2016–2018	$3.08
Subbasin B/C Structural Repair		$1.23
7th Street Relief System	2019–2021	$1.40
Subbasin H Structural Repair		$1.21
Nebraska Avenue Sewer	2022–2024	$0.81
Total Collection System Capital Improvement Plan:		**$18.08**

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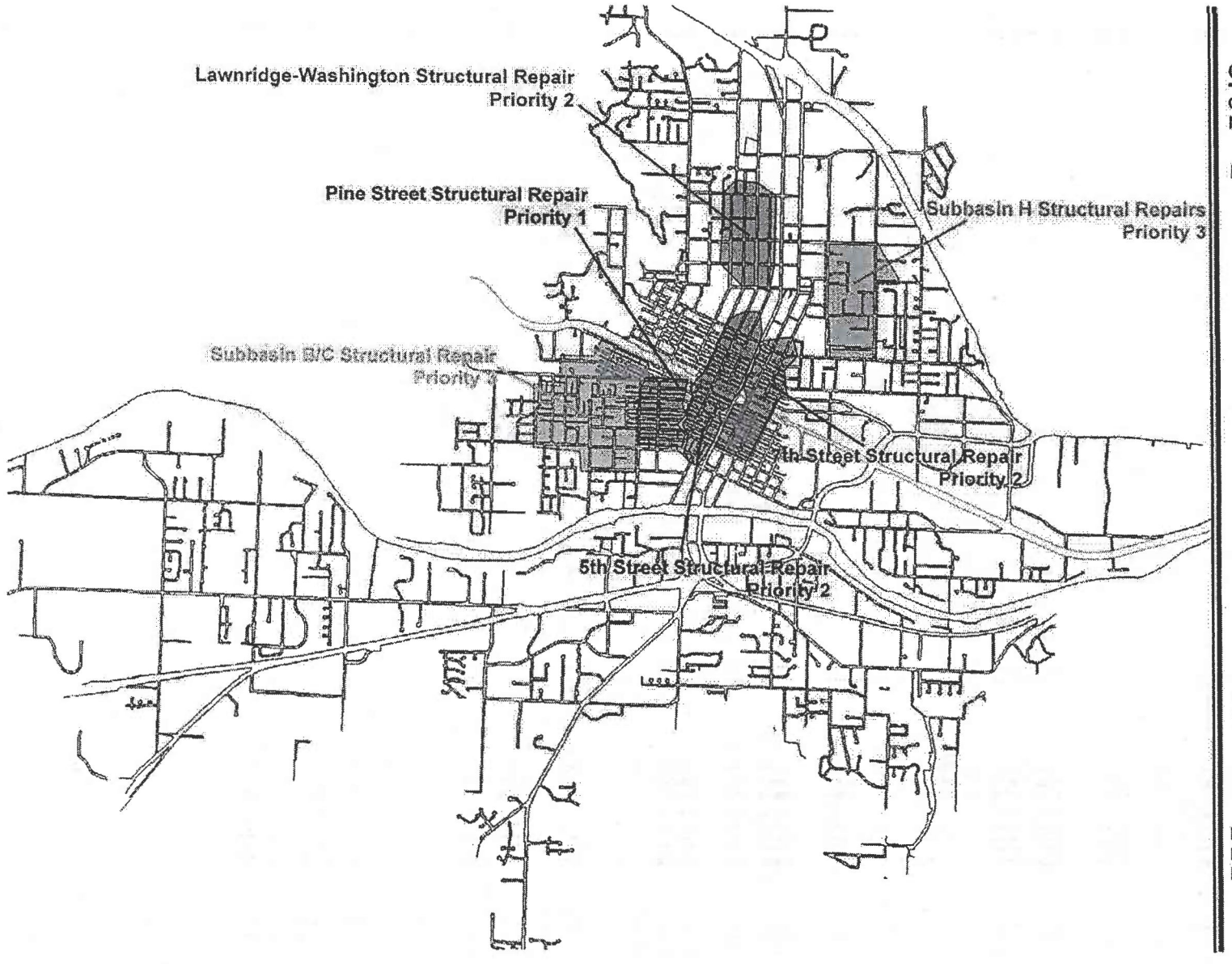
227,000



CITY OF GRANTS PASS

COLLECTION SYSTEM MASTER PLAN

Parametrix

Project Area by Priority

- Pine Street Structural Repair Priority 1
- 6th Street Structural Repair Priority 2
- 7th Street Structural Repair Priority 2
- Lawnridge-Washington Structural Repair Priority 2
- Subbasin B/C Structural Repair Priority 3
- Subbasin H Structural Repairs Priority 3

SCALE IN FEET

0 1,250 2,500

N

Figure ES - 2
Proposed Structural Repair Areas

The City of Grants Pass needs to begin to implement this CIP as soon as possible. The Pine Street sewer project has already begun and construction is planned for 2005-2006 if funding is available. Implementation of further improvements in the CIP is, however, contingent upon funding and the City should begin an evaluation of their sewer customer rates in order to provide funding to implement the remainder of the CIP.

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Introduction

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1. INTRODUCTION

1.1 BACKGROUND

The present and future needs of wastewater collection systems are assessed through preparation of Collection System Master Plans. These plans evaluate the existing system, its operation and maintenance, and growth forecasts for the service area to anticipate potential system deficiencies and prioritize collection system needs within the designated planning period.

The City of Grants Pass (City) last completed a Collection System Master Plan (Plan) in 1983 (James Montgomery, 1983). A number of the recommended collection system improvements in the 1983 Plan have been implemented. The City has, however, experienced sewer line failures and occasional overflows due to sewer line obstructions. The City has also expanded its collection system to include the Redwood Sanitary Sewer Service District (RSSSD) collection system located west of the City. For these reasons, an update of the existing Plan is warranted. This need was recognized by the Oregon Department of Environmental Quality (ODEQ), which included completion of an updated Collection System Master Plan as a provision in the National Pollutant Discharge Elimination System (NPDES) permit for the City's Water Restoration Plant (WRP) as shown in Appendix A.

The updated Plan will be the most recent in a series of recent reports/studies focusing on wastewater infrastructure. In 2000, the City adopted a *Wastewater Facilities Plan Update* (Parametrix, 2000) for improvements to the City WRP. The 2000 *Facilities Plan* updated the information presented in a draft *Water Restoration Plant Facilities Plan* (Brown and Caldwell, 1999). The *Facilities Plan* focuses on the WRP but also provides some analysis of the existing collection system to assess the impact of infiltration and inflow (I/I) on peak flow events at the WRP. Additional collection system analyses that were presented in the RSSSD Facilities Plan prepared in 1999 by Parametrix (Parametrix, 1999), assessed the RSSSD wastewater collection system. Where appropriate, these documents are referenced herein.

1.2 PURPOSE

This Collection System Master Plan is an examination of the existing wastewater collection system of the City of Grants Pass and the needs of the system throughout the 20-year planning period. (A map of the entire wastewater collection system is included in Appendix B.) The Plan is intended to provide the City with a tool for implementing orderly, logical, and cost-effective improvements. The Plan will serve as a guide to the City for anticipating collection system improvements to accommodate new wastewater flow as a result of development and as a guide for creating an efficient framework for improving ongoing operation and maintenance of the collection system. By implementing the recommended improvements in this plan and properly maintaining the collection system, the risk of sewer line failures and the occurrence of system overflows due to sewer line obstructions and/or hydraulic limitations will be reduced.

Service Area Characteristics



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2. SERVICE AREA CHARACTERISTICS

Development of sound, long-range collection system plans for the service area requires consideration of both natural and socioeconomic environmental characteristics. The natural environment, including topography, geology, soils, climate, and water resources, is the primary determinant of development in the service area. Other factors, such as economic activity, population growth, and land use interact with the natural resources of an area and may significantly affect the overall environment as development continues. This section defines the service area and discusses the characteristics of both its physical and economic environment.

2.1 COLLECTION SYSTEM SERVICE AREA

The service area covered in this Plan is consistent with the sewer service area described in the 1983 *Sewage Collection System Master Plan* (James Montgomery, 1983). The service area follows the urban growth boundary (UGB) defined in the *Grants Pass Comprehensive Plan for Community Development* (City of Grants Pass, 1982) and includes the City of Grants Pass and the unincorporated Harbeck-Fruitdale area south of the city. The Harbeck-Fruitdale area is a special service district that contracts with the City for wastewater services.

The service area also includes the RSSSD, which is located roughly west of the downtown core of Grants Pass and on the south side of the Rogue River from the city. The City recently entered into a service agreement with the RSSSD for conveyance and treatment of wastewater generated within the District. Note that portions of the unincorporated Redwood area lie outside of the UGB.

The North Valley, also known as Merlin, lies outside the UGB to the north of the city and has largely been dependent on on-site sewage treatment systems. Although this area may begin contracting with the City for wastewater services, there are no current plans for service expansion. Therefore, consideration of flows from the North Valley area was not addressed in this Plan.

The service area evaluated in this Plan is as shown in Figure 2-1.

2.2 NATURAL ENVIRONMENT

The natural environment includes the topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items in relation to collection system planning and analysis. When possible, the information provided herein has been updated from the *Water Restoration Plant Facilities Plan* (Brown and Caldwell, 1999).

2.2.1 Topography, Geology, and Soils

The topography, geology, and soils of a region can have a significant effect on the design and construction requirements of the collection system. Topography can determine the route and slope of sewer lines, as well as the need for and location of pumping stations. The geology and soil conditions in an area can affect construction costs for pipelines and determine locations for system components.

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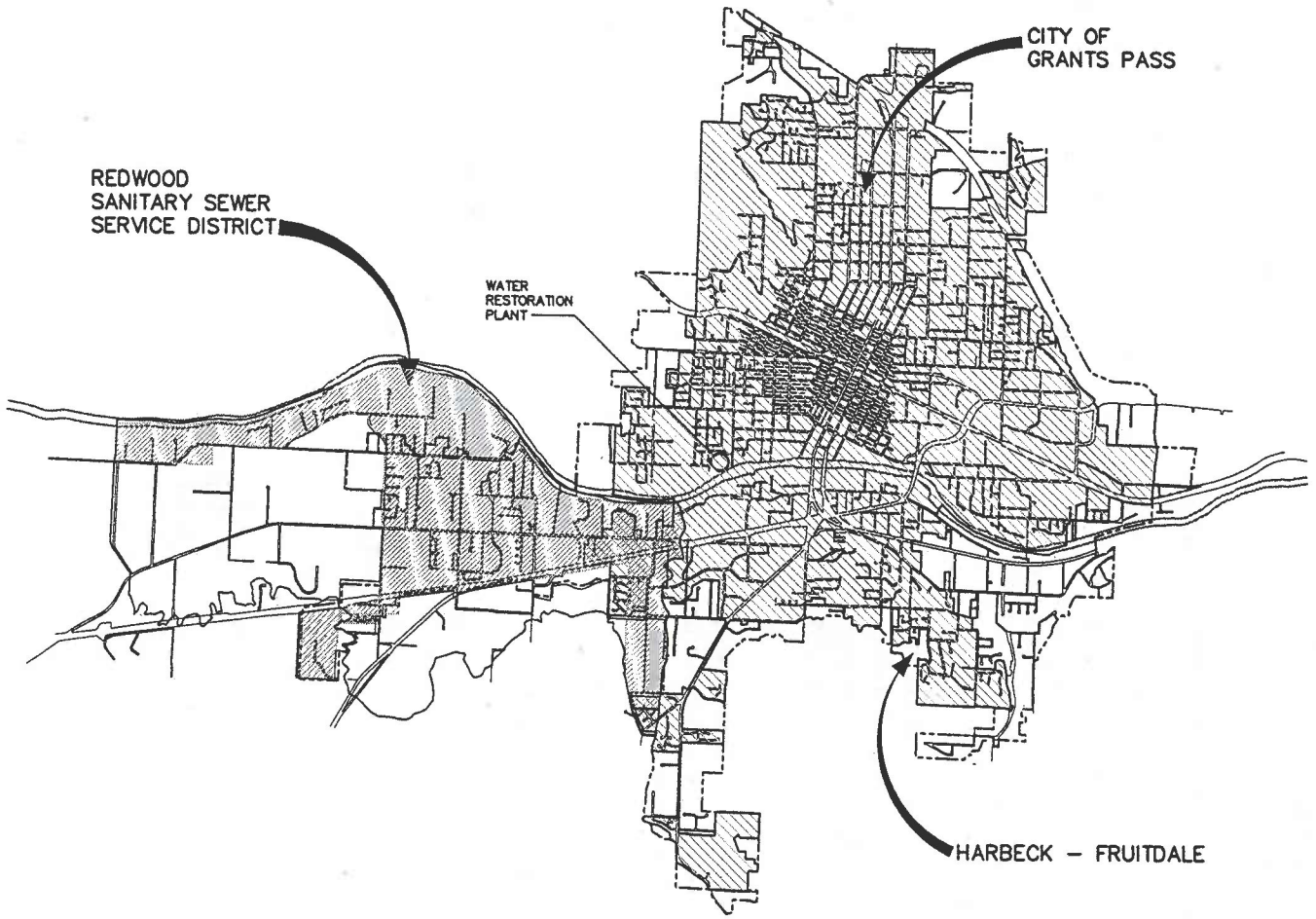

CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

LEGEND

- URBAN GROWTH BOUNDARY
- CITY OF GRANTS PASS
- REDWOOD SANITARY SEWER SERVICE DISTRICT (RSSSD)
- CURRENTLY SERVED PORTION OF RSSSD

SCALE IN FEET
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Figure 2-1
Study Area

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2.2.1.1 Topography

Grants Pass lies in the Rogue River Valley in the Klamath Mountain Range of Oregon. The Rogue River Valley begins at the base of the surrounding hills and exists as a well-defined stream terrace some 10 to 15 feet above the bed of the Rogue River. The valley slopes towards the river at an average gradient of 1 to 2 percent. Elevations on the low-lying valley floor range from 880 to 1,100 feet above sea level. The Rogue River traverses the valley in a general east-west direction on an average slope of about 6 feet per mile.

Away from the valley floor, the terrain grows steep relatively quickly. Beacon Hill (located northeast of the City) and Baldy Mountain (located southeast of the City) are 2,117 feet and 2,740 feet in elevation, respectively. The Siskiyou Mountains, part of the Klamath Mountains, lie to the south and west of Grants Pass. To the northeast, a spur connects the Klamath Mountains to the Cascade Range.

2.2.1.2 Geology

The service area contains several major geologic units, including alluvium deposits, diorites and granites, ultramafic and metavolcanic rocks, and gneisses and schists. The alluvium deposits, between 100 and 150 feet thick, formed the valley floor by eroding away from the surrounding rock units. The lava and metavolcanic rock composing Beacon Hill and Baldy Mountain does not weather easily. Its ruggedness has limited development in these areas. The softer granite of Dollar Mountain to the northwest and various hills to the south and southwest of the city shows greater weathering. The rounded ridges and gentle slopes of these areas have encouraged development.

2.2.1.3 Soils

Weathering of the different geologic units has given the soils of this area a wide range of characteristics. The soils that underlie the developable portions of the Rogue River Valley are of the greatest importance to the collection system analysis. Soils with poor drainage can increase the potential for I/I into the collection system, leading to increased flows to the WRP. A survey conducted by the United States Department of Agriculture (USDA, 1983) identified the soil types found in this area for agricultural purposes. A brief summary of the USDA survey with generalized engineering interpretations is presented herein.

The most important soil types in the valley are Newberg fine sandy loam, Barron coarse sandy loam, and Clawson sandy loam. Newberg fine sandy loam is the principal soil type in the floodplain and terraced areas of the valley. It occupies a strip along the Rogue River that is generally about a mile in width; however, it narrows to about 2,500 feet at Grants Pass. The soil is well drained and presents no major problems for the collection system.

Barron coarse sandy loam occupies extensive portions of the Rogue River Valley and underlies most of the city west of Gilbert Creek. The soil generally occurs upslope from Columbia fine sandy loam and extends as valley fill material into most of the minor tributary valleys. This soil has a slightly higher clay content than the Newberg loam, but does not significantly impact drainage or increase I/I impacts.

Clawson sandy loam underlies a major portion of the city east of Gilbert Creek. This soil typically consists of about 1 foot of smooth-textured silt loam overlying a compact silty loam or clay loam subsoil. At a depth of about 30 inches, the subsoil assumes an extremely gritty texture, reflecting the presence of coarse granitic material. The subsoil terminates at shallow depths in coarse granitic rock. The soil is flat-lying and poorly drained. Because of the impervious nature of the shallow bedrock, it is waterlogged during the winter and spring months. In some areas, the water table is less than 3 feet below ground level

well into the summer. The high groundwater conditions that accompany this soil type can be a problem when wastewater pipelines lying in the soil have cracks or leaks. Groundwater infiltrates into the cracks and leaks, significantly increasing the flow of liquid to the WRP.

2.2.2 Climate

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of wastewater facilities. Rainfall is especially significant, because it can cause large flow increases in the collection system due to stormwater runoff, illicitly connected roof drains, and raised groundwater levels.

2.2.2.1 General Climatic Conditions

The climate of Grants Pass is generally mild, although temperatures below freezing and above 100°F occur for short periods annually. Climate is influenced by the Pacific Ocean, which is located about 60 miles west of the city. The intervening coastal mountains modify the effect of the marine air masses, causing this portion of the Rogue River Valley to receive less annual rainfall and to have fewer cloudy and rainy days than most other portions of Western Oregon. Monthly temperature and precipitation data for Grants Pass are summarized in Table 2-1.

Table 2-1. Climate Summary[a]

Month	Temperature, °F					Precipitation in Inches	
	Mean Maximum	Mean Minimum	Mean	Highest Recorded	Lowest Recorded	Mean	Greatest Daily
January	47.4	31.1	39.3	69	13	5.0	3.4
February	54.1	32.7	43.4	76	12	4.4	2.2
March	59.8	34.1	47.0	81	22	3.7	2.2
April	65.6	35.8	50.7	93	24	2.0	1.4
May	73.1	40.5	56.8	102	26	1.2	1.5
June	80.8	45.4	63.1	106	33	0.5	1.0
July	88.8	49.5	69.2	109	39	0.4	1.3
August	89.0	48.9	69.0	110	36	0.5	0.8
September	82.7	43.0	62.9	108	29	0.9	2.9
October	70.4	37.4	53.9	98	20	2.1	1.7
November	53.3	34.6	44.0	77	12	5.1	4.8
December	45.7	31.3	38.5	67	-1	5.4	4.0
Annual	67.6	38.7	53.2	110	-1	31.2	4.8

[a] Records of the Oregon Climate Service, 1971–2000.

2.2.2.2 Precipitation

Nearly 75 percent of annual rainfall in Grants Pass occurs between the months of November through March. A majority of the annual precipitation is in the form of rain, although about 4 to 5 inches of snow falls each year. Seasonal snowfalls rarely exceed 10 inches, which usually melts immediately.

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2.2.2.3 Temperature

Temperatures in Grants Pass usually remain moderate through the winter. Subfreezing temperatures may persist long enough to freeze water in aboveground facilities; however, rarely last long enough to cause freezing in buried facilities. Summers are warm and dry. Temperatures exceed 100°F an average of 6 days a year. Nighttime summer temperatures are generally cool, averaging about 51°F during July, the warmest month.

2.2.2.4 Other Climatic Factors

Sunshine is usually abundant during the spring, summer, and fall, but the area is generally cloudy during the winter months. Early morning fog occurs frequently during November, December, and January. Fog is less common in October and February and rarely occurs during the rest of the year. Wind speed and direction are not routinely measured at Grants Pass. The prevailing wind direction, however, is from the west, approximately parallel to the axis of the Rogue River Valley.

2.2.3 Water Resources

The principal water resources in the service area are surface water from the Rogue River and its tributaries and groundwater from the alluvium covering the river valley. The water resource most important for this Plan, the Rogue River, drains 2,460 square miles above Grants Pass before traversing the study area. The Rogue River is used for the City's potable water supply, irrigation, and recreation.

A number of individual wells rely on area groundwater. The alluvium is the major aquifer, with a typical yield of 40 gallons per minute (gpm). Volcanic formations usually yield less water, but in a few highly fractured areas wells have yields as high as 60 gpm. Many of the high yields are not sustainable, as the aquifers are small and substantial drawdown occurs.

Both the quantity and quality of water resources are important to wastewater system planning. A receiving stream must have adequate volume and assimilative capacity to accept discharge of treated wastewater effluent. Also, the quality of the water must be examined to ensure that it remains safe for beneficial uses. This section examines water quality and water quantity issues concerning the Rogue River, flood potential, and the Grants Pass Irrigation District as it relates to collection system planning.

2.2.3.1 Water Quality

Oregon Administrative Rules (OAR), Division 41, Section 340-41-365, sets standards for water quality in the Rogue River basin. The rules cover dissolved oxygen concentrations, temperature increases, pH values, coliform counts, creation of tastes or odors, toxic conditions that harm aquatic life or affect drinking water, and the formation of sludges. The regulations relating to wastewater treatment and disposal are discussed in Section 3.2.

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2.2.3.2 *Water Quantity*

The most recent 30 years of Rogue River flow data collected by the United States Geological Survey (USGS) are summarized in Table 2-2.

Table 2-2. Historical Rogue River Stream Flow Data

Month	Monthly Stream Flow Data (1973–2002)[a]		
	Maximum (ft^3/s)	Minimum (ft^3/s)	Mean (ft^3/s)
January	16,610	1,348	5,123
February	10,960	1,162	4,512
March	10,760	1,099	4,364
April	8,395	997	4,076
May	6,428	1,538	3,703
June	4,572	1,016	2,796
July	3,484	974	2,060
August	3,080	878	2,009
September	2,642	1,098	1,724
October	2,282	1,008	1,503
November	9,086	1,160	2,833
December	17,620	1,386	4,866

[a] Data from United States Geological Survey (USGS) Rogue River Monitoring Station (14361500) at Grants Pass, Oregon.

Flows in the Rogue River can fluctuate widely from year to year. The largest recorded discharge, 152,000 cubic feet per second (cfs), occurred during a December 1964 flood. The lowest recorded discharge was a minimum day of 606 cfs during 1968. Reservoirs have since been constructed in the Rogue River basin to provide storage of high wet weather flows for release during dry weather periods.

2.2.3.3 *Flood Potential*

It is necessary to identify flood-prone areas in order to safely locate wastewater collection and treatment system components. The *Grants Pass Comprehensive Plan for Community Development* (City of Grants Pass, 1982) describes flood history in the area, identifies flood-prone areas, evaluates the degree of hazard from flooding, and describes appropriate safeguards for the community.

The City has adopted an ordinance regulating development within the 100-year floodplain. As described in City Ordinance 4471, land within the floodway should not be considered for development. Regulations do allow development in the floodway if the developer can demonstrate that floodwater will not be diverted by a new structure in such a way as to adversely affect adjacent development in the floodway fringe. However, the stringent regulations generally preclude development within urban floodway areas. Development in the floodway fringe is permitted, but the first livable floor must be constructed at least 1 foot above the 100-year flood elevation.

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2.2.3.4 *Grants Pass Irrigation District*

The Grants Pass Irrigation District was formed in the early 1920s to supply irrigation water to land located between the town of Rogue River and the confluence of the Applegate and Rogue Rivers. Currently, about 7,700 acres of agricultural and residential lands are irrigated. The district has water rights to divert up to 150 cfs from the Rogue River during irrigation season. A system of unlined canals carries water from the Savage Rapids Dam throughout the area covered by the District. Many residents in the service area use water from the canals to irrigate landscaping and gardens. The canals are also used to carry stormwater away from these lands.

The irrigation season typically extends between April 15 and October 1. Flow at the City WRP increases during those months, even though little rain typically falls during that period. It appears that one source of groundwater inflow conveyed to the WRP is irrigation water that has seeped into the ground through the unlined canals and infiltrated the collection system. Although there are currently discussions about removal of the Savage Rapids Dam, this Plan assumes that the Grants Pass Irrigation District will continue to function as it does now. Even if the dam were removed, pumps would likely be installed to deliver water to irrigation district canals.

2.3 SOCIOECONOMIC ENVIRONMENT

The socioeconomic environment, primarily population and land use, of an area can profoundly affect wastewater collection system and treatment facility planning. Residential, commercial, and industrial development patterns are determining factors in the location, design, and cost of pipelines, while land use and recreation patterns can influence site selection for wastewater disposal. Historic and existing land use patterns can be used to make projections of future development patterns. These projections are then used to develop planning design criteria.

2.3.1 Population

The Grants Pass area has experienced steady population growth since the 1920s. This increase has been consistent with the national population trend of people moving to the west and southwest from the northeastern and central states and to rural areas from urban areas. The population in Josephine County rose during the 1970s (5.11 percent annually), but slowed dramatically during the 1980s (0.6 percent annually). Grants Pass grew more rapidly than the county during the last decade (1.5 percent annually). The 1990 census lists the population of Grants Pass at 17,424. Current population within the city limits in 2003 is estimated at 22,444.

The population within the area currently served by the collection system consists primarily of the population within the city limits and the population within the Harbeck-Fruitdale area (approximated at 4,620 through sewer connection and billing records). The future sewerage service area will include the existing service area and the Redwood area. In 2003, the Redwood population was about 5,714. Adding this population to the existing sewer service area population yields a population of 32,778 in 2003. It is also necessary to take into account the commercial and industrial contribution as a form of a population equivalent. This Plan assumes that commercial/industrial population equivalent is 35 percent of the total residential population.

Existing service area populations and the total population equivalent served are summarized in Table 2-3. Average growth rates assumed for the future population are based upon projections presented in the City's *Comprehensive Plan* (City of Grants Pass, 1982).

Table 2-3. Existing and Future Service Area Populations

Area	1997 Population	2003 Population	2020 Population	Average Growth Rate, Percent/Year
City Limits	20,526	22,444	28,908	1.5
Harbeck-Fruitdale	4,200	4,620	6,053	1.6
Redwood	4,758	5,714	9,600	3.1
Commercial/Industrial Population Equivalent	10,319	11,472	15,596	1.8
Total Service Area Population Equivalent:	**39,803**	**44,250**	**60,157**	

2.3.2 Land Use

The three main categories of land use are residential, commercial, and industrial. The City planning department keeps records on the amount of land in each zoning category within the city limits. The City assigns each land parcel a designation of improved, unimproved, or exempt. This Plan assumes that only improved land is currently served by the City WRP. The planning department estimates that there are approximately 2,912 acres of improved land within the city limits. The total amount of improved, unimproved, and exempt land within the city limits is 3,986 acres.

Land use records for Harbeck-Fruitdale area were unavailable, so land use within these areas was estimated. This Plan assumed that the Harbeck-Fruitdale area is primarily residential with minor commercial and industrial land use, and is currently developed at 40 percent capacity. Total land area within Harbeck-Fruitdale was determined from City maps and is estimated as 1,900 acres.

Land use estimates are summarized in Table 2-4. Future land use for each category was estimated as the sum of existing improved, unimproved, and exempt land for that category. While it is unlikely that all land within the service area will be developed by 2013, annexations of additional land into the service area should offset any remaining undeveloped land.

Table 2-4. Service Area Land Use Summary

Land Use Category	Area in Acres			
	1980[a]	1990	2000[a]	2013
Residential	1,816	2,746	3,755	4,414
Commercial	362	591	743	993
Industrial	464	334	473	477
Public and Semi-Public	493	[b]	776	200[b]
Total:	**3,135**	**3,671**	**5,747**	**5,885**

[a] Obtained from 1983 Sewerage Collection System Master Plan.

[b] Public and semi-public land area included in other values.

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Regulatory and Procedural Issues



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3. REGULATORY AND PROCEDURAL ISSUES

Federal, state, and local regulatory agency policies and procedures affect the installation, upgrades, and operation of the City's wastewater collection system and treatment facility. The impacts of these policies and procedures on wastewater management planning in the Grants Pass area are described below. The discussion of regulations presented is not exhaustive and is focused on those regulations and laws that are relevant to wastewater conveyance and treatment.

3.1 FEDERAL POLICY

Federal policies that will affect the planning process include the Federal Water Pollution Control Act/Clean Water Act; Safe Drinking Water Act; and the proposed Capacity, Management, Operation and Maintenance Rules.

3.1.1 Federal Water Pollution Control Act/Clean Water Act

Since its enactment, the Federal Water Pollution Control Act, also known as the Clean Water Act (CWA), has formed the foundation for regulations detailing specific requirements for pollution prevention and response measures. The CWA requires states to adopt water quality standards consistent with federal limitations on pollutant and thermal loading. The standards are to take into consideration the use of the waters for public water supplies; propagation of fish and wildlife; recreational purposes; and agricultural, industrial, and other beneficial uses.

The City's collection system conveys wastewater to the City WRP, where it is treated before discharge to the Rogue River. State policies specifically regulate pollutant and thermal loading to comply with federal policy as detailed in the CWA.

3.1.2 Safe Drinking Water Act

The Safe Drinking Water Act (SDWA) authorizes the Environmental Protection Agency (EPA) to set national health-based standards for drinking water to protect against contaminants that may be found in drinking water. Wastewater flows that are collected in or travel through substandard collection systems have the potential to contaminate drinking water systems. State and local regulations are designed to comply with the SDWA, and prohibit activities that could cause an adverse impact on existing or potential beneficial use of groundwater.

3.1.3 Proposed Capacity, Management, Operation, and Maintenance Rule

EPA is proposing rules that will govern the manner in which municipalities and special service districts manage and operate wastewater collection systems. The proposed Capacity, Management, Operation and Maintenance (CMOM) Rule, depending on its final promulgated form, may have a significant affect on collection system development and operation and maintenance (O&M) for the City.

Under the proposed rule, sanitary sewer overflows (SSOs) would be prohibited unless caused by severe natural conditions such as widespread flooding, earthquakes, or other natural disasters. Owners of collection systems would be required to provide adequate capacity for peak flows in all parts of the system, monitor and report on SSOs, and make the SSO control program and reports available for public review.

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There are two aspects of the proposed rule that are under close scrutiny of the reviewing community. First, satellite sewer systems would be operated under separate NPDES permits. Satellite sewer systems are loosely defined in the proposed CMOM rule as any agency that conveys wastewater to another agency for additional conveyance and final treatment and discharge. For Grants Pass, the RSSSD would be considered a satellite system requiring its own NPDES permit.

Second, the proposed CMOM rule is vague on the design threshold to which SSOs must be controlled. For instance, the proposed rule is silent on the recurrence interval of a storm event above which SSOs would be allowed (e.g., would SSOs be permitted during storms greater than a 1-in-5 year event?). As such, EPA offers wastewater agencies neither clear guidance regarding the amount of additional pipeline and pump station construction that would be required under CMOM, nor understanding about the amount of additional maintenance effort required to ensure elimination of SSOs.

The CMOM rule has not yet been implemented. Based upon a recent (April 2004) statement on the EPA SSO web page, "SSO Proposed Rule was withdrawn from publication in the Federal Register," the timeline for implementation of the CMOM rule is uncertain. How the CMOM rule will eventually be interpreted and applied in Oregon is also uncertain. One possibility is that Oregon's "bacteria rule," (see Section 3.2.2), will be used to set a minimum threshold for SSO prevention. A record of SSOs from the City's collection system since 1997 is presented in Section 4.2.

3.2 STATE POLICY

3.2.1 National Pollutant Discharge Elimination System

Section 402 of the CWA provides the legal basis for the NPDES permit program, which regulates point and nonpoint discharges. ODEQ is authorized by the EPA to administer the NPDES program through Oregon Revised Statute 468B and associated OARs. These rules and statutes include regulations for wastewater collection, treatment, control, and disposal. Under the conditions of NPDES permits, permittees are allowed to construct, install, modify, or operate these systems only in conformance with the Federal Clean Water Act and the above-mentioned State statutes that set forth requirements, limitations, and conditions for such activities.

The Grants Pass WRP plant operates under NPDES Permit Number 101985 issued December 29, 2000. The permit expires November 30, 2005. A copy of the NPDES permit is provided in Appendix A.

3.2.2 Bacterial Control Management Plan

As noted previously, EPA is currently considering proposed CMOM rules that will limit the number of allowable SSOs. While the proposed CMOM rule is silent on collection system design criteria, Oregon has already adopted a rule that addresses sewer overflows as a function of rainfall events. This rule, commonly referred to as the "bacteria rule," therefore, indirectly offers some guidance to design engineers and collection system owners. The OAR seeks to protect receiving waters and drinking water sources by prohibiting discharge of untreated wastewater to waters of the state except during the following conditions:

- During the period of November 1 through May 21, except during a storm event greater than the 1-in-5-year, 24-hour duration storm.

- During the period of May 22 through October 31, except during a storm event greater than the 1-in-10-year, 24-hour duration storm.

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The State Environmental Quality Commission may approve a change to these rules on a case-by-case basis as described in OAR 340-41-120. Determining the causes and preventing against SSOs usually requires a municipality to thoroughly evaluate both the collection and treatment system to determine the extent of extraneous weather-related flow, system structural condition and reliability, system hydraulic and treatment capacity, and the efficacy of operation and maintenance practices. The City has already performed a thorough analysis of the treatment system as part of the WRP expansion. This Plan presents a parallel analysis of the collection system condition and operation.

3.2.3 Groundwater Regulations

The Federal SDWA requires that state underground injection control programs be established to ensure that underground injection will not endanger drinking water sources. In Oregon, groundwater regulations, including regulatory requirements for injection controls, are administered by ODEQ. On-site drainfields and septic systems that serve 20 or more persons are considered injection wells by ODEQ. The City Development Code requires that all new development within the service area be connected to the wastewater collection and treatment system. Existing development using septic systems is required to connect to the public sewer system at such time as repair or replacement of existing facilities is necessary, if the public sewer is within 300 feet of the property.

3.3 LOCAL POLICY AND ORDINANCES

Local requirements of particular concern to the planning process are related to the City Municipal Code, City Development Code, and the Sanitary Sewer Lateral Replacement Policy.

3.3.1 City of Grants Pass Municipal Code

City Ordinances 4861 and 5028 have been adopted by the City as Chapter 8.50 of the City Municipal Code. Chapter 8.50 is intended to protect public health and safety; protect the environment; and ensure compliance with all applicable state and federal laws as they pertain to wastewater collection, conveyance, treatment, and discharge.

Sewer use requirements set forth in Chapter 8.50 include general and specific prohibited discharge standards. The general prohibitions state that no user shall introduce or cause to be introduced into the City WRP any pollutant or wastewater which causes pass-through or interference, or which will cause the WRP to violate its NPDES permit or harmfully impact the receiving water quality standards. Chapter 8.50 also sets forth procedures for the allowance of intentional bypass occurrences, and the reporting of unanticipated bypass.

3.3.2 City of Grants Pass Development Code

Title 10 of the City Municipal Code may also be cited as the City Development Code. The purpose of the Development Code is to coordinate City regulations governing the development and use of land. Standards for sewer and septic systems are set forth in the Development Code to ensure compliance with state and federal statutes, policies, and laws designed to prevent harmful impact to receiving waters.

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3.3.3 Sanitary Sewer Lateral Replacement Policy

Substandard or combined sewer laterals discovered during public sewer, water, or storm drain projects are required to be replaced. The City considers the sewer lateral to be the responsibility of the private property owner from the point of connection to the main to the building being served. Replacement of substandard sewer laterals may often include work within the public right-of-way, with the possibility of additional costs such as pavement patching, traffic control, and other construction items not usually associated with work within private property boundaries. To assist the property owner in the cost of lateral replacement, the City has adopted a Sanitary Sewer Lateral Replacement Policy. Under this policy, the City Manager can authorize payment of 50 percent of the cost of replacing failed or otherwise substandard laterals. A copy of the policy is included in Appendix C.

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Existing Collection System



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4. EXISTING COLLECTION SYSTEM

The extent of the City's service area is shown in Figure 4-1 (page 4-4) together with the location of major collection system components such as lift stations, pump stations, and the WRP. The service area consists of area within the city limits, the Harbeck-Fruitdale area, and the Redwood Sanitary Sewer Service District (RSSSD).

4.1 COLLECTION SYSTEM PIPELINES

The City's collection system is divided into 23 drainage basins that are denoted alphabetically (in Figure 4-2 [page 4-5]). Each major drainage basin is further divided into subbasins that are denoted alphanumerically (i.e., Subbasins A1, B1, etc.). Drainage basin boundaries almost always follow topographic features such as ridge lines, streams, and rivers. To some extent, subbasin delineations also follow topographic features. However, the City set up the subbasin boundaries primarily to aid operation and maintenance efforts.

Pipeline length and age data for each of the drainage basins are presented in Table 4-1 (page 4-11). Age data is further presented in Figure 4-3 (page 4-6) as a color-coded map. In addition, a summary of the pipeline size, length, and age are presented in Figure 4-4 (page 4-7).

Approximately one-half of the gravity sewers comprising the City's collection system are at least 30 years old and may be nearing the end of their operational life. Most of the older pipelines are clustered in the downtown area, with newer pipelines located in the periphery of the City in areas of more recent development. Beginning in the late 1920s and extending through the mid-1960s, pipelines were constructed of locally made concrete. These pipelines have had poor long-term performance characteristics, being prone to corrosion, spalling, collapse, and joint leakage. Some of the most critical trunk and mainline pipelines in the collection system were constructed of this substandard material.

There are numerous diversion structures located throughout the collection system. These diversions are shown in Figure 4-5 (page 4-8). Diversions shunt flow from over-capacity pipelines to adjacent pipelines and effectively reduce peak flows in the over-capacity pipelines during rain events. Diversions can be an operationally effective and economic means of alleviating hydraulic constraints and preventing SSOs. However, the diversions were not constructed in accordance with a comprehensive operational strategy. Despite diversions, SSOs still have occurred within the collection system. SSOs that have occurred since 1997 are listed in Table 4-2 (page 4-12).

Since 1986, the City has been compiling sewer condition and maintenance data into a central electronic database. These condition and maintenance data have been linked to the City's geographic information system (GIS). The result is a powerful tool for tracking and visualizing collection system data. These data offer a historical record of a number of structural conditions and maintenance issues for individual pipeline sections. Table 4-3 (page 4-12) is a comprehensive listing of records available for individual pipeline sections based on records developed and recorded by City maintenance staff. These GIS data have been used in the Maintenance and Reliability Analysis presented in Section 7.

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4.2 PUMP STATIONS

The topography of the City's service area is such that most of the system is operated under gravity flow conditions. As such, there are only a few pump stations in the collection system (see Figure 4-1 [page 4-4]). The Webster No. 1 Lift Station, Webster No. 2 Lift Station, and Bridge Street Pump Station are all located in the southwestern portion of the city. Under an intergovernmental agreement, the City also operates two pump stations serving the RSSSD. The RSSSD pump stations include the Darnielle Pump Station, located on South River Road, and the Redwood Pump Station, located at the site of the abandoned RSSSD wastewater treatment plant at the end of Leonard Road.

Pump station operational data are presented in Table 4-4 (page 4-13).

4.2.1 City Pump Stations

The service areas of the Webster No. 1 and No. 2 Lift Stations and the Bridge Street Pump Station are as shown in Figure 4-6 (page 4-9). The Webster No. 1 and No. 2 Lift Stations are separate wet well/dry well type stations, each with a firm capacity of 0.14 million gallons per day (mgd), constructed in 1967 to serve the wide, flat Rogue River Valley located generally between Lower River Road and the Rogue River from the WRP to the Western edge of the UGB. These lift stations predominately serve mobile home parks and All Sports Park. Webster No. 2 Lift Station is located at the east end of Roguelea Estates and the west end of Webster Road. Webster No. 1 Lift Station is located within All Sports Park along the access road to several sports fields. Both stations contain dual self-priming, vertical close-coupled, nonclog centrifugal pumps. Detailed drawings of both Webster No. 1 and No. 2 Lift Stations are included in Appendix D.

The Bridge Street Pump Station is a duplex submersible type station, with a firm capacity of 0.94 mgd, constructed in 1994 to serve the area generally between the Lower and Upper River Roads, from the eastern edge of All Sports Park to the western edge of the UGB. Construction of this station was very difficult because the contractor encountered extensive groundwater and sloughing soil issues. This station was planned to serve a very large area west of Leonard and Cottonwood Streets to the western edge of the UGB. Due to construction issues at the pump station, however, several shallow sewers were built along Lower River Road, which prevent gravity sewer extensions beyond Schaeffers Lane. Several building lots in this area are served with individual grinder pump stations and long small-diameter force mains to the existing shallow gravity sewer. In the future, a new deep gravity sewer must be constructed to extend gravity sewer service to the west edge of the UGB. This new gravity sewer would enable these grinder pumps to be eliminated in the future.

The Bridge Street Pump Station is located at the southeast corner of Bridge Street and Tami Road, and dual (4- and 8-inch-diameter) force mains travel east on Bridge Street about 1,900 feet and discharge in Manhole B111. Detailed drawings of the Bridge Street Pump Station are included in Appendix D. Appendix D also includes a drawing prepared during the design of the Bridge Street Pump Station showing the proposed service area of this station and the preliminary design of gravity sewers in its service area.

A future Spalding Lift Station will very likely be required to serve the east edge of the UGB south of Northeast "N" Street and east of Shannon Lane. This lift would also serve the area generally east of Ament Road that is too low in elevation to be served by the existing sewer in Spalding Avenue and Shannon Lane.

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The exact location of this lift station has not yet been identified; however, it would probably be located near the intersection of Shannon Lane and Portola Drive and discharge into the existing sewer in Portola Drive. This lift station could then serve the homes along Inman Lane next to the Rogue River. The probable service area of this lift station is shown in Figure 4-6.

4.2.2 RSSSD Pump Stations

The Redwood Conveyance System, which transfers all flow from the old abandoned Redwood Wastewater Treatment Plant (WWTP) to the Grants Pass WRP, includes the Redwood Pump Station, the Redwood force main, the Darneille Pump Station, the Darneille force mains, and a gravity sewer which brings the transferred flow to the Grants Pass WRP. The general configuration of the system is as shown in Figure 4-7 (page 4-10).

The Redwood Pump Station is located at the old Redwood WWTP. It is a duplex submersible pump station with a firm capacity of 0.48 mgd, based on one pump in operation. The station has a Bioxide chemical injection system with a 3,000-gallon chemical storage tank. From the Redwood Pump Station, flow is routed through approximately 10,300 feet of 6-inch-diameter force main to an influent manhole at the Darneille Pump Station.

The Darneille Pump Station accepts the majority of the flow from the RSSSD, as well as the flow pumped from the Redwood Pump Station. The Darneille Pump Station has a firm capacity of 4.2 mgd, based on operating two of three pumps. The station is a wet well/dry well type station with above-grade electrical panels, generator, and chemical feed system. The chemical feed system is identical to that provided at the Redwood Pump Station, except that the chemical feed pumps are slightly larger.

From the Darneille Pump Station, flow is pumped through approximately 17,940 feet of dual 12-inch force main and then 1,000 feet of single 14-inch force main. From the pump station, the dual force mains are routed south to South River Road; then east through road rights-of-way and easements to the south side of the Pedestrian Bridge. The dual 12-inch force mains join into a single 14-inch force main that crosses the Rogue River on the Pedestrian Bridge and then discharges into a gravity sewer which flows to the WRP.

Detailed drawings of the Redwood and Darneille Pump Stations are provided in their respective Operation and Maintenance Manuals (Parametrix, 2002).

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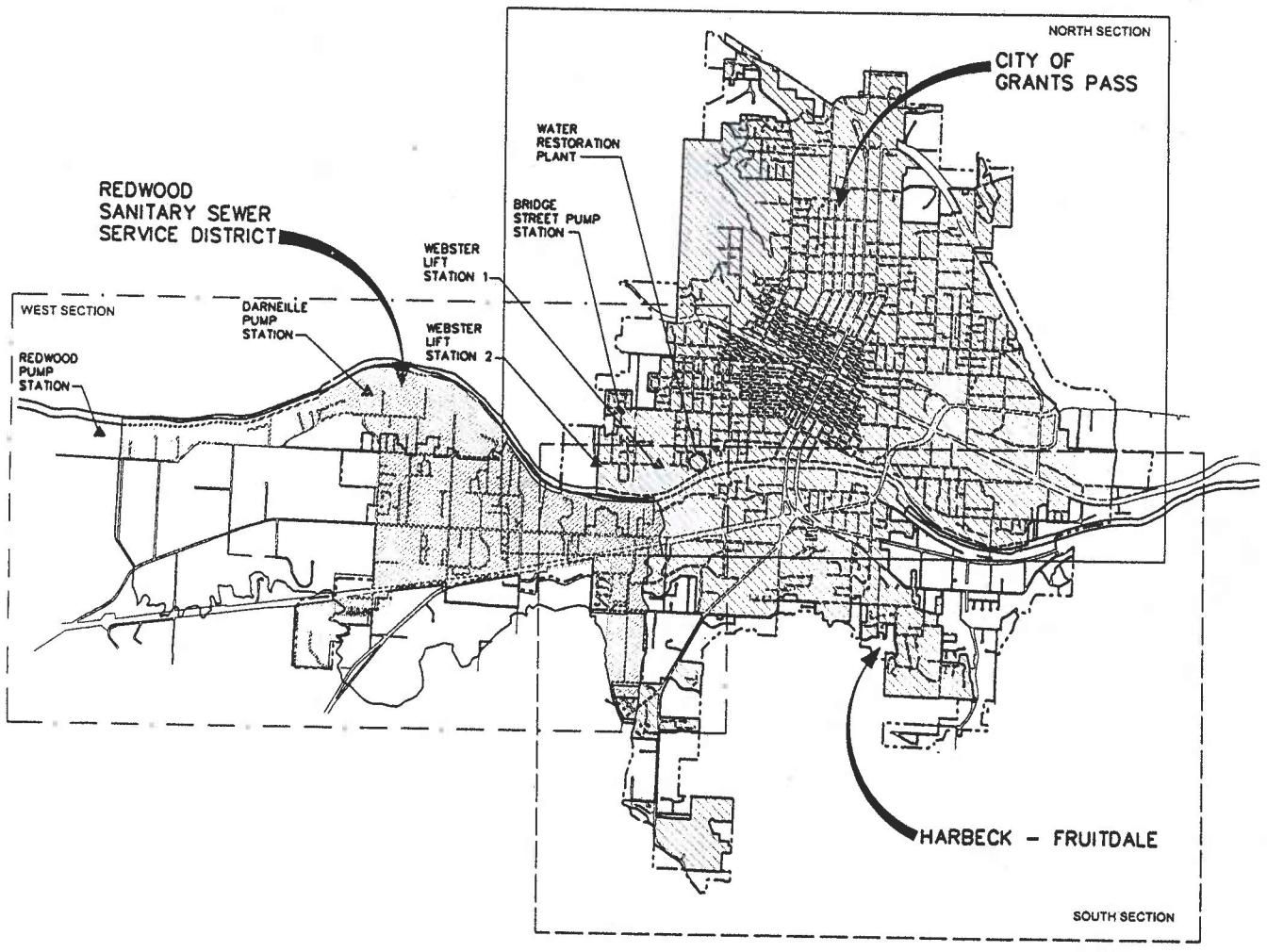

CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

LEGEND

- NORTH SECTION OF STUDY
- SOUTH SECTION OF STUDY
- WEST SECTION OF STUDY
- URBAN GROWTH BOUNDARY
- CITY OF GRANTS PASS
- REDWOOD SANITARY SEWER SERVICE DISTRICT (RSSSD)
- CURRENTLY SERVED PORTION OF RSSSD

SCALE IN FEET
0 2000' 4000'

N

Figure 4-1
Major Collection System Components and Service Area

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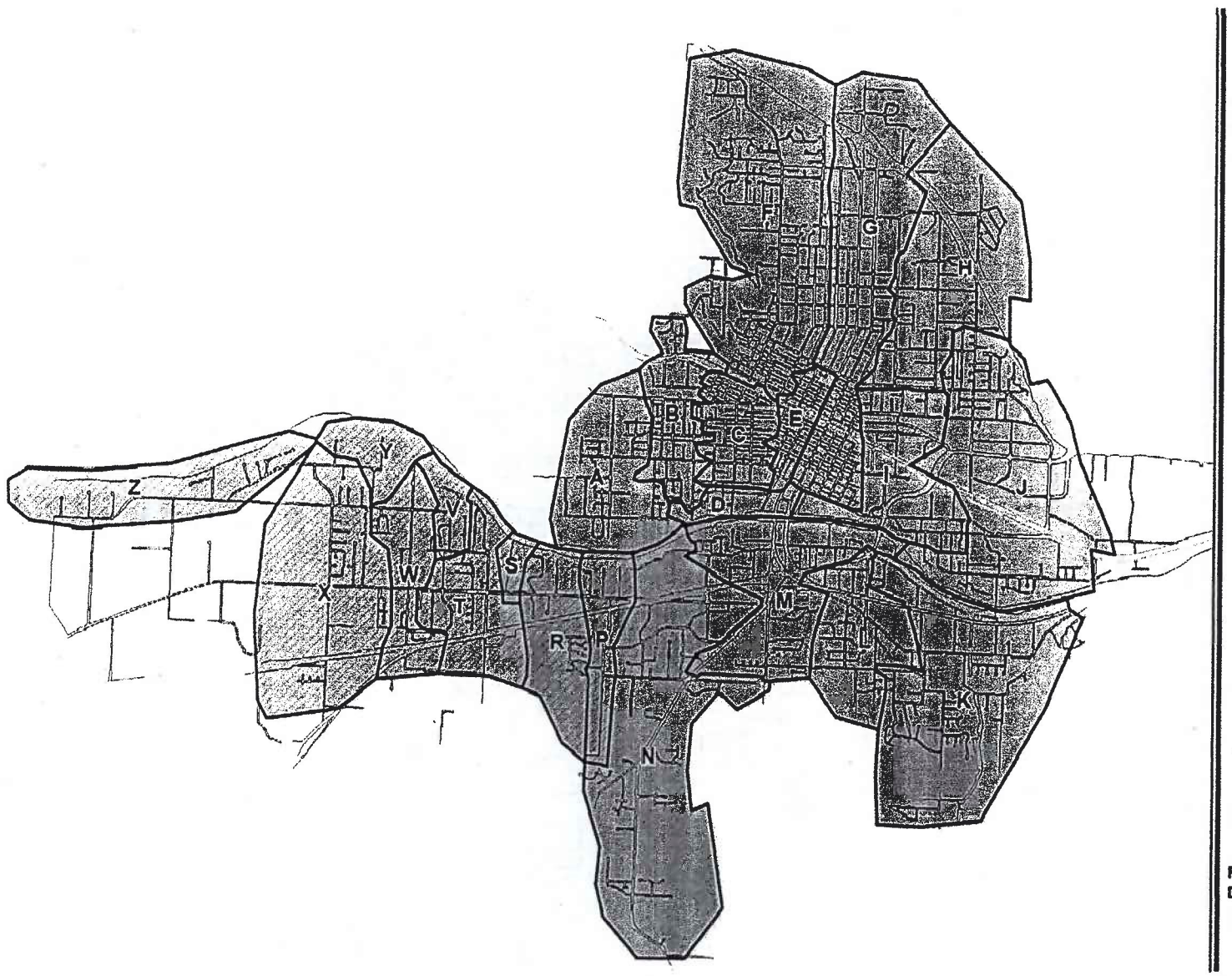


CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

Legend

City of Grants Pass Drainage Basins

Redwood Sanitary Sewer Service District Drainage Basins

SCALE IN FEET
0 1,500 3,000

N

Figure 4 - 2
Drainage Basins

000745

96-746



CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

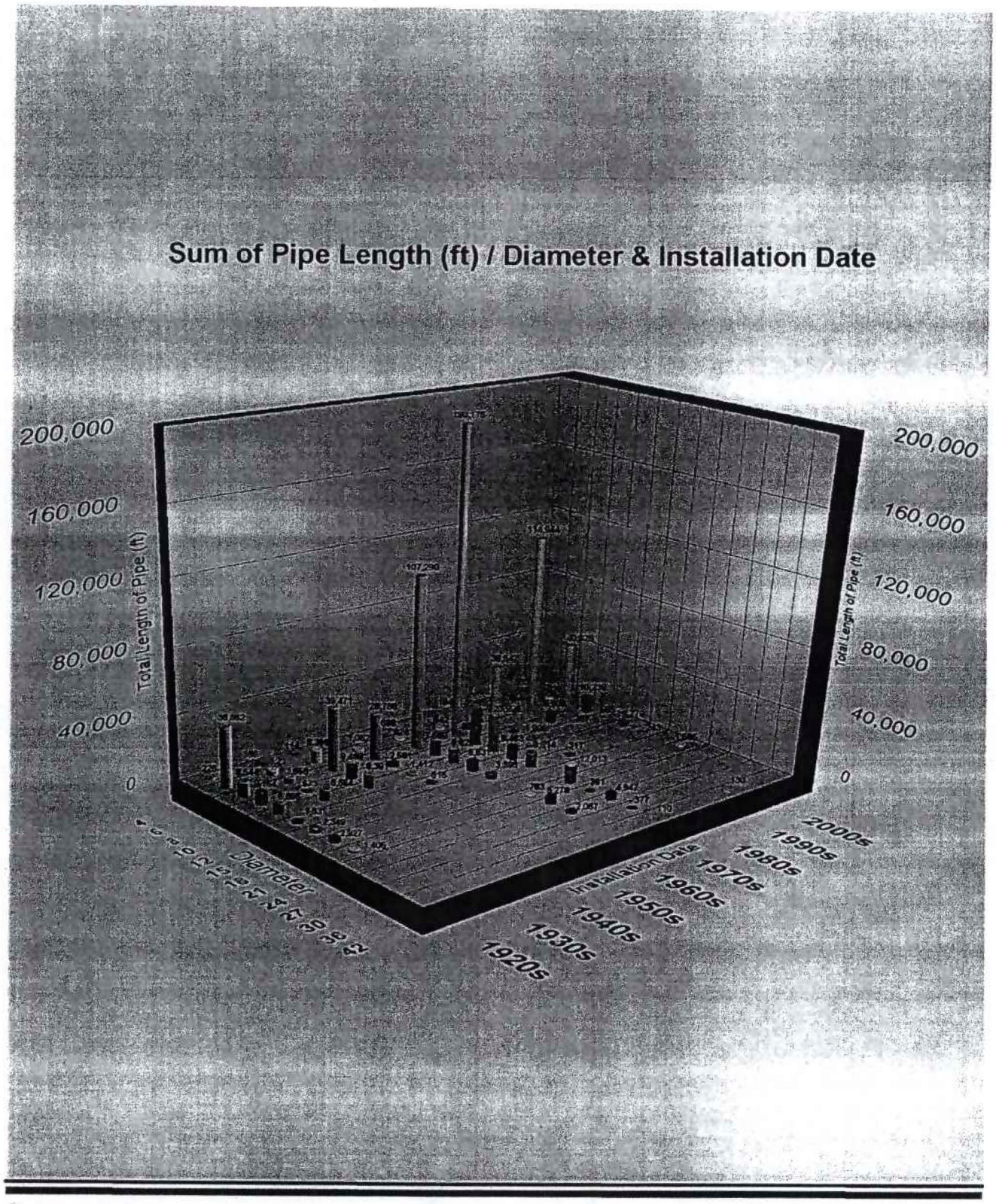
Pipe Age Distribution

1920 - 1939
1940 - 1959
1960 - 1965
1966 - 1979
1980 - Present

SCALE IN FEET
0 1,500 3,000

N

Figure 4 - 3
Pipe Age Distribution

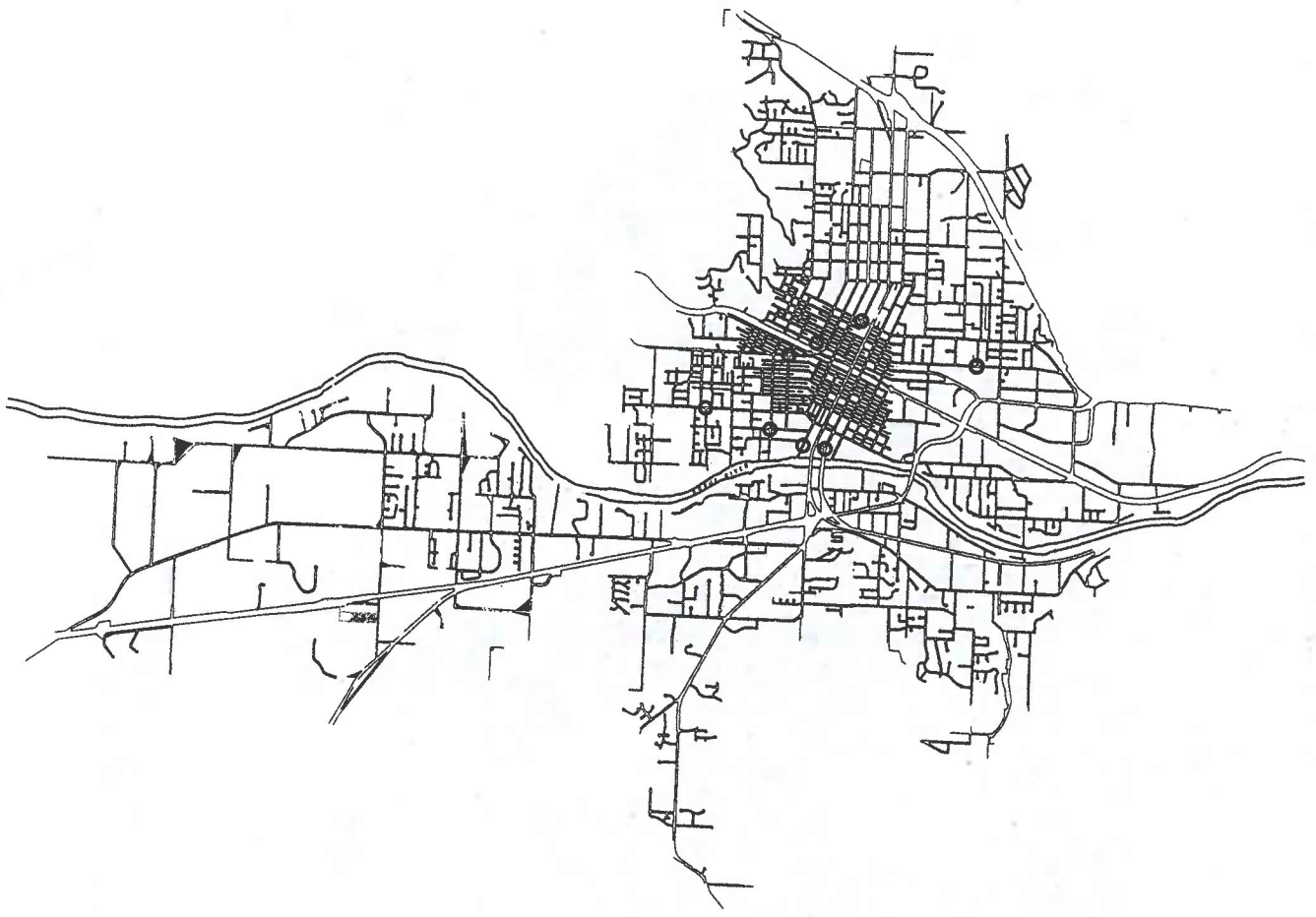


Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 4 - 4

Figure 4 - 4
Sum of Pipe Length/Diameter & Installation Date

COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON

006748



CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

LEGEND

O EXISTING DIVERSION

SCALE IN FEET
0 2000' 4000'

N

Figure 4-5
Existing Diversion Locations

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CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

SERVICE AREAS

- BRIDGE STREET PUMP STATION
- WEBSTER No.1 LIFT STATION
- WEBSTER No.2 LIFT STATION
- SPALDING LIFT STATION (PROBABLE FUTURE SERVICE AREA)

Figure 4-6
The City of Grants Pass
Pump Station Service Areas

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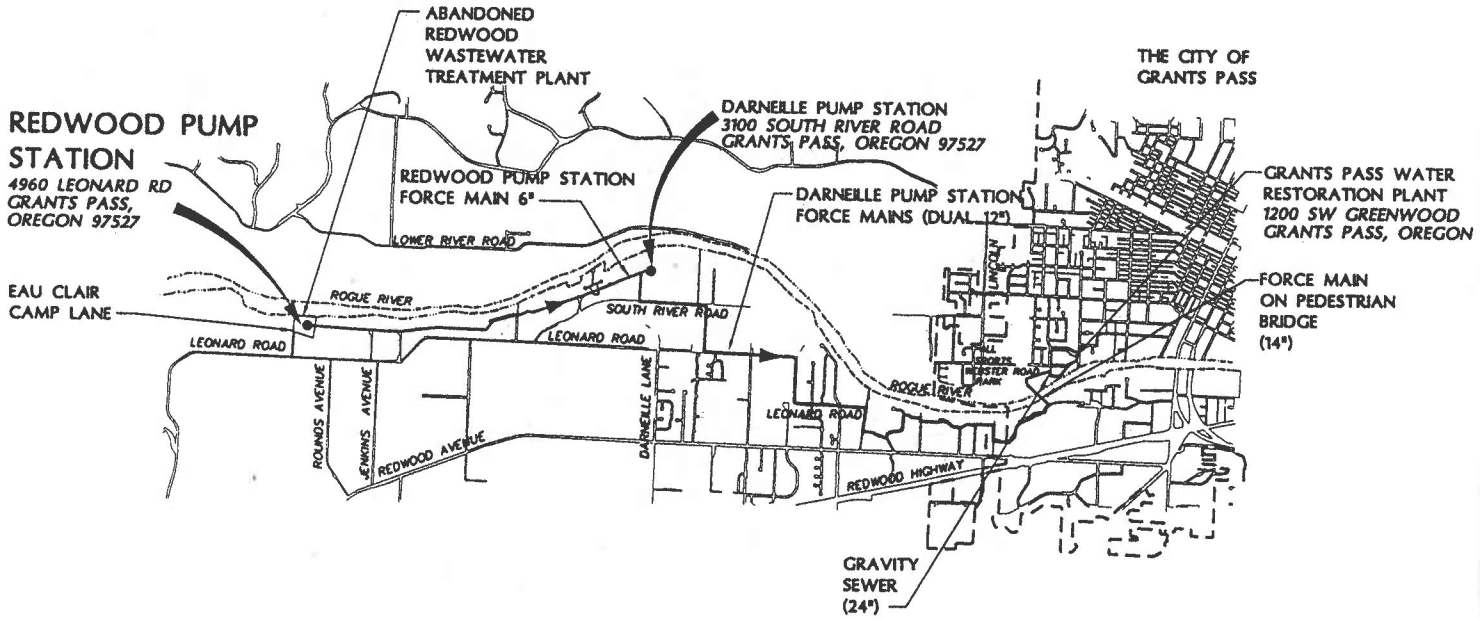

CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

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Figure 4-7
General Configuration of
RSSSD Pump Stations

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Table 4-1. Pipeline Length and Age Data by Basin

Basin	1920 and Before		1921 to 1930		1931 to 1940		1941 to 1950		1951 to 1960		1961 to 1965		1966 to 1970		1971 to 1980		1981 to 1990		1991 to 2000		2000 and Beyond		Total	
	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent of Basin[a]	Length, Feet	Percent
A													4,108	14.6	367	1.3	3,618	12.8	17,851	63.3	2,235	7.9	28,179	3.5
B							8,811	25.9	2,625	7.7	7,308	21.5	8,703	25.6	2,285	6.7			3,639	10.7	610	1.8	33,981	4.2
C	7,919	24.5	7,589	23.4	4,631	14.3	4,967	15.3	2,373	7.3	1,092	3.4	1,176	3.6	485	1.5	876	2.7	747	2.3	526	1.6	32,381	4.0
D			5,807	45.3					826	6.4					6,181	48.2							12,814	1.6
E	9,803	60.5	4,924	30.4									314	1.9					1,169	7.2			16,210	2.0
F	2,482	2.5	3,993	4.0			18,507	18.7	2,073	2.1	7,097	7.2	14,044	14.2	16,045	16.2	14,001	14.1	17,753	17.9	3,164	3.2	99,159	12.4
G	10,132	14.1	14,959	20.9	588	0.8	11,824	16.5	6,506	9.1	4,781	6.7	8,580	12.0	4,712	6.6	1,362	1.9	7,560	10.5	675	0.9	71,679	9.0
H			8,064	20.2			8,454	21.1	5,771	14.4	1,959	4.9	6,132	15.3	6,622	16.6	1,273	3.2	1,643	4.1	66	0.2	39,984	5.0
I							14,131	46.2	6,847	22.4	4,365	14.3	321	1.0	2,209	7.2	1,247	4.1	1,481	4.8			30,601	3.8
J							6,348	8.5	1,690	2.3	18,729	25.2	16,020	21.5	11,432	15.4	11,313	15.2	5,862	7.9	2,984	4.0	74,378	9.3
K											204	0.3	37,432	54.2	11,141	16.1	1,420	2.1	17,124	24.8	1,701	2.5	69,022	8.6
L													23,156	66.9	4,860	14.0	2,906	8.4	2,119	6.1	1,563	4.5	34,604	4.3
M											26,051	49.4	18,785	35.6	920	1.7	2,009	3.8	4,691	8.9	328	0.6	52,784	6.6
N											10,214	14.6	19,082	27.2	16,696	23.8	8,733	12.5	8,499	12.1	6,804	9.7	70,028	8.7
P													9	0.1	11,743	85.9	17	0.1	1,000	7.3	909	6.6	13,678	1.7
R															6,916	54.9			5,678	45.1			12,594	1.6
S															3,450	91.0					343	9.0	3,793	0.5
T															13,303	62.0	2,380	11.1	5,772	26.9			21,455	2.7
V															5,701	68.9			1,571	19.0	998	12.1	8,270	1.0
W															9,678	54.2	1,945	10.9	5,471	30.6	758	4.2	17,852	2.2
X															18,607	53.9	4,979	14.4	7,108	20.6	3,825	11.1	34,519	4.3
Y															9,125	90.6			943	9.4			10,068	1.3
Z															10,114	78.9			2,706	21.1			12,820	1.6
Totals:																								
Length, Feet	30,336		45,336		5,219		73,042		28,711		81,800		157,862		172,592		58,079		120,387		27,489		800,853	
Percent[b]	3.8		5.7		0.7		9.1		3.6		10.2		19.7		21.6		7.3		15.0		3.4			
Cumulative Percent[c]	3.8		9.4		10.1		19.2		22.8		33.0		52.7		74.3		81.5		96.6		100.0			

[a] Percent of all pipes in any basin, as reported in the City database. For instance, 60.5 percent of all pipes currently reported in Basin E were constructed in or before 1920, while 30.4 percent of all pipes in Basin E were constructed between 1921 and 1930.

[b] Percentage of all pipes in the service area that were constructed in any time period. For instance, 3.8 percent of all pipes were installed in or before 1920, while nearly 30 percent of all pipes were installed between 1961 and 1970 (10.2 percent between 1961 and 1965 plus 19.7 percent between 1966 and 1970).

[c] Cumulative percentage of all pipes installed in the system. For instance, about 53 percent of all pipes in the service area are at least 30 years old, as can be seen for the cumulative percentage report for the period 1966 to 1970.

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Table 4-2. Collection System Overflows (1997 to Present)

Date	Location	Cause
June 9, 1997	738 NW Amelia Drive	Broken Service Line
July 25, 1997	1044 NW 6th Street	Maintenance Accident
March 23, 1998	NE 9th Street	Sewer Obstruction During Storm Event
April 21, 1998	928 NE 12th Street	Sewer Obstruction During Storm Event
April 22, 1998	Treatment Plant Pump Station	Power Outage and Backup Generator Failure
June 29, 1998	NW Savage Street	Sewer Obstruction
August 29, 1998	2075 Highland Street	Support Pillar Dislodged Sewer Pipe at Creek Crossing
September 18, 1998	NW Savage Street	Unpermitted Discharge Water Line Flushing Water into Collection System
January 14, 2000	Manzanita Avenue	Storm Event
December 16, 2000	Bellaire Drive	Grease Problem

Table 4-3. Historical Records Available for Pipe Segments

Structural Data (Defect)	Age Data (Year Constructed)	Maintenance Data (Maintenance Effort/Condition)
Missing Pipe	1920–1939	6-Month Cleaning
Hole in Pipe	1940–1965	<6 Gallons of Rock Removed During Cleaning
Broken Pipe	1966–1979	3–5 Gallons of Rock Removed During Cleaning
Circumferential Crack	1980–Present	1–2 Gallons of Rock Removed During Cleaning
Longitudinal Crack		Heavy Grease
Crack (Other)		Medium Grease
		Light Grease
		Heavy Root Encroachment
		Medium Root Encroachment
		Light Root Encroachment
		Service Call

00752

Table 4-4. Pump Station Data

Location	Year Constructed	No. of Pumps	Pump Type	Horsepower (hp)	Drive Type	Capacity (gpm)	Head (feet)
Webster No. 1 Lift Station:							
East edge of Roguelea Estates	1967	2	Self-priming, vertical close-coupled, nonclog centrifugal	7.5	Constant Speed	100	23
Webster No. 2 Lift Station:							
All Sports Park – Basin A	1967	2	Self-priming, vertical close-coupled, nonclog centrifugal	3	Constant Speed	100	10
Bridge Street Pump Station:							
Bridge Street and Tami Court – Basin A	1994	2	Submersible, nonclog centrifugal	20	Variable Speed	650	75
Redwood Pump Station:							
4960 Leonard Road – RSSSD	2000	2	Submersible, nonclog, centrifugal	40	Variable Speed	335	163
Darneille Pump Station:							
3100 South River Road – RSSSD	2000	3	Immersible dry-pit, screw centrifugal	110	Variable Speed	1,460	180

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Evaluation Criteria

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5. EVALUATION CRITERIA

An important activity of this planning effort was to work with the City to develop evaluation criteria. These criteria were used to assess the condition and capacity of existing collection system components to dependably convey existing and future flows to the Grants Pass WRP. The evaluation and analysis criteria were used to develop the weighting factors used in the Maintenance and Reliability Analysis presented in Section 7 and to develop the cost estimates for the recommended improvements that are shown in the Capital Improvement Plan in Section 8.

5.1 BASIS OF ESTIMATED CONSTRUCTION COST

Construction costs evaluated in this Plan are based on preliminary layouts of proposed projects. Unit prices were then applied to the estimated length of each size of collection system pipeline for each project to determine the anticipated construction cost of the project based on 2003 costs. Unit prices applied to each size of pipeline are shown in Table 5-1 (page 5-2). Table 5-1 contains unit pricing for each component for gravity sewer pipeline construction together with the total unit pricing for pipeline installation per linear foot. In considering the estimates, it is important to realize that changes during final design and future changes in the cost of materials, labor, and equipment will cause changes in cost estimates presented.

5.2 BASIS OF TOTAL PROJECT COSTS

The total project cost of a proposed project is defined as the sum of estimated construction cost as described in Section 5.1; a project contingency; the cost of administration, engineering, legal, and financial services; and the cost for any additional land or right-of-way required for the project, if required. Each of these cost components is discussed below.

5.2.1 Estimated Construction Cost

Construction cost estimates consist of costs the contractor is expected to charge the City for building proposed facilities. Construction costs include the cost of labor, materials, equipment, subcontractors, mobilization, overhead and profit, and contingencies. These costs are described in Paragraph 5.1, Basis of Estimated Construction Cost.

5.2.2 Contingencies

A contingency is an allowance for undefined cost items. This allowance covers work items that will have to be performed, or other cost elements that will be incurred, that were not explicitly foreseen at the time of the estimate because of lack of complete and accurate information. In this Plan, a contingency of 40 percent on construction costs was applied to all cost estimates. (Note: The level of accuracy normally associated with planning-level cost estimates should not be confused with contingency.)

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Table 5-1. Gravity Sewer Pipeline Construction Cost Unit Prices Used for Estimating Project Construction Costs

	Assume 2′ Trench 8′-8″ Deep		Assume 4′ Trench 12′ Deep						
	4″ PVC ($/LF)	8″ PVC ($/LF)	12″ PVC ($/LF)	15″ PVC ($/LF)	18″ PVC ($/LF)	21″ RCP ($/LF)	24″ RCP ($/LF)	30″ RCP ($/LF)	36″ RCP ($/LF)
Saw Cut	$4.50	$4.50	$4.50	$4.50	$4.50	$4.50	$4.50	$4.50	$4.50
3/4 Minus, Assume Total Depth 12′	11.22	18.94	18.94	18.94	18.94	18.94	18.94	18.94	18.94
Excavation and Backfill	6.98	23.75	23.75	23.75	23.75	23.75	23.75	23.75	23.75
Dispose AC	0.56	0.83	0.83	0.83	0.83	0.83	0.83	0.83	0.83
Remove Existing Pipe	5.00	7.05	7.05	8.20	8.20	8.20	10.25	10.25	13.70
New AC	3.70	5.56	5.56	5.56	5.56	5.56	5.56	5.56	5.56
Install Pipe (includes pipe cost)	4.50	6.11	10.95	17.14	20.17	35.00	39.50	82.00	101.50
Trench Shoring	29.60	51.36	51.36	51.36	51.36	51.36	51.36	51.36	51.36
Subtotal:	**$66.06**	**$118.10**	**$122.94**	**$130.28**	**$133.31**	**$148.14**	**$154.69**	**$197.19**	**$220.14**
Unaccounted for Items – 25%:	16.52	29.53	30.74	32.57	33.33	37.04	38.67	49.30	55.04
Total (Rounded):	**$80.00**	**$150.00**	**$155.00**	**$165.00**	**$170.00**	**$185.00**	**$200.00**	**$250.00**	**$275.00**

5.2.3 Administrative, Engineering, Financial, and Legal Services

Cost of engineering services may include special investigations, surveys, foundation explorations, location of interfering utilities, detailed design, preparation of contract drawings and specifications, general construction assistance, on-site construction inspection, materials testing, final inspection, pumping station start-up services, operator training, and preparation of record drawings. Total costs for engineering services typically range from 12 to 30 percent of total construction cost, depending on the complexity of the design and the overall construction cost of the project. The lower percentage applies to projects relatively simple or repetitive in nature. The higher percentage applies to projects that require a great deal of preliminary investigation or permitting or that involve considerable remodel work.

Other costs directly associated with the cost of constructing facilities include administrative, financial, and legal services; costs associated with bond sales; and interest on borrowed money. Administrative and legal proceedings can represent a significant cost for the formation of special utility districts, for preparation of intergovernmental agreements, and for O&M contracts. Based on past experience, allowances for administrative, financial, and legal services range from 5 to 10 percent of construction cost.

5.2.4 Land and Right-of-Way Costs

Land and right-of-way costs are part of the capital investment the City must account for in implementing the recommended plan. The amount of land needed will vary with the specific site identified for a facility or the location of a pipeline route. No additional land will be needed for most pipeline alternatives. As a general rule, most new gravity sewers will be located in the existing City of Grants Pass, Josephine County, or Oregon State rights-of-way. In the case of county and state rights-of-way, the City may need to obtain either utility permits for crossings or utility franchises for pipelines paralleling a state road.

Land and right-of-way costs have not been included in the cost estimates for recommended facilities. These costs are important and should be considered at the earliest possible time, both for property acquisition and for budgeting purposes. It is recommended that these costs be identified during the budget level stage of project development.

5.2.5 Total Project Costs

A breakdown of a percentage of total project cost components is shown in Table 5-2.

Table 5-2. Percentage of Total Project Cost Components

Cost Item	Percent
Construction	100
Contingency	40
Total Construction Cost	130
Engineering, Legal, and Administration	30
Total Project Cost[a, b]	182

[a] Does not include land or right-of-way acquisition costs.

[b] Total project costs were calculated by multiplying estimated construction costs by 1.82 to allow for contingencies, engineering, legal, and administrative costs.

5.3 PERFORMANCE CRITERIA

Performance criteria relate to the way each project achieves the City's objectives and how well the project is expected to function. Performance factors include reliability, effectiveness, flexibility, and public health protection. Each of these criteria was used to develop the weighting factors used in the Maintenance and Reliability Analysis presented in Section 7 and also to prioritize the Pipeline Improvement and Structural Repair Projects identified in Sections 6 and 7. Each of the criteria is briefly described as follows:

- **Reliability** – Assurance of design performance is the overall screening consideration for reliability. This evaluation considers the relative risk of mechanical failure; susceptibility of the alternative to disruption from natural catastrophes; and consequences of functional system failures, regardless of cause. The preference is to provide sewer service to new or existing customers using new gravity flow sewers. Only in the event that gravity flow sewers cannot be made available will consideration be given to either a lift station or pump station with a force main(s), in this order of preference. Only as a last resort will consideration be given to individual grinder pumps and force mains to provide sewer service.

- **Effectiveness** – NPDES compliance is the primary factor to be evaluated under effectiveness. The relative ease of achieving compliance is a primary consideration; however, the ratio of expected performance to total cost is also considered in this evaluation.

- **Flexibility** – Evaluation of flexibility considers responsiveness of the alternative to new land-use plans, development patterns, and lifestyle changes such as water and energy conservation.

- **Public Health Protection** – Each project was evaluated in terms of potential risk to public health. In terms of regulatory requirements that pertain to this study, the Oregon "bacteria rule" has special significance in the evaluation of existing and future collection system components. In 1996, Oregon adopted amendments to the state water quality standard for bacteria that address the frequency of SSOs. Those amendments stipulate that SSOs must be prohibited except during 1-in-5-year, 24-hour-duration storm events (or greater) between November 1 and May 21. During the rest of the year, SSOs must be prohibited except during 1-in-10-year, 24-hour-duration storm events (or greater).

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Hydraulic Analysis



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6. HYDRAULIC ANALYSIS

The hydraulic analysis performed for this Plan consisted of three steps: (1) gathering flow data through a flow monitoring program and data collection at the Water Restoration Plant; (2) developing a calibrated computer model of the collection system; and (3) analyzing the system hydraulics to identify system deficiencies and develop an understanding of the need for collection system improvements during the next 20 years.

6.1 FLOW MONITORING PROGRAM

A 2-month (December 12, 2002, to February 14, 2003) flow monitoring program was conducted to provide accurate flow data for hydraulic model input and to calibrate the model. Six flow monitors were installed within the City's service area at six manhole locations (see Figure 6-1 [page 6-2] and Table 6-1) selected to best represent flows from major drainage basins, areas of known deficiencies, and areas of anticipated growth. The flow monitoring program also included collection of rainfall data by a rain gauge located at the City Water Reclamation Plant (Water Treatment Plant, [WTP]). Flow monitoring methods, equipment, and results are presented in a report submitted to the City in April 2003 (Geotivity, Inc., 2003).

Table 6-1. Flow Monitoring Manhole Location

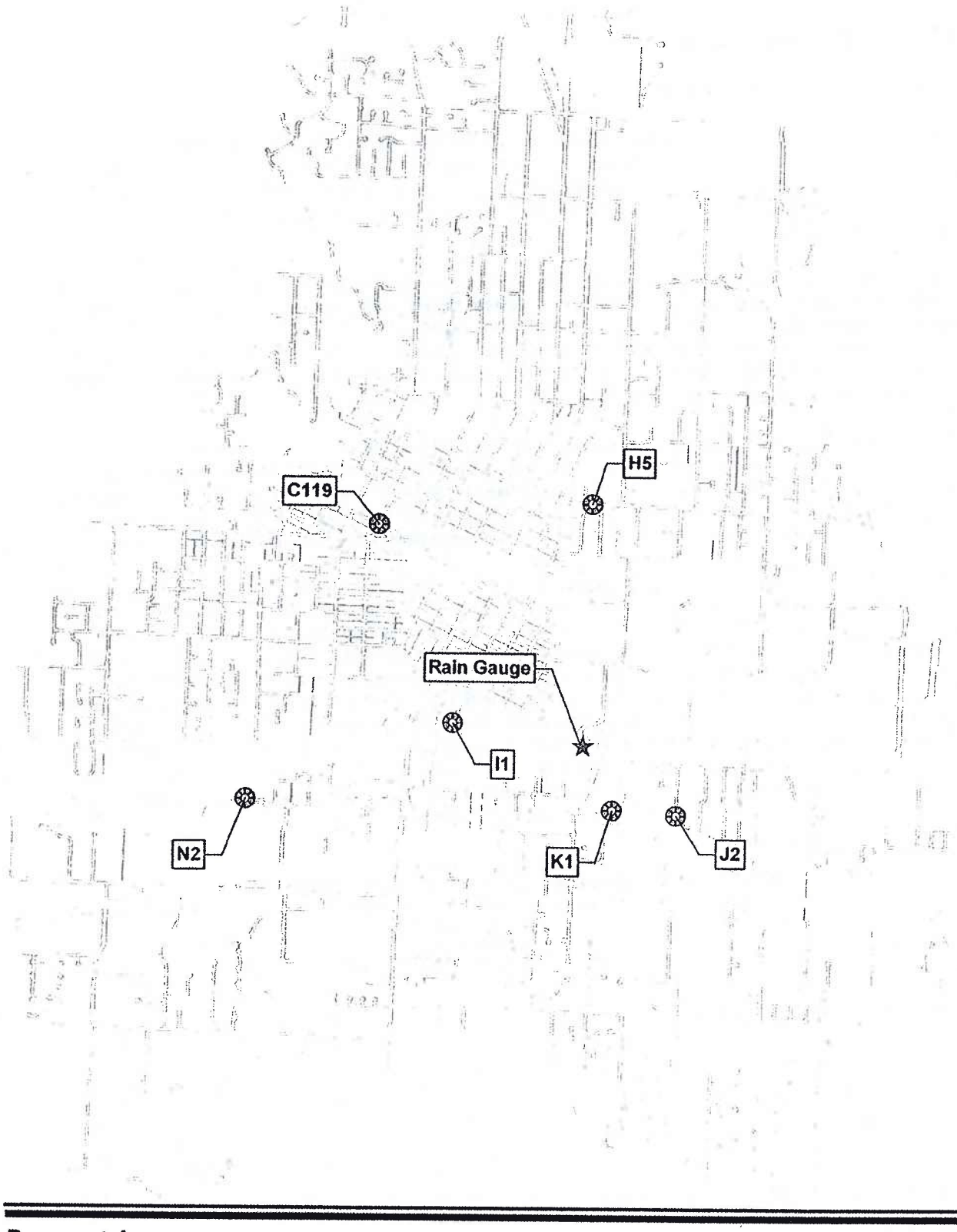
Basin	Monitored Manhole	Location Description	Monitored Pipe Diameter (inches)
F	C119	Booth Street between F and G Streets	12
H	H5	East of the intersection of A Street and Dean Drive	12
I	I1	Near intersection of 8th Street and Rogue View Lane	12
J	J2	Near the south end of Belle Aire Drive	18
K	K1	Gold River Lane	18
N	N2	West Park Street	18

6.1.1 Rainfall Data

Rainfall data were collected throughout the flow monitoring program from a single rain gauge installed at the WTP. The WTP is near the center of the distribution of flow monitoring locations; therefore, any potential spatial errors resulting from using a single collection point should be spread fairly equally among the flow monitoring sites.

Rainfall data collection began on December 2, 2002, and extended until February 18, 2003. This data collection period followed closely behind a fall season during which rainfall was below average. The two most significant rain events occurred during the first month of the monitoring program. A 2-day storm (Storm 1) began the day the rain gauge was activated. A more significant storm (Storm 2) occurred about 10 days later. Storms 1 and 2 measured 2.9 inches and 4.4 inches of rain, respectively. Although additional storms were recorded throughout the monitoring period, Storms 1 and 2 produced the most significant flow monitoring results.

000760



Parametrix Date 09/28/04 10:08am

SCALE IN FEET
0 1,250 2,500

N

LEGEND

- Flow Monitoring Manhole
- Rain Gauge Location (Water Treatment Plant)

Figure 6 - 1
Flow Monitoring Manholes

CITY OF GRANTS PASS
COLLECTION SYSTEM
MASTER PLAN

6.3 HYDRAULIC ANALYSIS

6.3.1 Model Development

The collection system schematic was developed in HYDRA (Pizer Incorporated, 2000), a computer model used for the analysis of wastewater collection systems. HYDRA has been used throughout the United States to provide useful information regarding the location of hydraulic constrictions, general response to rainfall related flow, and reliable guidance on system improvements for analyses such as those included in this Plan.

HYDRA is an analysis tool that automates the extensive, and often tedious, calculations required for traditional collection system design. Collection system design data, such as pipeline length, pipeline diameter, and manhole elevation, were gathered from the City's GIS database and as-built drawing archives. These data were used to develop the model schematic of the collection system. Pipelines serving only a local collection role were not included to reduce the effort required to construct the model and analyze results. Pipelines included in the model are as shown in Figure 6-9.

The capacity of each pipeline is determined from the model input data by means of the Manning's Equation (open channel flow) and the Hazen & Williams Equation (pressure flow). The pipe capacity is then compared to the predicted flow to determine if the existing pipe is adequate for the given conditions and desired service area. The HYDRA model also calculates the hydraulic grade line, reports the extent of surcharging for over-capacity pipelines, and the upstream impact of limited downstream capacity.

The population projections used in this work are shown in Table 6-2. Modeled flows were generated from the combination of wastewater and rain-related flows. Wastewater flows consisted of separate estimates for flow from residences, businesses, and industrial areas that were derived from the land use analysis presented in Section 2. Modeled wastewater flows were developed using City GIS data and unit flow factors for land use. Current land use was used to build a present day basis for the model which was then calibrated using flow monitoring date (see Section 6.3.2). This base model was then modified by increasing service area population in accordance with the population projections presented in Table 6-2. In addition to population projections for Year 2025, a projection was created assuming all available land was at saturation levels of development, and this was referred to as Year 2060. The 2060 projections provide a good representation of the build-out conditions that are typically used to determine the size of long-term infrastructure improvements, such as collection system pipelines.

Table 6-2. Population Estimates Used in the HYDRA Model

Area	1997	Average Growth Rate (percent/year)	2003	2025	2060
City Limits	20,526	1.5	22,444	31,143	52,440
Harbeck-Fruitdale (ref Area N, M, K)	4,200	1.6	4,620	6,550	11,417
Redwood	4,758	3.1	5,714	11,186	32,563
Comm./Ind. Equivalent	10,319	1.8	11,472	17,107	33,747
Total EQ Population:	**39,803**		**44,250**	**65,986**	**130,167**

The distribution of the year 2060 population is shown in Table 6-2; however, at that time, it would be likely that the city limits and UGB would have enlarged from today. The Redwood and Harbeck-Fruitdale populations would probably then be reflected in the city limit population.

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As discussed in Section 3, Oregon has adopted a rule (the "Bacteria Rule") that limits the maximum frequency with which SSOs can occur. In general, SSOs must be prevented during wintertime rainfall events that are less than the 5-year, 24-hour rainfall event for an area. According to the National Oceanic and Atmospheric Administration's (NOAA) weather atlas for Oregon, the statistical 5-year, 24-hour rainfall event for the City is 3.5 inches. This is only about 18 percent higher than the greatest 24-hour accumulation that occurred during Storm 2. Therefore, flows occurring during Storm 2 are an important measure of collection system capacity under high-flow conditions, especially at the regulatory threshold defined by the Bacteria Rule.

6.1.2 Flow Monitoring Data

Flow monitoring was generally successful. However, there are several factors that affected the interpretation of the flow monitoring program data. These factors include limited lapses in data due to equipment failure, diversion manholes within the collection system that dampen peak flows during rain events, and low groundwater levels due to abnormally dry periods prior to flow monitoring. The flow diversions and low groundwater levels result in data that tends to underestimate peak flows during rain events.

A summary of flow monitoring results is presented below. For a complete discussion on flow monitoring results and graphs of all monitored flow, refer to *Grants Pass A/V Flow Study Report* (Geotivity, 2003).

6.1.2.1 Basin F (Manhole C119)

Wastewater flow in Basin F was monitored at Manhole C119, and a portion of this flow monitoring data is shown in Figure 6-2 (page 6-4). Basin F is one of the most critical (largest and oldest) basins in the collection system. While Basin F is fairly well developed, there is still capacity for growth through new development on the northern and western margins of the basin, and through urban renewal near the downtown core. The City has constructed a pipeline diversion to alleviate SSOs that have occurred in Basin F. The extent of pipeline corrosion in the basin is generally high, and some pipe sections have collapsed causing sink holes, notably along Pine Street. In general, the City's operations staff spend a high level of effort and funds maintaining pipelines in Basin F.

Manhole C119 is actually at the upstream end of Basin C since the manholes at the downstream end of Basin F were unsuitable for flow monitoring equipment. Basin F flow data are further impacted by a diversion that shunts flow from Basin F to Basin E during high flows. Therefore, measured peak flows are lower than would otherwise be observed without the diversion.

A peak flow of about 11 cfs was recorded during Storm 2. Flow velocity is between 2 and 3 feet per second (fps). One would expect that minimum nighttime velocity would approach zero, as occurred for the other basins included in the monitoring program. The sustained velocity indicates that there is appreciable flow due to infiltration and inflow (I/I).

6.1.2.2 Basin H (Manhole H5)

Wastewater flow in Basin H was monitored at Manhole H5 and a portion of this flow monitoring data is shown in Figure 6-3 (page 6-5). According to City records, Manhole H5 often surcharges and occasionally overflows. Overflows at other manholes immediately upstream from Manhole H5 have also been recorded, indicating that there is a hydraulic bottleneck in the vicinity of A Street and Dean Drive that is severe enough to influence the hydraulics of the upstream collection system.

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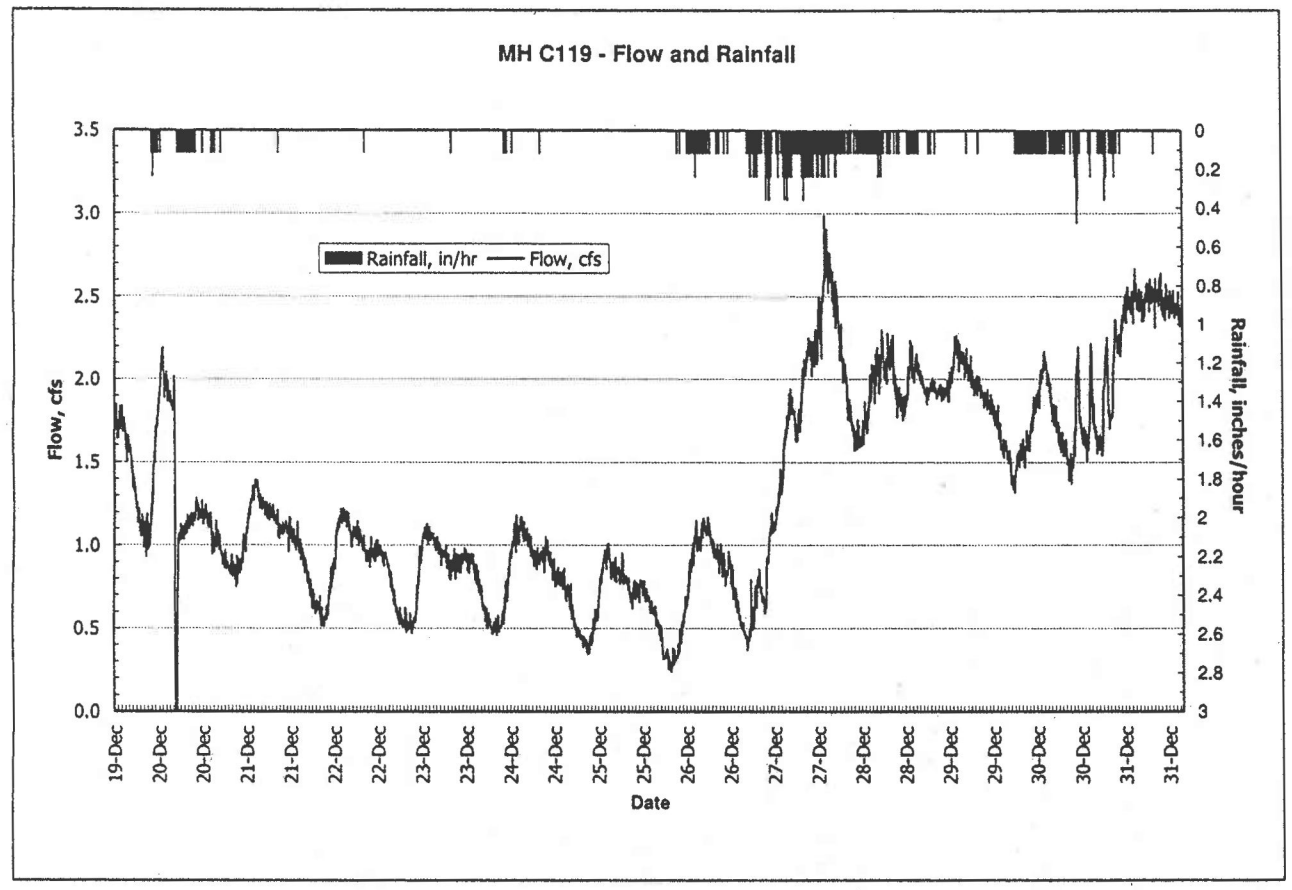

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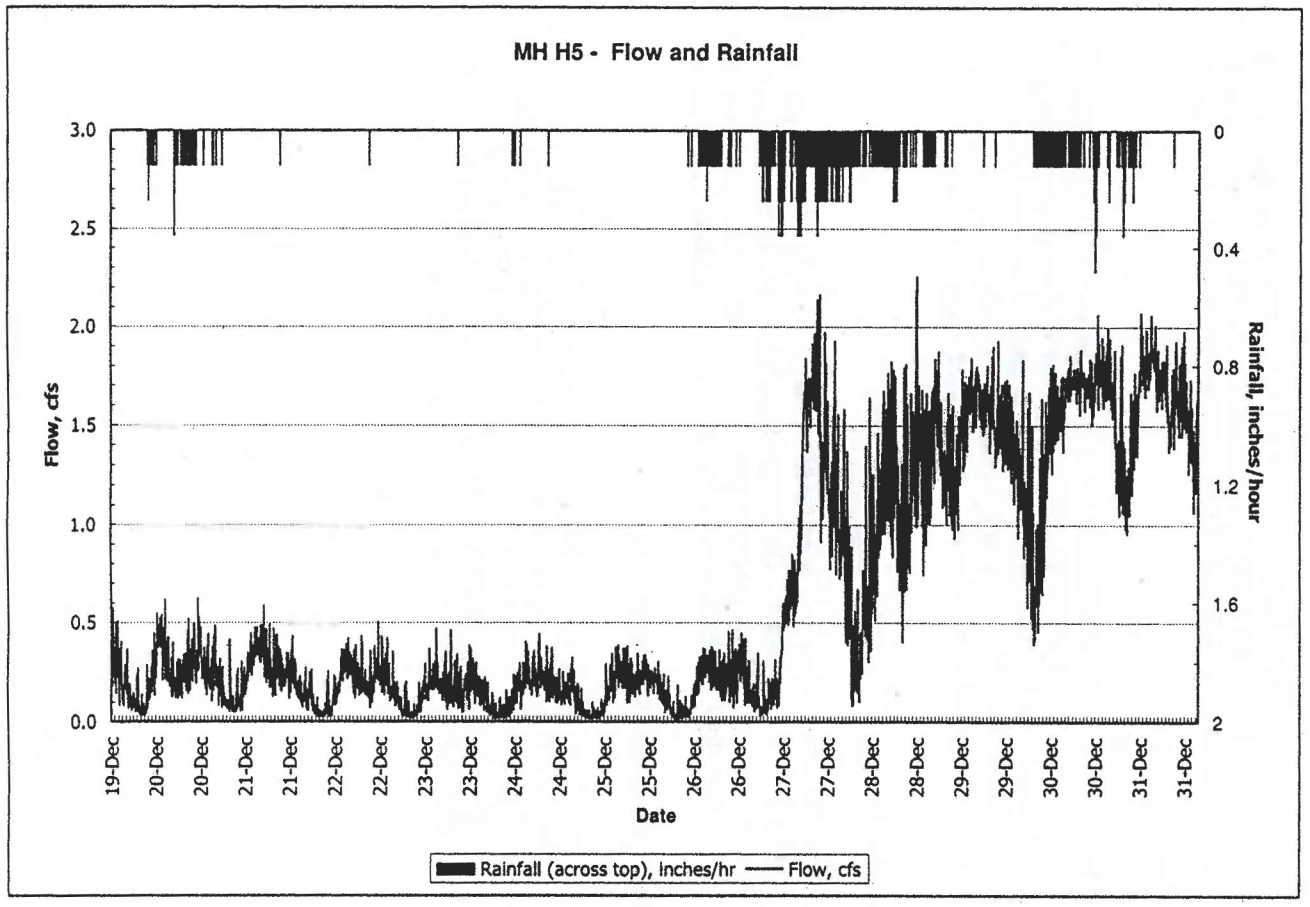
Parametrix

Figure 6-2
Flow Monitoring Data
Manhole C119



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Parametrix

Figure 6-3
Flow Monitoring Data
Manhole H5

S92 765

Sewage flow depth data collected at Manhole H5 indicate that pipes connected to the manhole became surcharged for a short time during Storm 1. Given the history of this manhole as a source of overflows, it is believed that the data collected during Storms 1 and 2 represent a typical Basin H response to high rainfall events.

6.1.2.3 Basin I (Manhole I1)

Flow monitoring data collected from Manhole I1 (a portion of data is shown in Figure 6-4, page 6-7) indicate inflow or shallow lateral sources, but no significant groundwater infiltration during storm events. In general, some infiltration is present at all times, but the basin appears to drain well at depth.

6.1.2.4 Basin J (Manhole J2)

Flow monitoring data collected from Manhole J2 (a portion of data is shown in Figure 6-5, page 6-8) suggest that groundwater infiltration is not a significant concern for the basin. Inflow and infiltration from shallow sources, such as leaking service laterals, likely contribute most of the extraneous flow.

6.1.2.5 Basin K (Manhole K1)

Wastewater flow in Basin K was monitored at Manhole K1, and a portion of data is shown in Figure 6-6 (page 6-9). Data collected from Manhole K1 indicate a strong presence of groundwater in the basin (base flow increased rapidly with precipitation). However, the diurnal pattern of flow was relatively unaffected by rainfall except for the significant storm events (Storms 1 and 2). This indicates that there are relatively few sources of extraneous flow at the surface. This is to be expected since most pipes in the basin are relatively new.

6.1.2.6 Basin N (Manhole N2)

Basin N was the only basin for which flow was monitored on the south side of the Rogue River (a portion of data is shown in Figure 6-7, page 6-10). The data collected from Manhole N2 indicate that the system is in good condition since there was little to no nighttime flow recorded. The quick and brief flow response to rainfall may be an indication of illegal storm drain connections or leaking service laterals.

6.2 WATER RESTORATION PLANT FLOWS

During flow monitoring, the total collection system flows were measured at the WRP using a 48-inch-diameter ultrasonic flow meter. The flow meter is located at the effluent end of the plant, just upstream of the UV disinfection process. The 24-hour average flow, as well as the peak and minimum instantaneous flow rates for each day, are recorded by the plant SCADA system. This information is included in monthly monitoring reports submitted to ODEQ. These data are shown in Figure 6-8 (page 6-11).

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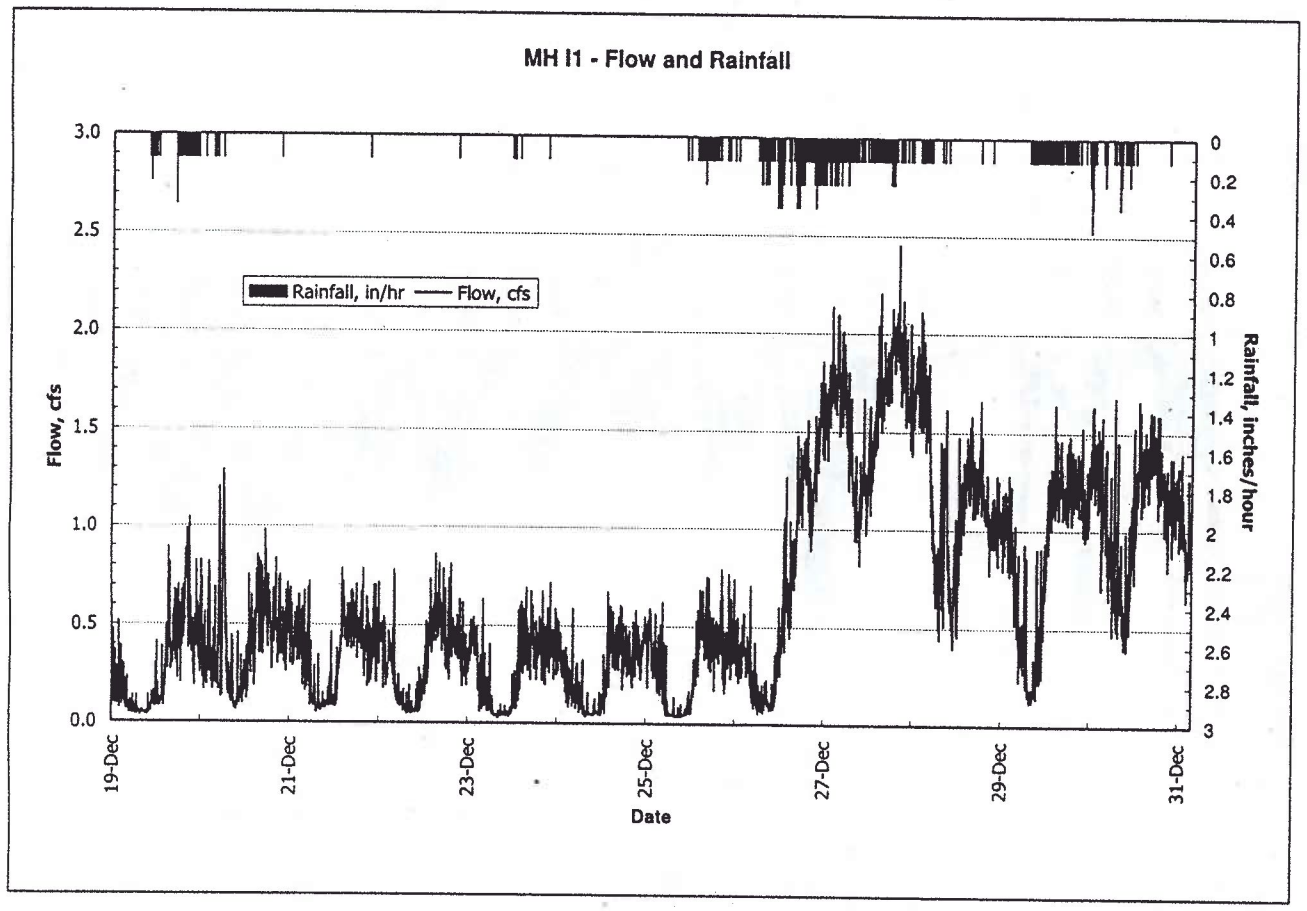
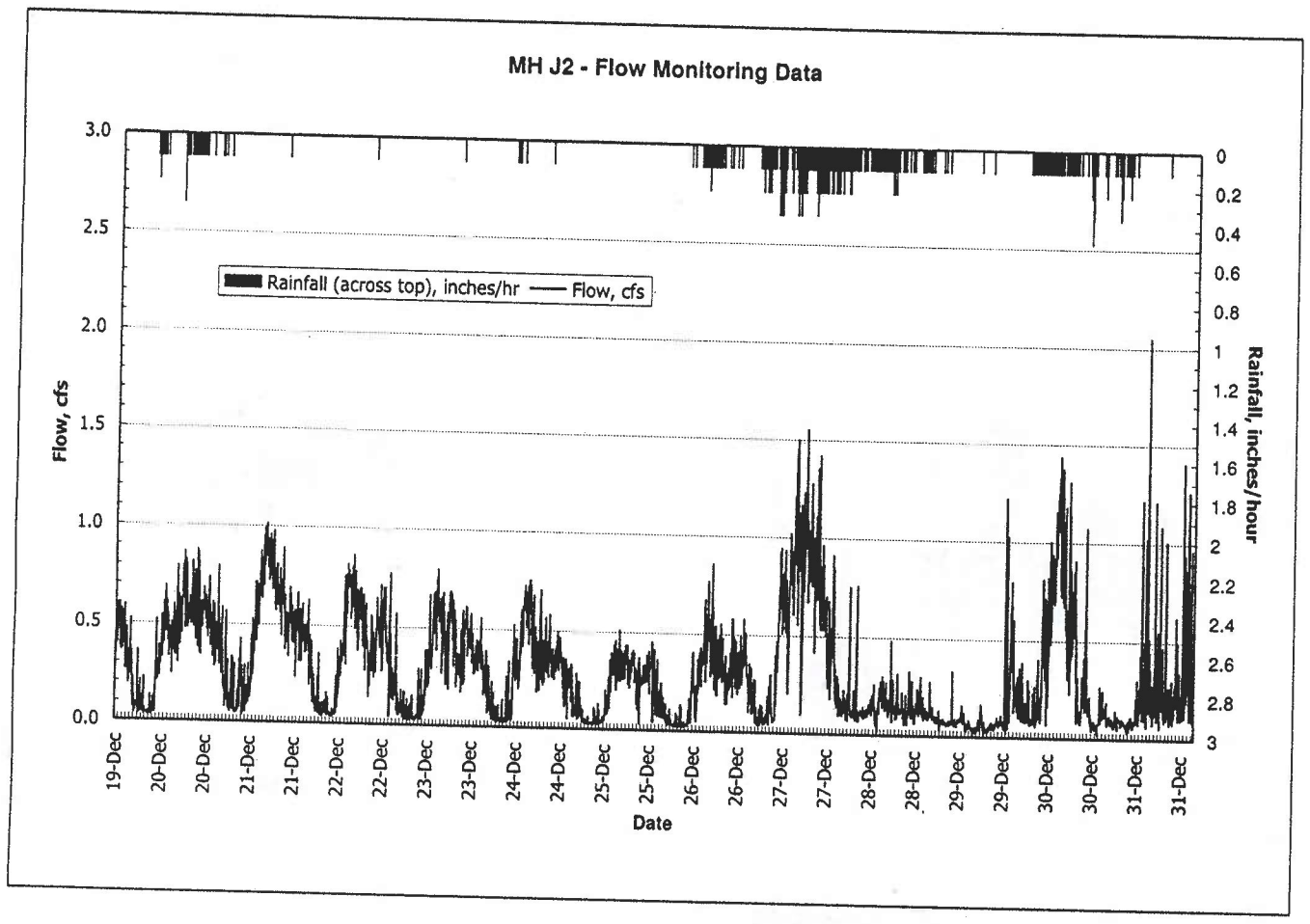


Figure 6-4
Flow Monitoring Data
Manhole I1

792 767

892006



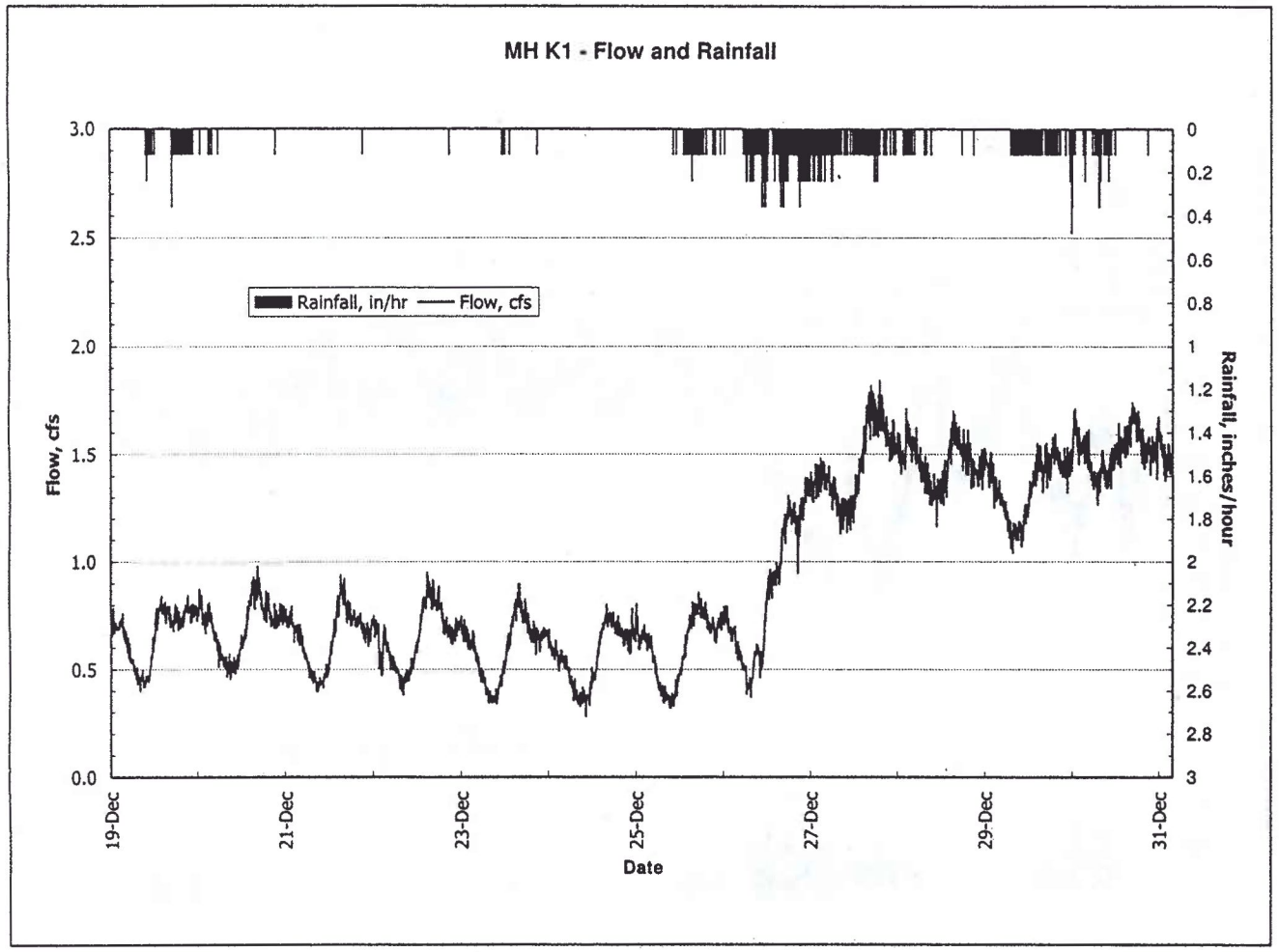
CITY OF GRANTS PASS

COLLECTION SYSTEM MASTER PLAN

Parametrix

Figure 6-5
Flow Monitoring Data
Manhole J2

276-3416-025 (03/0350)



CITY OF GRANTS PASS

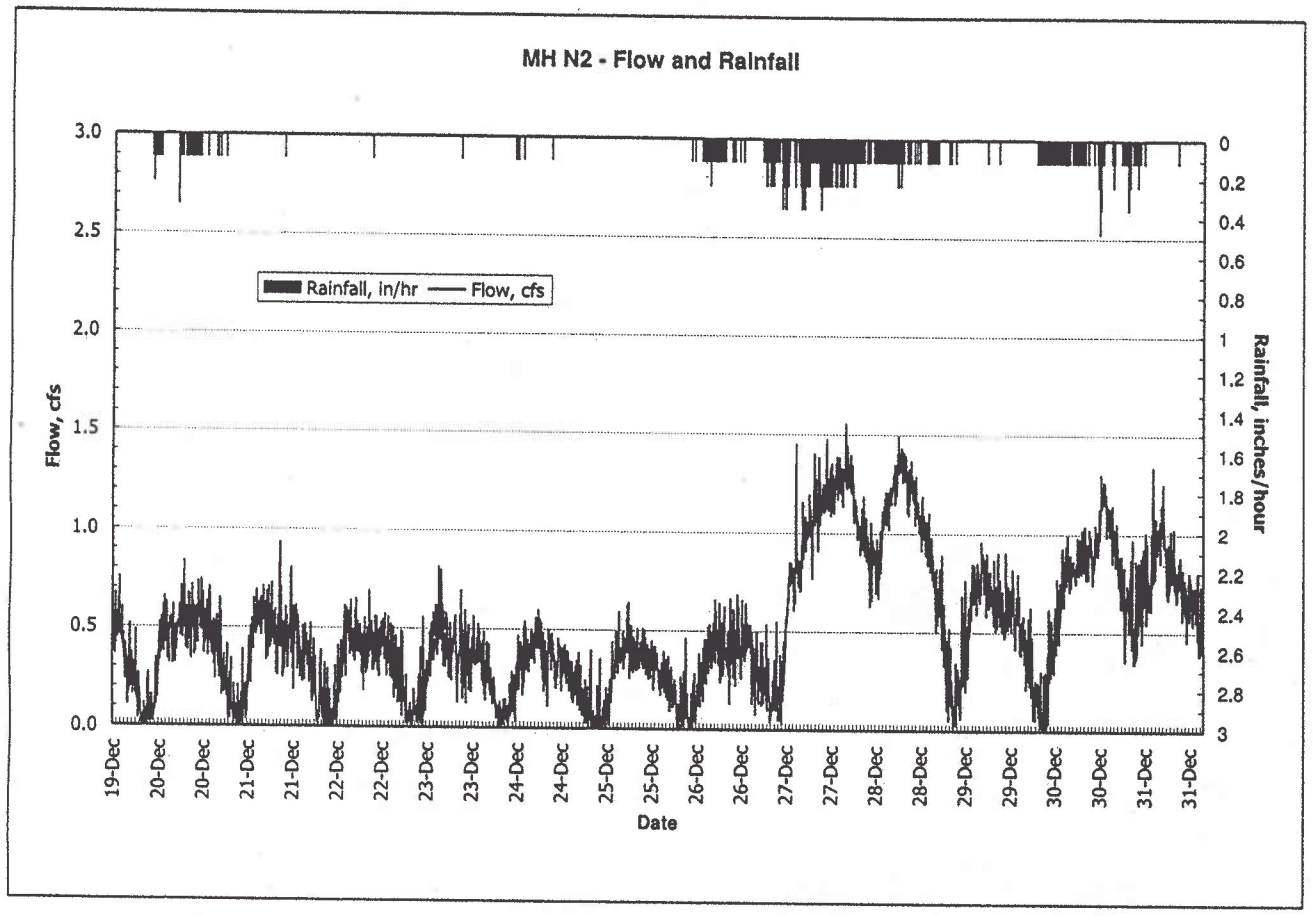
COLLECTION SYSTEM MASTER PLAN

Parametrix

Figure 6-6
Flow Monitoring Data
Manhole K1

6927 08

0227006



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Parametrix

Figure 6-7
Flow Monitoring Data
Manhole N2

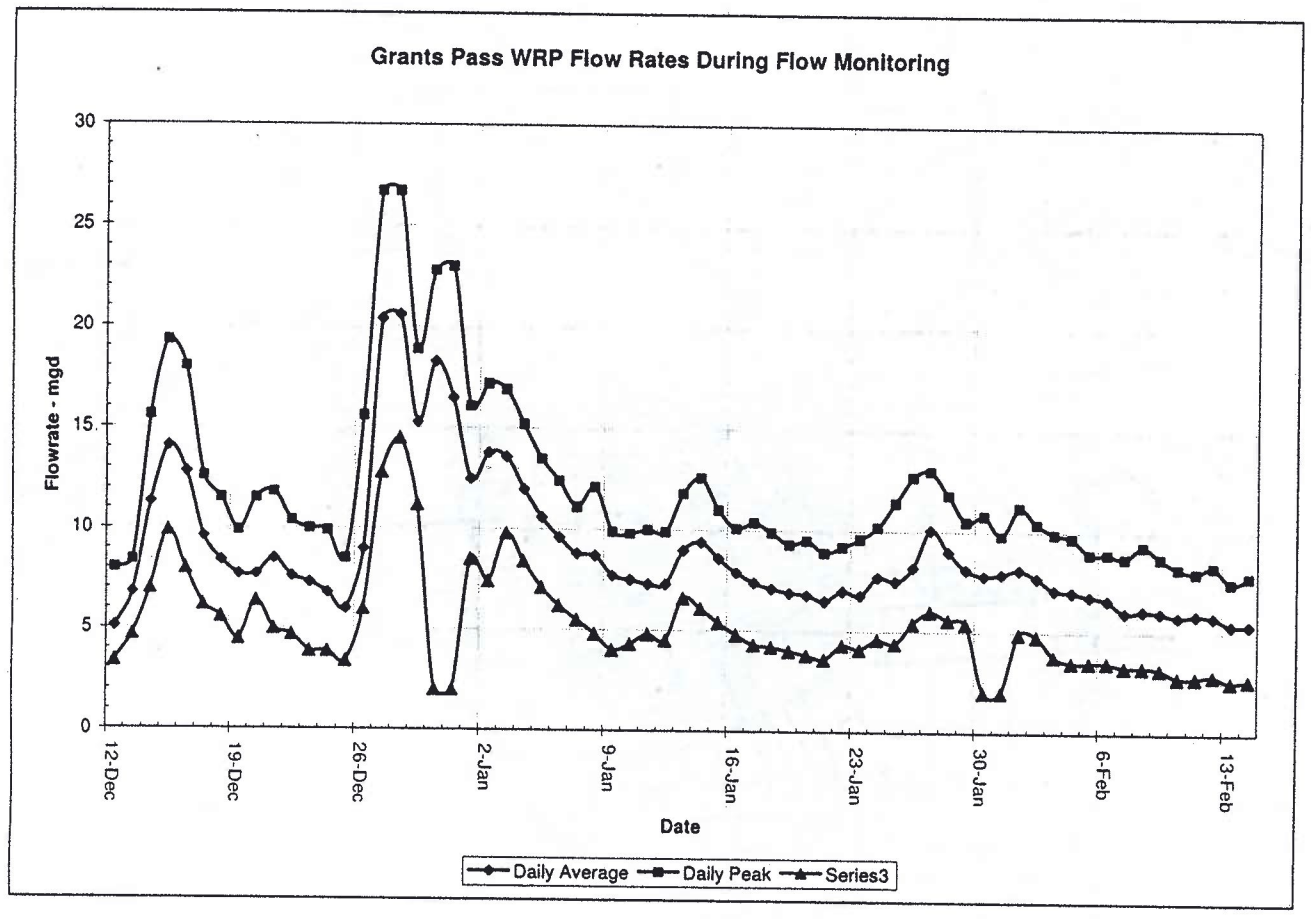

Figure 6-8
WRP Flow Rates During Flow Monitoring (December 12, 2002 to February 14, 2003)

T22'08

24L006



CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

LEGEND

PIPELINES MODELED

SCALE IN FEET
0 1000' 2000'

N

Figure 6-9
Pipelines Included in the Hydraulic Model

6.3.2 Model Calibration

Following initial development, the collection system model was calibrated using flow and rainfall monitoring data (see Section 6.1). WRP flow records were used to supplement the collected data since only a select number of City drainage basins were included in the flow monitoring program. Where applicable, peak basin flows were adjusted to match the peak flows that occurred during Storm 2. In general, the calibration process consisted of holding the wastewater flow component relatively constant while varying rainfall related flow components as necessary to match recorded flow.

In general, to verify the calibration process, the modeled flows at each flow monitoring manhole were compared to actual flow monitoring results. The results of this process are shown in Figures 6-10 through 6-15 (beginning on page 6-16). Each of the different components of the modeled flow (sanitary, infiltration, storm, miscellaneous) is shown in each of these figures. Each of these figures also shows the total modeled flow and measured flow for comparison. Based on these comparisons, the hydraulic model of the collection system was determined to be sufficiently calibrated so that future flow conditions could then be modeled.

6.3.3 Model Simulations and Results

The collection system model was modified to better serve as an analysis tool following calibration. Modifications included increasing the volume of Storm 2 by 18 percent to match the 5-year, 24-hour storm volume for the Grants Pass area and merging projected land use data for years 2025 and 2060 so that the effects of growing population could be represented. Modifications also included eliminating active diversions (see Section 4.1 and Figure 4-5). Since the goal of this Plan is to provide the City with a long-term plan for collection system improvements, it is useful to gauge the impact of removing existing diversions and sizing new pipelines on the basis of build-out conditions for each drainage basin.

In all, six models were created for the analysis in this Plan; a set of three models based on Year 2003, 2025, and 2060 land use projections for which all existing diversions were continued; and a set of three corresponding models with all diversions removed. For each of the six models, a threshold of performance was established to be 2 feet of surcharge for any pipeline in the system during wet weather conditions simulating a 5-year, 24-hour rainfall event. The 2-foot surcharge criterion was selected to provide guidance on which pipelines in the collection system may need replacement while providing some factor of safety that was neither too restrictive (thereby resulting in a recommendation to replace too many pipelines to assure good performance), or too lax (thereby resulting in recommendations without addressing potential inaccuracies in predicted flow). This level of surcharge was deemed an acceptable compromise between the SSO constraints set forth in the "Bacteria Rule" and the economic impact of too many required collection system improvements.

Model simulation results are presented in Figures 6-16 through 6-19 for the following situations/ assumptions:

- Figure 6-16 (page 6-21) – North Service Area with all Diversions Active.
- Figure 6-17 (page 6-22) – South Service Area with all Diversions Active.
- Figure 6-18 (page 6-23) – North Service Area with Diversions Removed.
- Figure 6-19 (page 6-24) – South Service Area with Diversions Removed.

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The coloring convention, identical for Figures 6-16 through 6-19, is a guide for determining when a pipeline is modeled to surcharge by 2 feet or more. Pipelines that are colored blue are predicted to become surcharged 2 feet or more under existing flow conditions. Similarly, the depth of flow in pipelines colored green was modeled to not exceed surcharge criterion until approximately 2025. Finally, pipes colored red were modeled to flow with less than 2 feet of surcharge until land use approaches were estimated for 2060.

As expected, removing diversions exacerbates surcharge conditions. The impact of diversion removal can be reduced by providing a larger pipeline diameter. Since a significant portion of pipeline replacement cost is associated with earthwork, providing a pipeline that is larger by one or two standard diameters presents only a marginal increase in construction cost. In some cases, removing diversions can avoid new pipeline construction in areas of the city where such construction would be extremely disruptive. A good example of this is the removal of the diversion at Manhole F69. This diversion shunts high flows from Basin F into Basin E that currently has excess capacity. A new pipeline will be required in the Pine Street area of Basin F regardless of the removal or continued operation of the diversion. A larger pipeline will be required for Pine Street if the diversion is removed, but continued operation of the diversion will eventually require increased pipeline capacity for Basin E. Removing the upstream diversion will allow the City to consider repair options for Basin E that do not require extensive and disruptive surface excavation.

The modeled surcharge conditions presented in Figures 6-16 through 6-19 form the basis, along with the Maintenance and Reliability Analysis in Section 7, for collection system improvement recommendations presented in Section 8.

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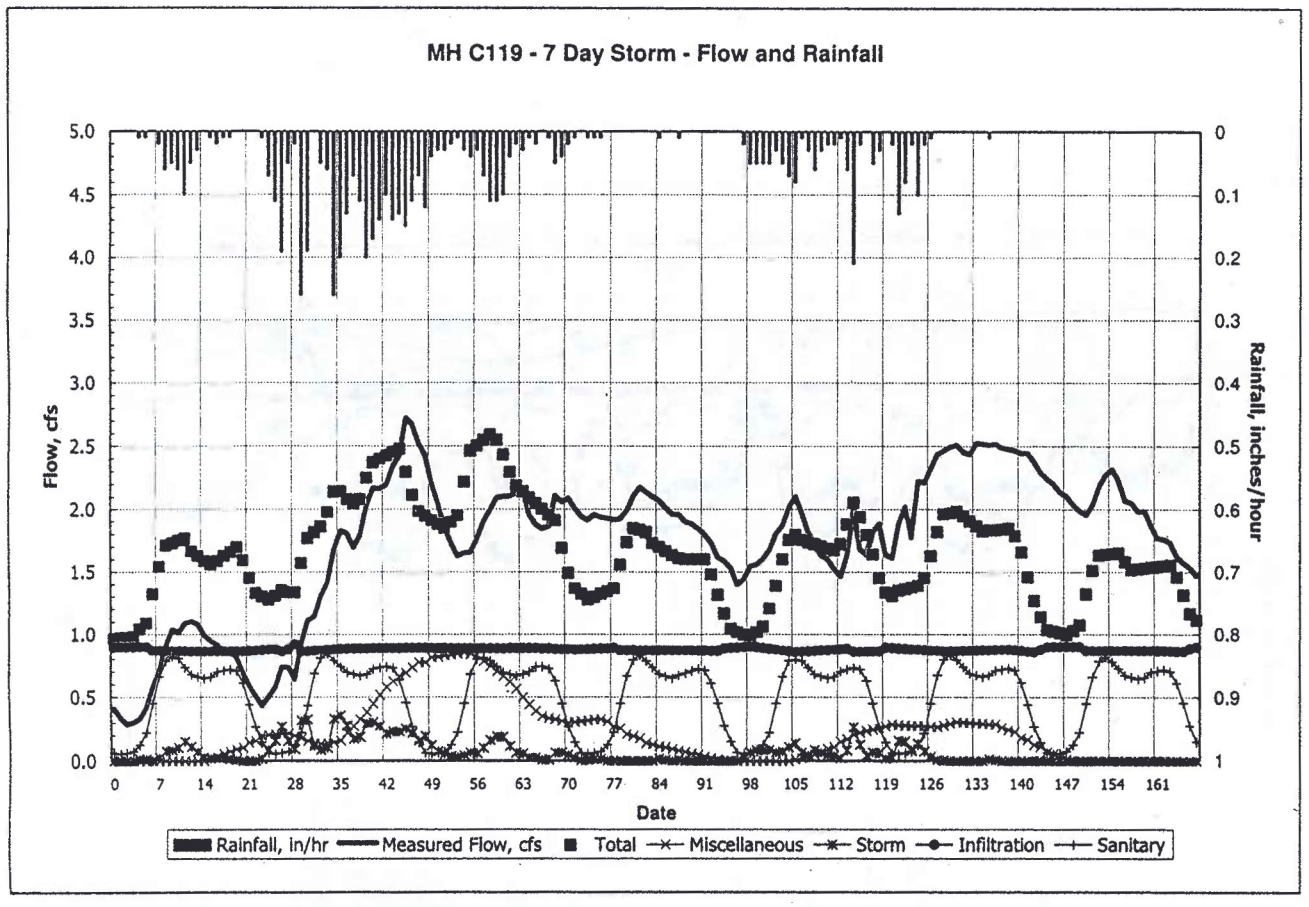

Figure 6-10
Flow Monitoring Data and Hydraulic Model Predicted Flow Manhole C119

S44 00

942006

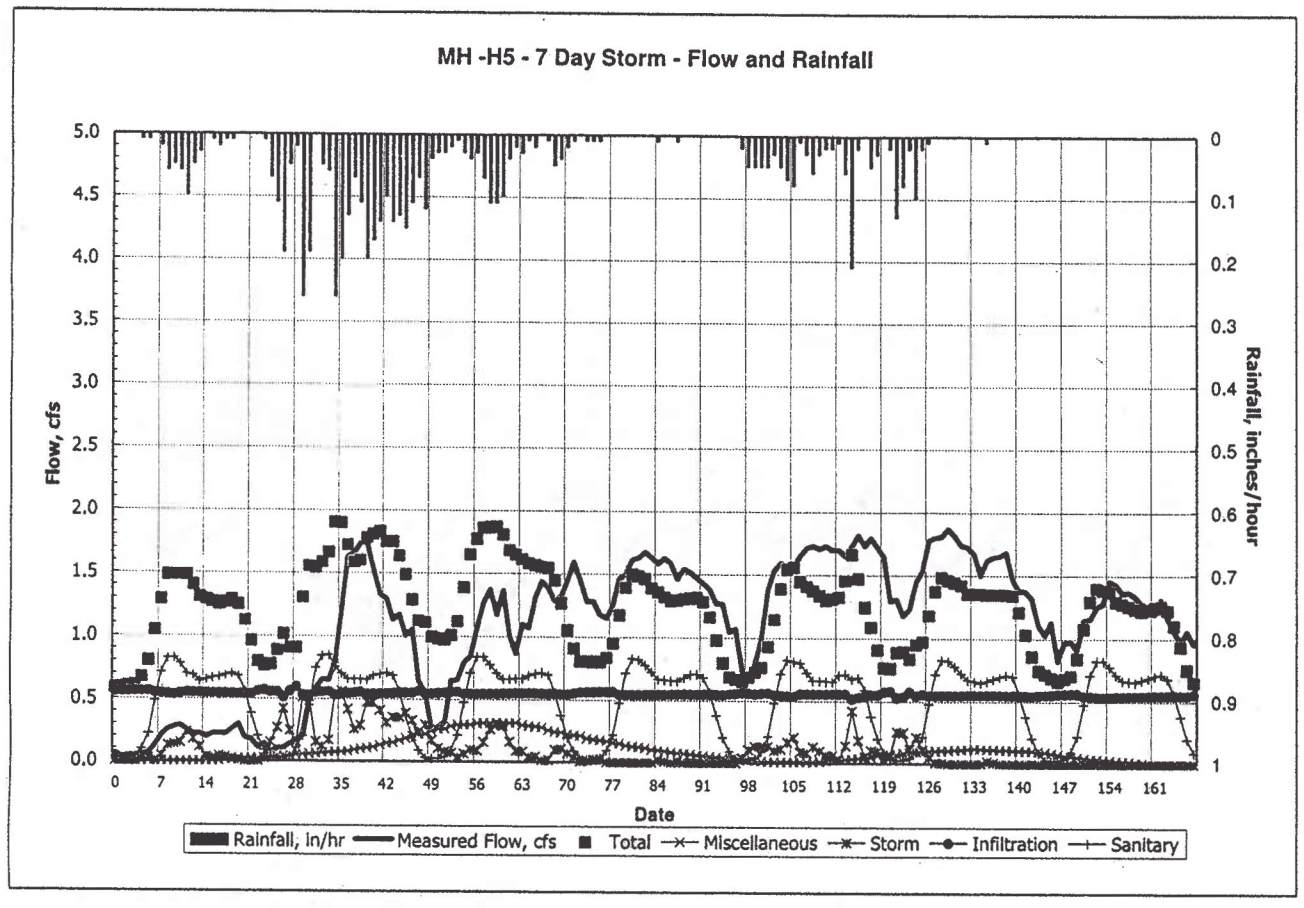

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Figure 6-11
Flow Monitoring Data and Hydraulic Model Predicted Flow Manhole H5

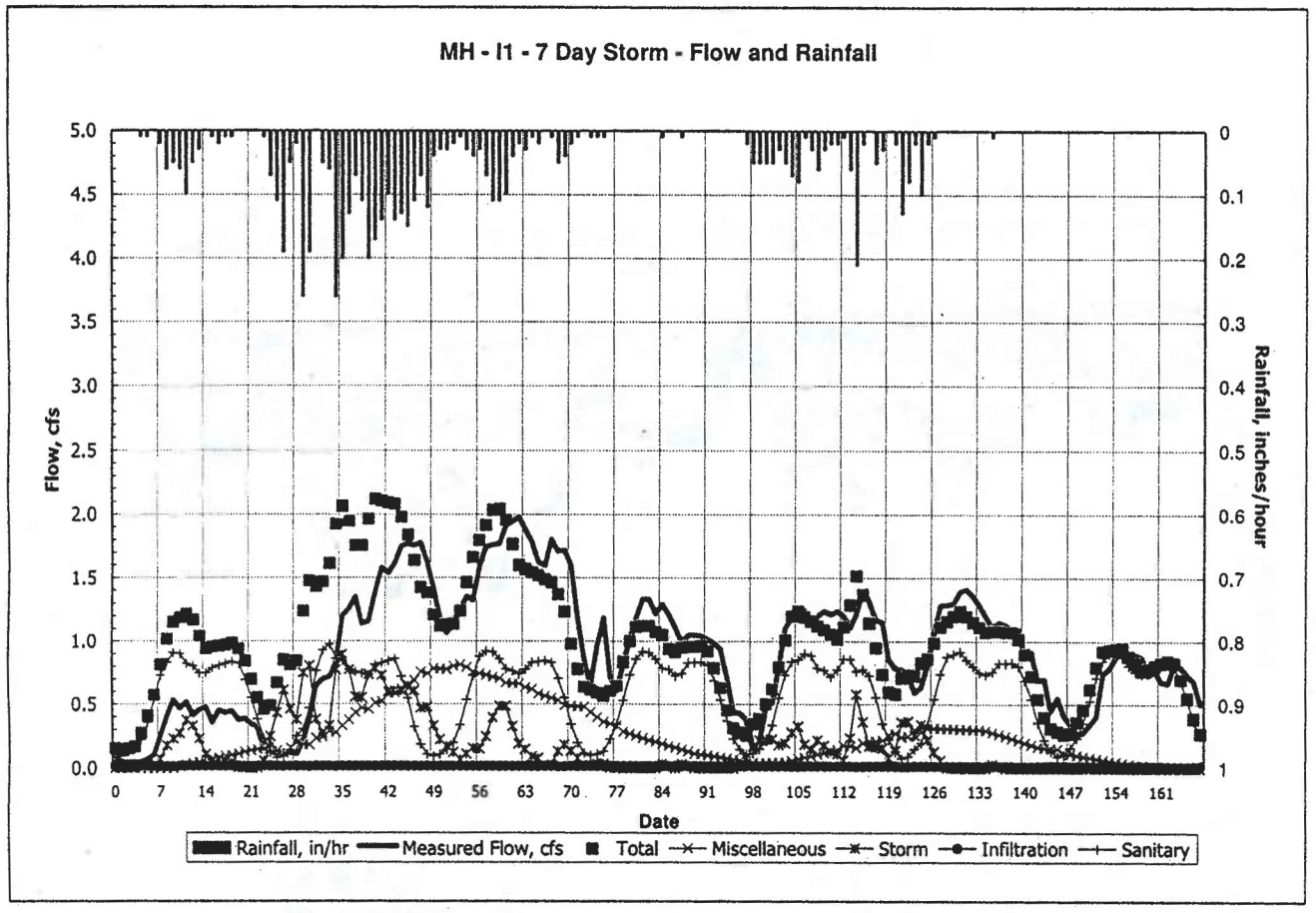

Figure 6-12
Flow Monitoring Data and
Hydraulic Model Predicted Flow
Manhole I1

8220778

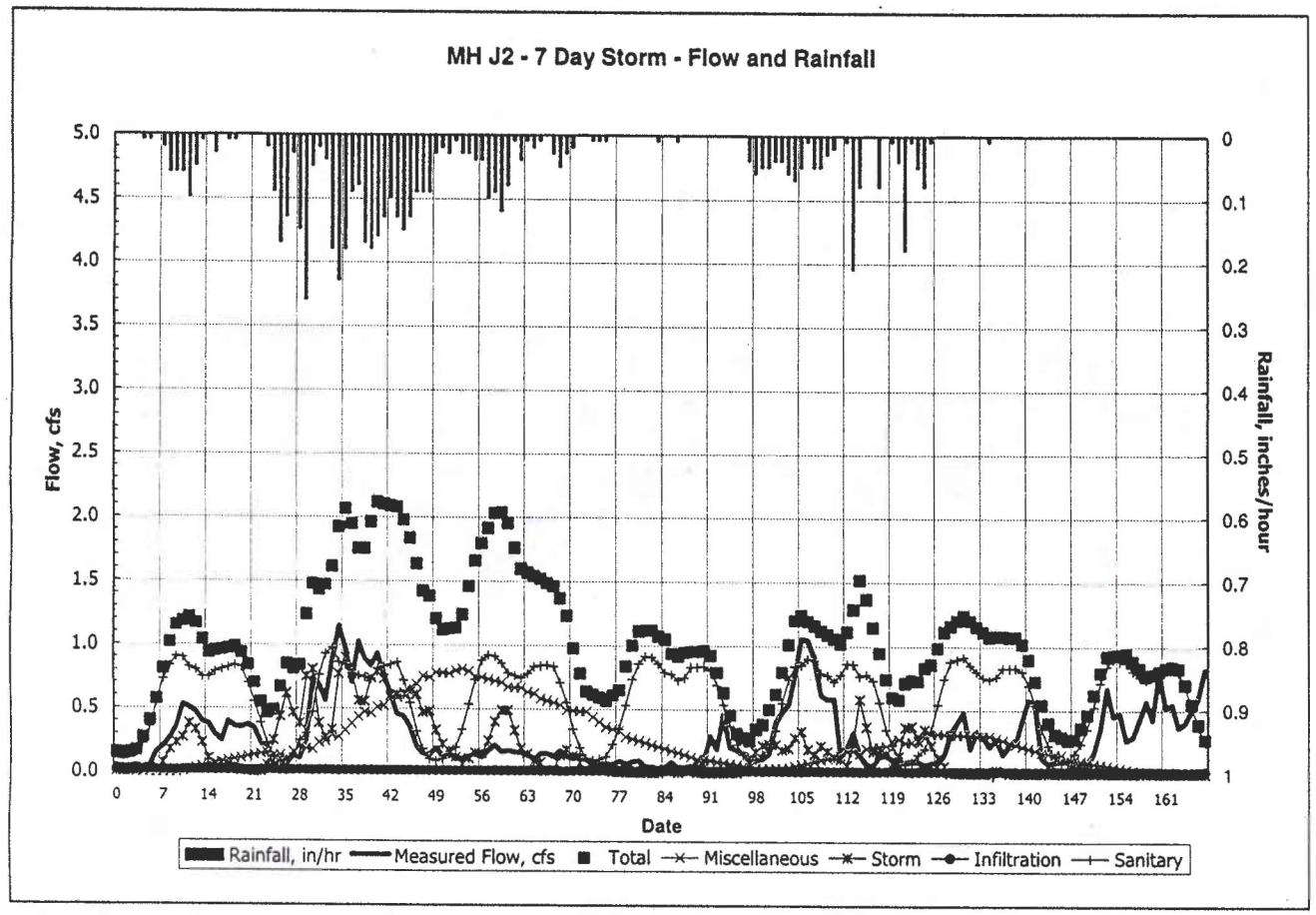

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Figure 6-13
Flow Monitoring Data and Hydraulic Model Predicted Flow Manhole J2

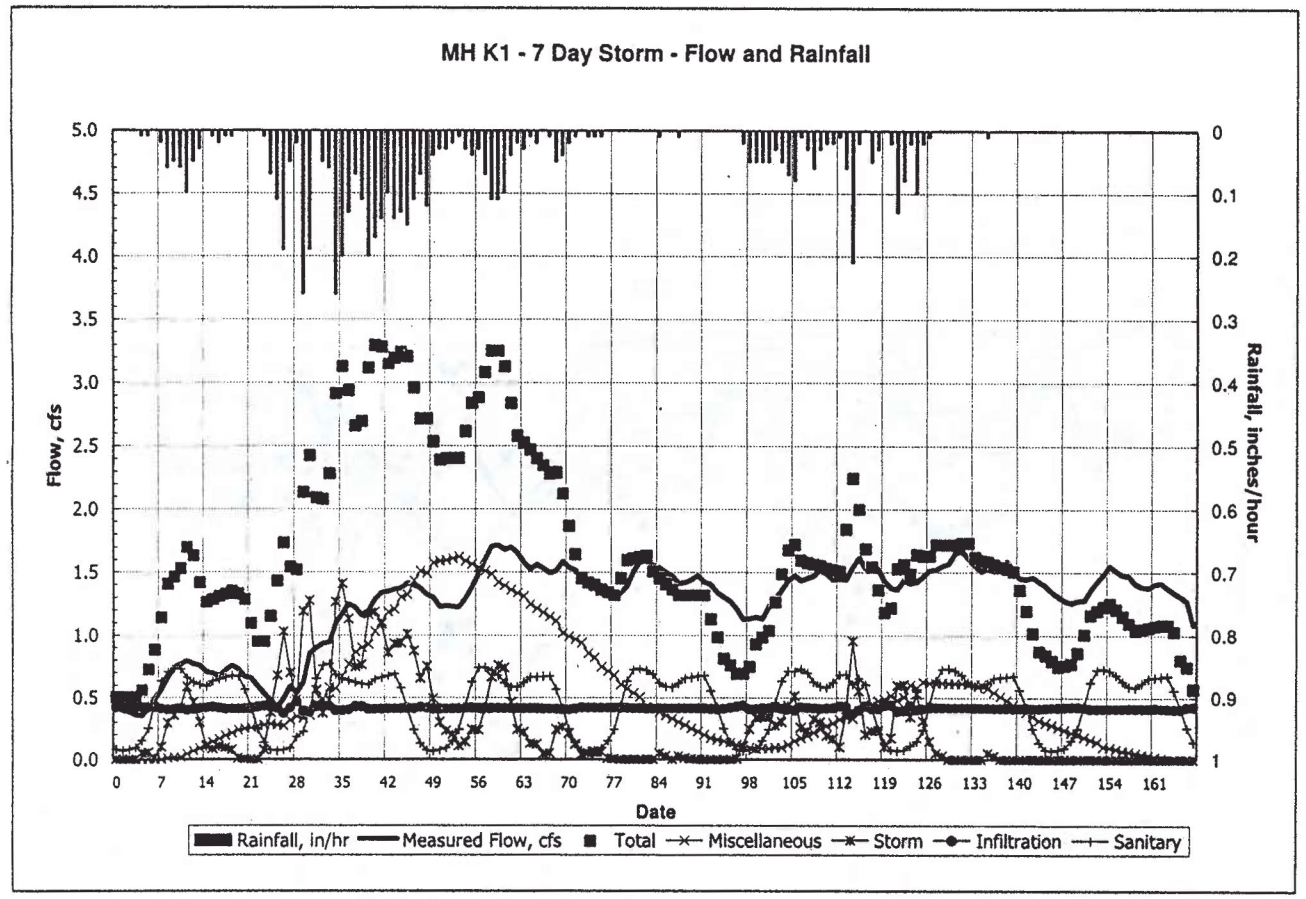

Figure 6-14
Flow Monitoring Data and Hydraulic Model Predicted Flow Manhole K1

6224 009

082000

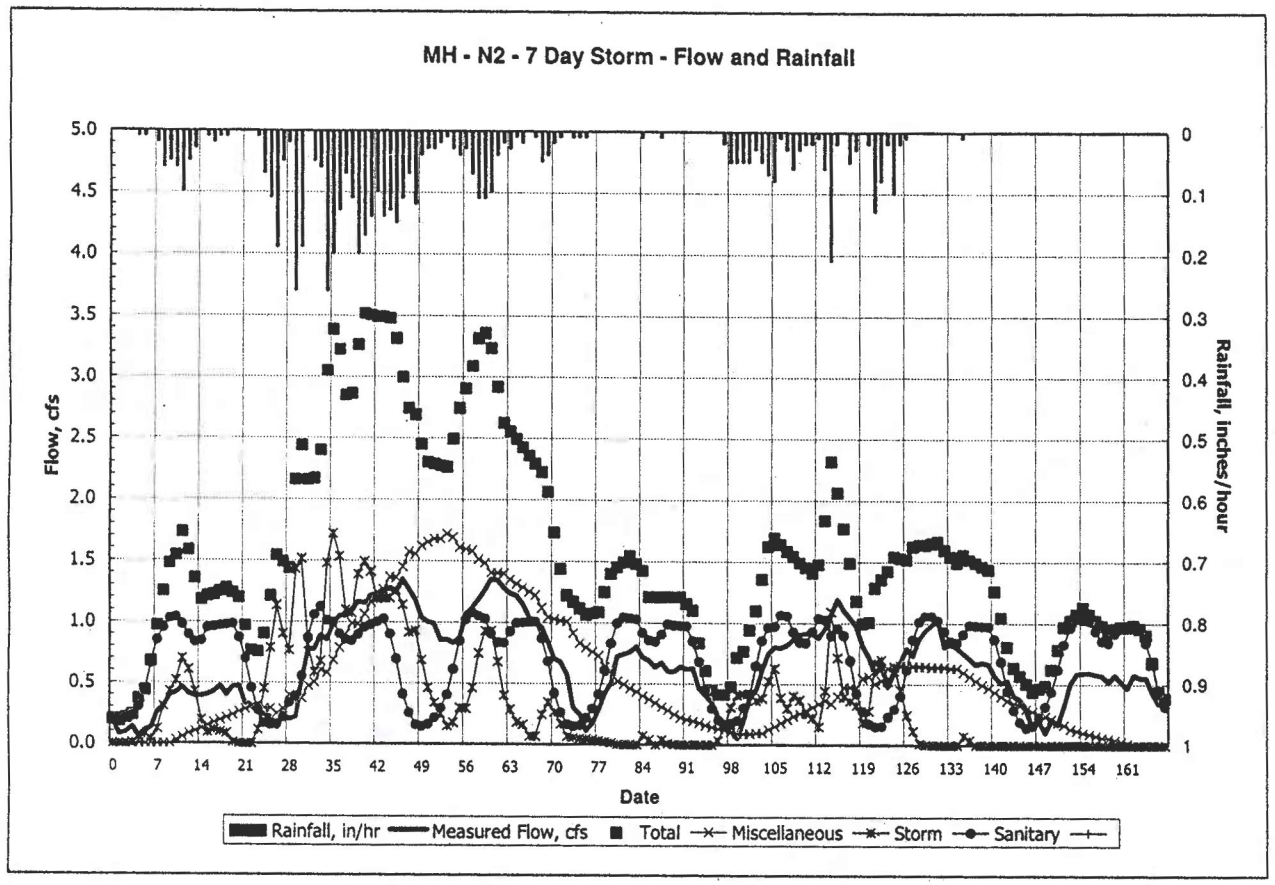

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Figure 6-15
Flow Monitoring Data and Hydraulic Model Predicted Flow Manhole N2



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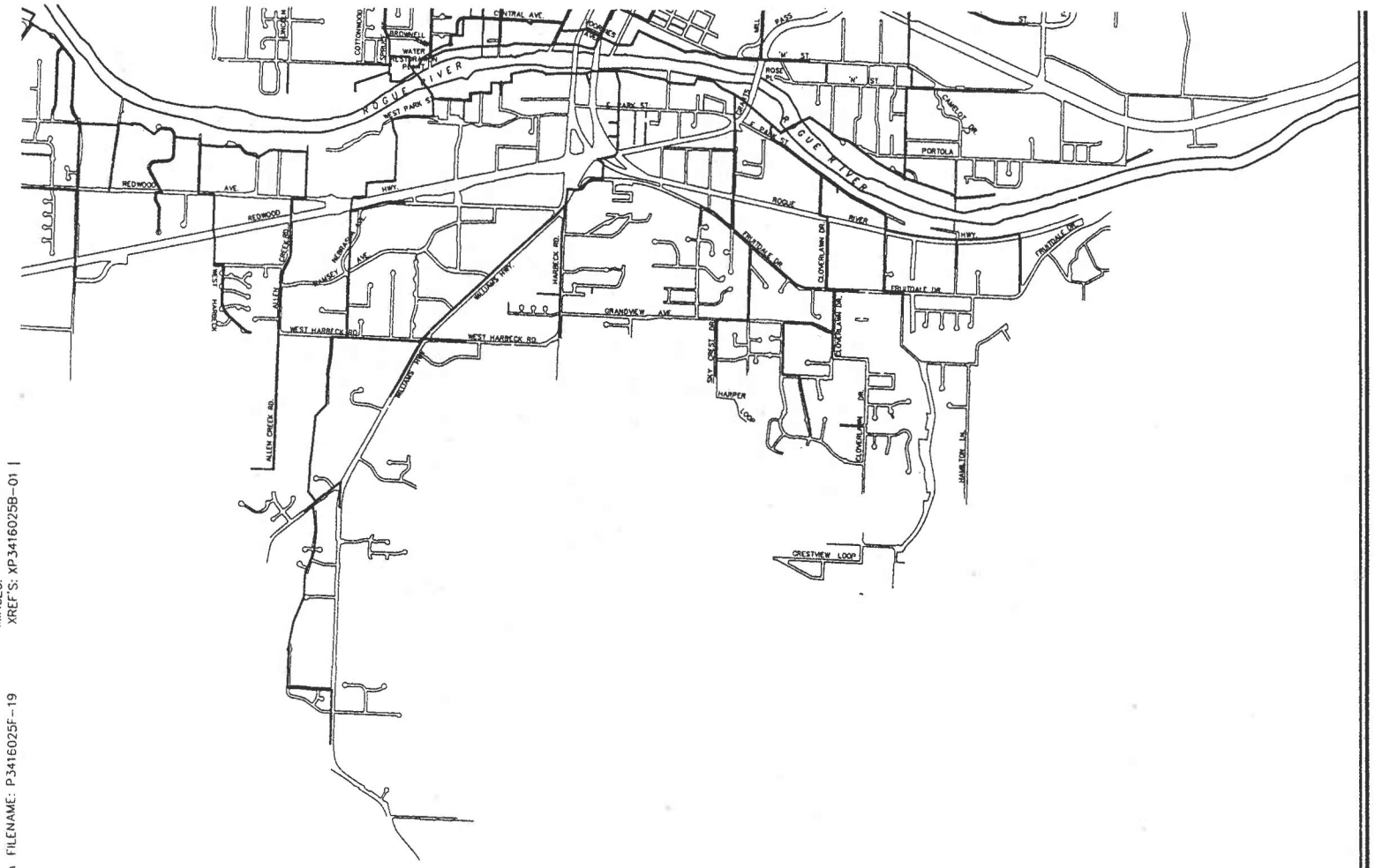
LEGEND

- 2060 SURCHARGE
- 2025 SURCHARGE
- 2003 SURCHARGE
- OTHER MODELED PIPES

SCALE IN FEET
0 1000' 2000'

Figure 6-16
North Section
2 Foot Minimum Surcharge With
All Existing Diversions Active



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Parametrix

LEGEND

- 2060 SURCHARGE
- 2025 SURCHARGE
- 2003 SURCHARGE
- OTHER MODELED PIPES

SCALE IN FEET
0 1000' 2000'

Figure 6-17
South Section
2 Foot Minimum Surcharge With
All Existing Diversions Active

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DATE: 09/27/04 08:37am FILENAME: P34160258-16



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LEGEND

- 2060 SURCHARGE
- 2025 SURCHARGE
- 2003 SURCHARGE
- OTHER MODELED PIPES

SCALE IN FEET
0 1000' 2000'

Figure 6-18
North Section
2 Foot Minimum Surcharge With
All Existing Diversions Removed

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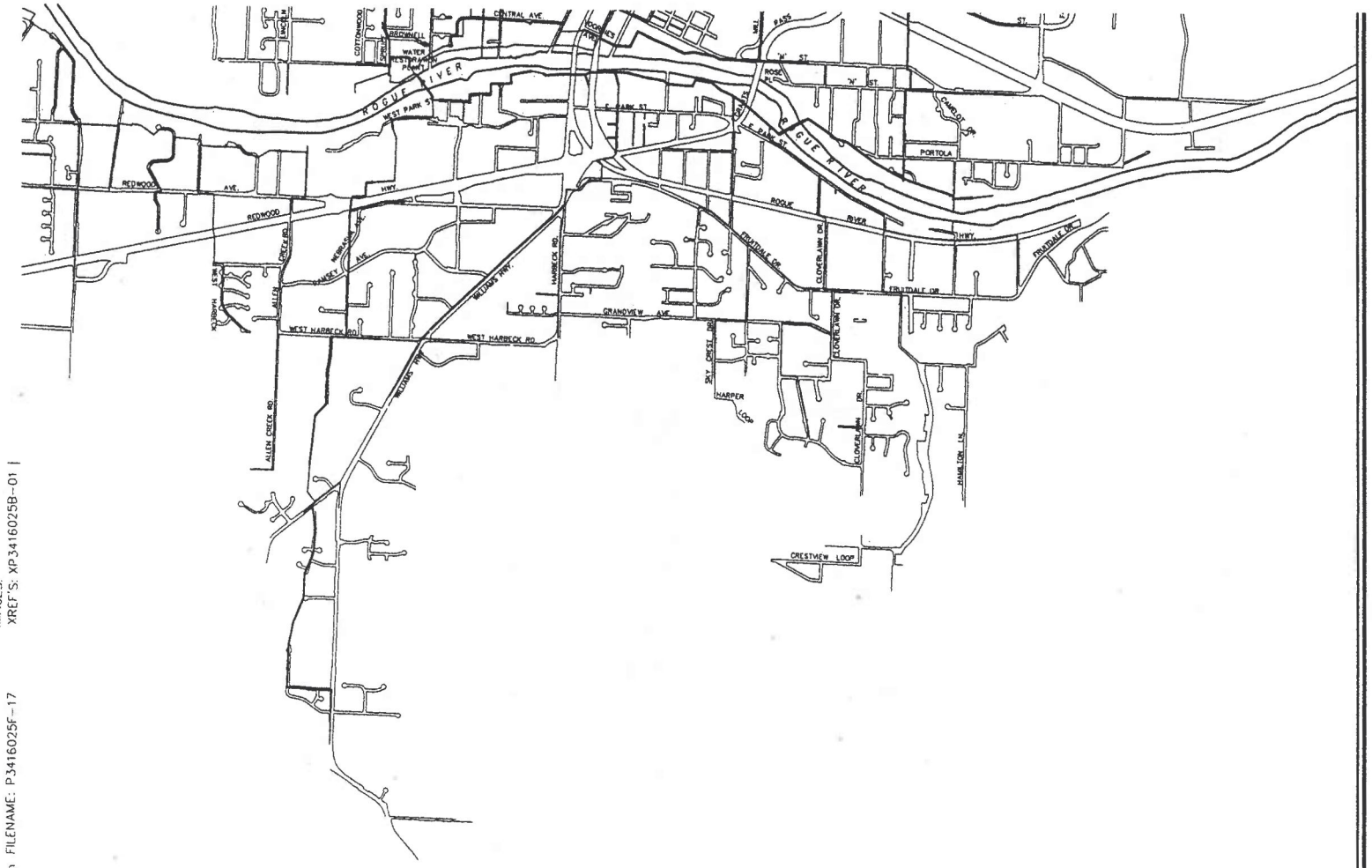

CITY OF GRANTS PASS

COLLECTION SYSTEM
MASTER PLAN

Parametrix

LEGEND

- 2060 SURCHARGE
- 2025 SURCHARGE
- 2003 SURCHARGE
- OTHER MODELED PIPES

SCALE IN FEET
0 1000' 2000'

N

Figure 6-19
South Section
2 Foot Minimum Surcharge With
All Existing Diversions Removed

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XREF'S: XP3416025B-01

Maintenance and Reliability Analysis



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7. MAINTENANCE AND RELIABILITY ANALYSIS

The structural condition of a collection system is a critical issue to examine when considering system upgrades or expansion. For example, repeated structural failure (such as pipes collapsing or the development of sink holes) at various locations along a pipeline alignment will make replacing such a pipeline a priority item. Maintenance history should also be considered since a high level of required maintenance for a pipeline can be an indicator of potential or pending structural problems. Coupling condition and maintenance information with information on hydraulic performance is an informative tool for both identifying and prioritizing both new pipeline projects and the need to rehabilitate or replace existing pipelines. Analysis of the existing City of Grants Pass collection system was based on an examination of condition and maintenance data that the City maintains in a GIS database, and on interviews, workshops, and site visits with the City's operations and maintenance staff. For a comprehensive review of the GIS database that was used in these interviews and workshops, refer to Appendix E. Contained in Appendix E are figures depicting each of the structural and maintenance data.

7.1 CONDITION ASSESSMENT OF EXISTING PIPELINES

As noted in Section 4, the City has compiled collection system condition and maintenance data into a central electronic database and further linked that data into graphical representation of the City's collection system. The analysis and figures presented herein are based on a comprehensive view of this data. There are two aspects to note when using the GIS data. First, GIS data is limited to pipelines within the city limits and the Harbeck–Fruitdale area south of the Rogue River. Condition data for pipelines within the RSSSD collection system were not available and this area was, therefore, excluded from this analysis. However, the RSSSD pipelines are relatively new (constructed after 1970) and, according to the City and RSSSD representatives, comparatively free of serious defects requiring pipeline replacement.

Second, the graphical and numerical analyzing of the City's GIS data is limited to manhole-to-manhole pipeline reaches. As such, it is often the case that defects at a single point, such as a hole in a pipe, will cause a much longer length of sewer to be identified as defective. Point defects, such as holes and cracks are often; however, an indication that the general condition of this pipeline section may be compromised. Overstating the affected length of pipeline defects should, therefore, not be considered a computational or graphical error, but instead should indicate that further investigation into the extent and severity of defects may be warranted.

For ease of evaluation and presentation of the data, the service area was arbitrarily split into north, south, and west sections (see Figure 4-1, page 4-4). The north section lies north of the Rogue River and comprises a majority of the city. The south section lies south of the Rogue River and represents the Harbeck–Fruitdale area. The west section corresponds roughly to the extent of the RSSSD service area.

Structural and maintenance data were analyzed and evaluated in several day-long work sessions with City staff where City maintenance staff and Parametrix tested various weighting factors, for each of the list of records available on pipeline sections as shown in Table 4-3 (page 4-12), to score and review the results. Based on this review and the performance criteria presented in Section 5.3, the weighting factors presented in Table 7-1 were selected to identify needed collection system improvements.

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Table 7-1. Weighting Factors for Collection System Improvements

Structural Data		Age Data		Maintenance Data	
Defect	Weight	Year Constructed	Weight	Maintenance Effort/Condition	Weight
Missing Pipe	10	1920–1939	8	6-Month Cleaning	7
Hole in Pipe	9	1940–1965	6	<6 Gallons of Rock Removed During Cleaning	7
Broken Pipe	8	1966–1979	3	3–5 Gallons of Rock Removed During Cleaning	5
Circumferential Crack	5	1980–Present	0	1–2 Gallons of Rock Removed During Cleaning	3
Longitudinal Crack	7			Heavy Grease	4
Crack (Other)	6			Medium Grease	2
				Light Grease	1
				Heavy Root Encroachment	5
				Medium Root Encroachment	3
				Light Root Encroachment	2
				Service Call	4

The total score of each pipeline segment using these criteria and weighting factors is an indication of the need for repair or improvements. Figures presenting a summary of the available data without applying the weighting factors in Table 7-1 for either the total structural data or the total maintenance data are presented in Figures F1 through F4 in Appendix F. A combination of these data, without the weighting factors, is also shown in Figures F5 and F6 in Appendix F. A compilation of all the available structural, age, and maintenance data evaluated in the Maintenance and Reliability Analysis, and using the weighting factors in Table 7-1, resulted in scores for individual pipeline segments that are shown in Figures 7-1 and 7-2. These figures were then used to identify six areas in the collection system that need further study and improvements (see Section 8.2.2).

7.2 CONDITION ASSESSMENT OF EXISTING PUMP STATIONS

7.2.1 City Pump Stations

The City's lift stations and pump stations were inspected as part of the WRP facility planning effort in 1999. The wet wells for the Webster No. 1 and No. 2 Lift Stations and wet well and force main(s) for the Bridge Street Pump Station were inspected and found to be in good condition. Pump run time meters at the Webster Lift Stations were analyzed to assess pump operation relative to incoming flow. A review of data collected during the high flow events that occurred during December 1996 indicated that Webster No. 1 Lift Station ran only about 20 percent of the time. Corresponding run time for the Webster No. 2 Lift Station was approximately 15 percent. Webster No. 1 and No. 2 Lift Stations run about 15 and 10 percent of the time, respectively, during dry weather conditions.

The runtime meters at the Bridge Street Pump Station were not functioning well during the inspection. The City estimated, however, that the pumps at this station operate only about 20 percent of the time. The Bridge Street Pump Station has air injection for sulfide control and a natural gas fueled generator for standby power.

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Combined Weighted Maintenance and Structural Scores

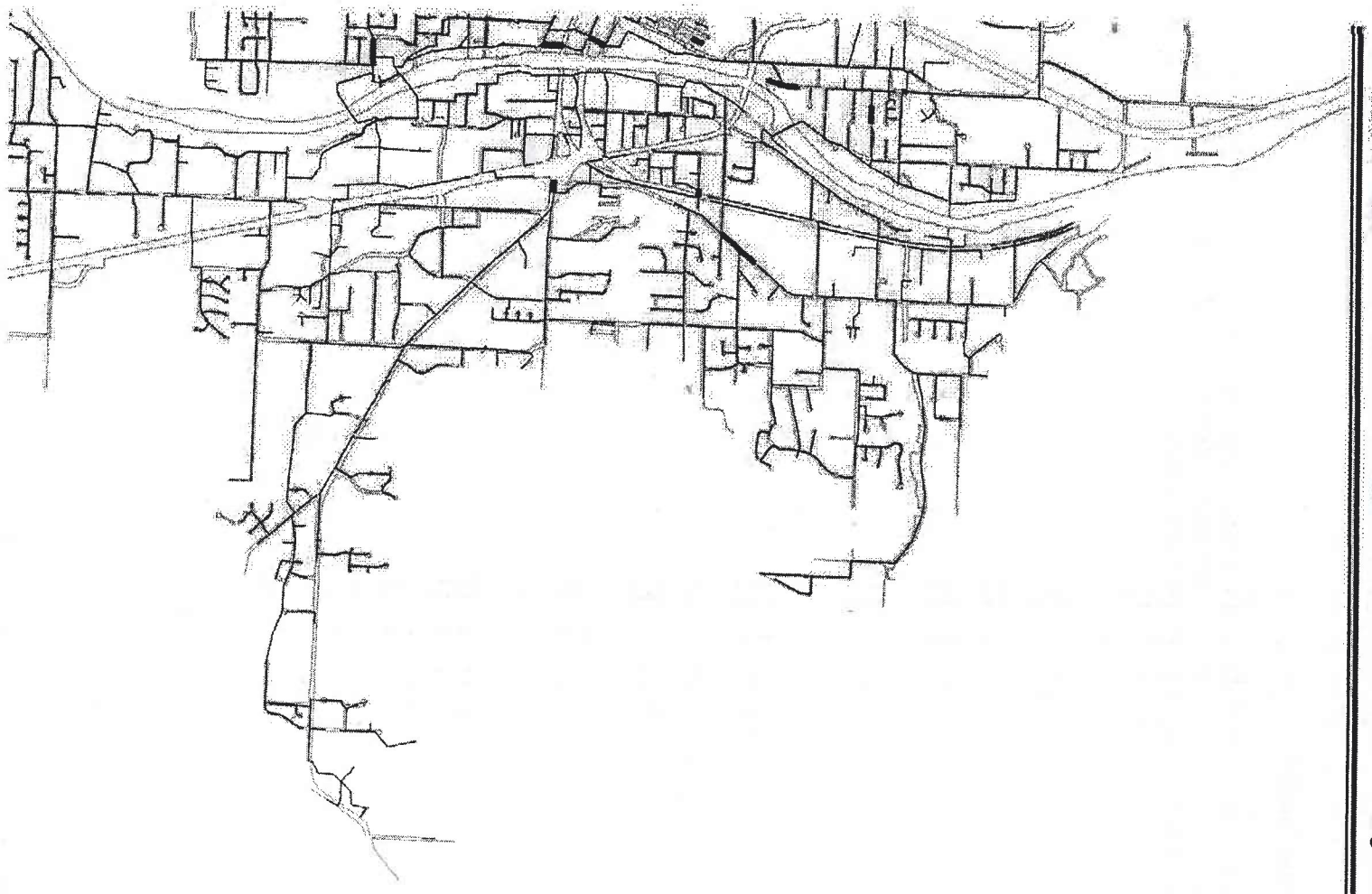
0 1-2 3-4 5-6 7-8 9-10 11-20

SCALE IN FEET
0 1,000 2,000

N

Figure 7 - 1
Combined Weighted Maintenance and Structural Scores

North Section



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Parametrix

Combined Weighted Maintenance and Structural Scores

0 1-2 3-4 5-6 7-8 9-10 11-20

SCALE IN FEET
0 1,000 2,000

N

Figure 7 - 2
Combined Weighted Maintenance and Structural Scores

South Section

682000

Webster No. 1 Lift Station is designed not to overflow even during power failure. Wastewater will flow through the station and to the WRP in pipelines in the event of a power failure. There is also no emergency overflow for Webster No. 2 Lift Station, but the station can be powered by a trailer-mounted generator in the event of power failure. Therefore, both of these lift stations can operate during power failures.

Based on these observations and condition assessment, no improvements are needed to the City's pump stations.

7.2.2 RSSSD Pump Station

As described in Section 4.2.2, both the Redwood and Darneille Pump Stations and their associated force mains are very new and include standby emergency generators. Both of these pump stations can operate during power failure. No improvements are needed to either station.

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Recommended Collection System Improvements



8. RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

The recommended collection system improvements presented in this section are based upon deficiencies in pipeline hydraulic capacity that were identified in the Hydraulic Analysis discussed in Section 6 and the needed collection system improvements that were identified in the Maintenance and Reliability Analysis presented in Section 7. A Capital Improvement Plan (CIP) for the collection system has then been developed based on a priority analysis of these improvements.

8.1 GOALS

Three goals were used in identifying the recommended collection system improvements required and a schedule for implementation and developing a CIP for the collection system.

- Service to Saturation-Level Populations: All of the needed improvements were selected to serve the 2060 populations which could occur in the collection system service area.

- Attention to Critical Improvements: Attention was given to collection system pipelines and subbasin service areas that City staff have identified as problem areas. City staff experience in the frequency of maintenance of various pipelines and witnessing surcharged pipelines (hydraulic capacity deficiencies) have been used, particularly to schedule needed improvements.

- Distribution of Capital Expenditures: In the selection and scheduling of the required collection system improvements, the goal was to develop a CIP that is financially viable for the City of Grants Pass.

8.2 RECOMMENDED IMPROVEMENTS

8.2.1 Hydraulic Capacity Improvements

Based on the hydraulic analysis conducted on the wastewater collection system serving the City of Grants Pass, five capital improvement projects have been identified that need to be completed in the next 20 years. These improvements are necessary to maintain adequate conveyance system capacity in the collection system and prevent sewer system overflows.

These five projects are shown in Figure 8-1 (page 8-2) and described further below.

8.2.1.1 Pine Street

The Pine Street sewer, traveling south along 2nd Street, Booth Avenue, and Pine Street, has already experienced surcharging during heavy rainfall events. To alleviate the surcharging, a diversion was installed at "F" Street to divert flow to the east. Extending from "A" Street to Central Avenue, the Pine Street sewer needs to be replaced with a new larger-diameter pipeline. This project is shown in Figure 8-2 (page 8-3).

The hydraulic analysis indicates that this new pipeline should be 18 to 24 inches in diameter, depending on its exact slope. Once constructed, the existing diversion at "F" Street should be removed. The beginning of the new Pine Street sewer starts at the end of a recently completed new pipeline in Hawthorne Avenue.

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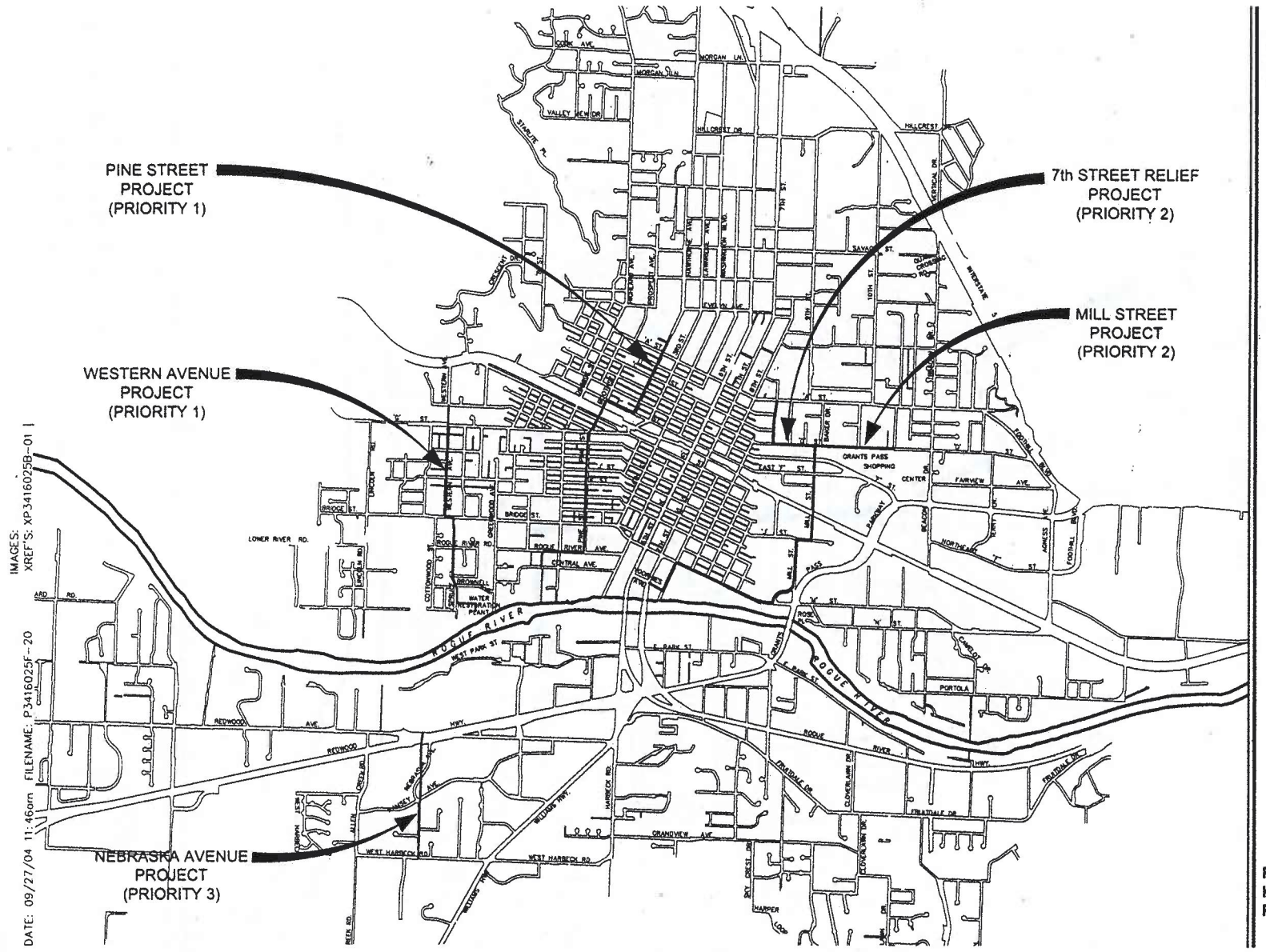

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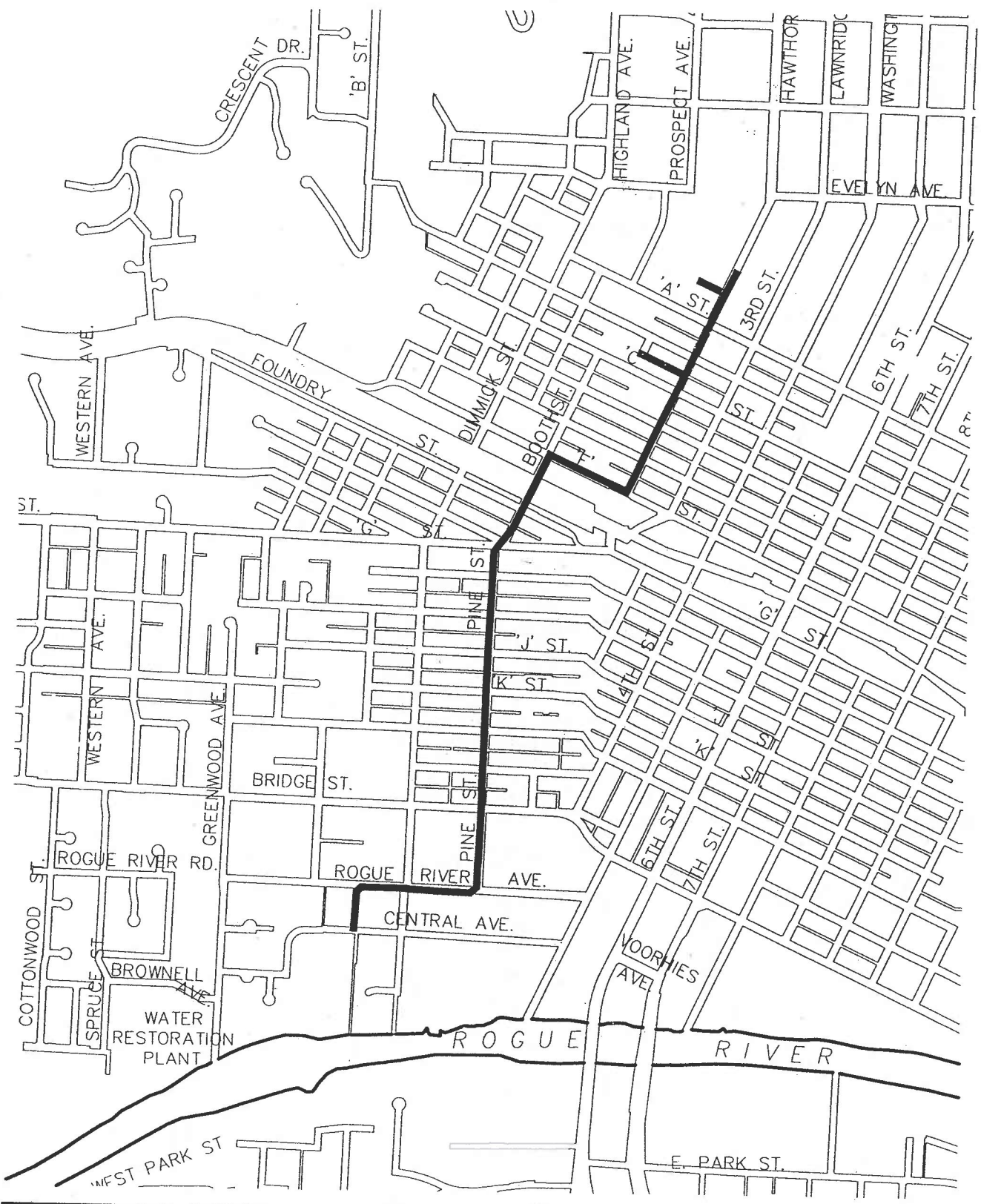
Figure 8-1
Key Map of Proposed
Pipeline Improvement Projects



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SCALE IN FEET
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Figure 8-2
Proposed Pine Street
Pipeline Improvement Project

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CITY OF GRANTS PASS, OREGON

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8.2.1.2 Western Avenue

Similar to the Pine Street sewer, the existing Western Avenue sewer from "G" Street to Bridge Street, and then along Spruce Street to the WRP, needs to be replaced with a new larger-diameter pipeline. The existing pipeline serves the area west of Western Avenue between Upper River Road and the Rogue River. Increasing wastewater flows in the future are expected in the area due to growth. This sewer also receives flow from the Bridge Street Pump Station. This pipeline has already experienced surcharging during peak rainfall events. To alleviate existing surcharge conditions, a diversion was installed at Bridge Street to divert flow to the east. The hydraulic analysis indicates that a new 18-inch-diameter pipeline, approximately 4,700 feet, is required as shown in Figure 8-3 (page 8-5). Once constructed, the diversion at Bridge Street should also be removed.

8.2.1.3 Mill Street

The proposed Mill Street sewer (Figure 8-4, page 8-6) is the replacement of an existing pipeline with a new larger-diameter sewer to provide several benefits. The area north and east of the existing sewer is experiencing growth, and this project will provide conveyance capacity for this growth. Also, however, if this new sewer were not installed, the existing dual siphons crossing the Rogue River and the existing sewer paralleling the river on the south side, would likely reach capacity about Year 2025.

To avoid construction of a new siphon and sewer on the south side of the Rogue River, the Mill Street sewer is necessary. This new pipeline would enable flow from the east to be diverted away from the existing sewer in Beacon Drive, which then flows to the dual siphons and would avoid having to construct a new siphon. This flow would reach the WRP using an alternate route through the sewer on the north side of the river.

The Mill Street sewer extends the hydraulic capacity of the dual siphons and south river sewer beyond Year 2060.

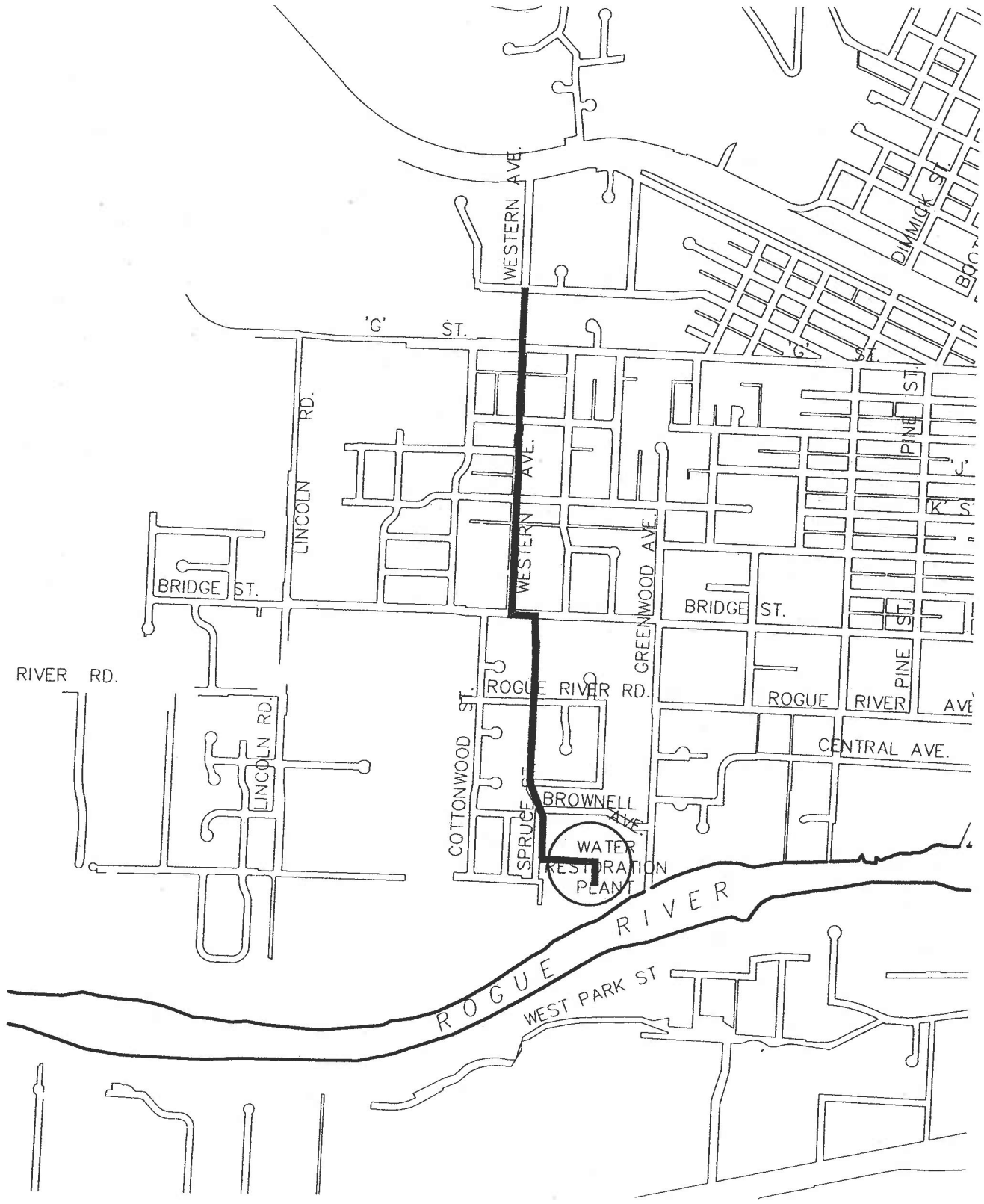
8.2.1.4 7th Street Relief Sewer

The 7th Street sewer is expected to experience significant increases in flow from the service area north of "A" Street and is expected to reach capacity based on the hydraulic analysis. To avoid replacement of this pipeline in 7th Street, which was just recently improved and caused significant traffic disruption, a new relief sewer is proposed. This relief sewer is shown in Figure 8-5 (page 8-7). This relief sewer would divert flow away from 7th Street and into the new Mill Street sewer once it was constructed.

The new 18-inch-diameter 7th Street relief sewer carries flow away from 7th Street and avoids replacement of the 7th Street sewer until beyond 2060.

8.2.1.5 Nebraska Avenue

Wastewater flow from the area south of the Redwood Highway is expected to increase significantly during the next 20 to 25 years, requiring a new 18-inch-diameter sewer at Nebraska Avenue as shown in Figure 8-6 (page 8-8). The area south of West Harbeck Road has already experienced growth and is expected to continue to grow. To serve this area in the future, a new 18-inch-diameter Nebraska Avenue sewer is necessary.



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SCALE IN FEET
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Figure 8-3
Proposed Western Avenue Pipeline Improvement Project
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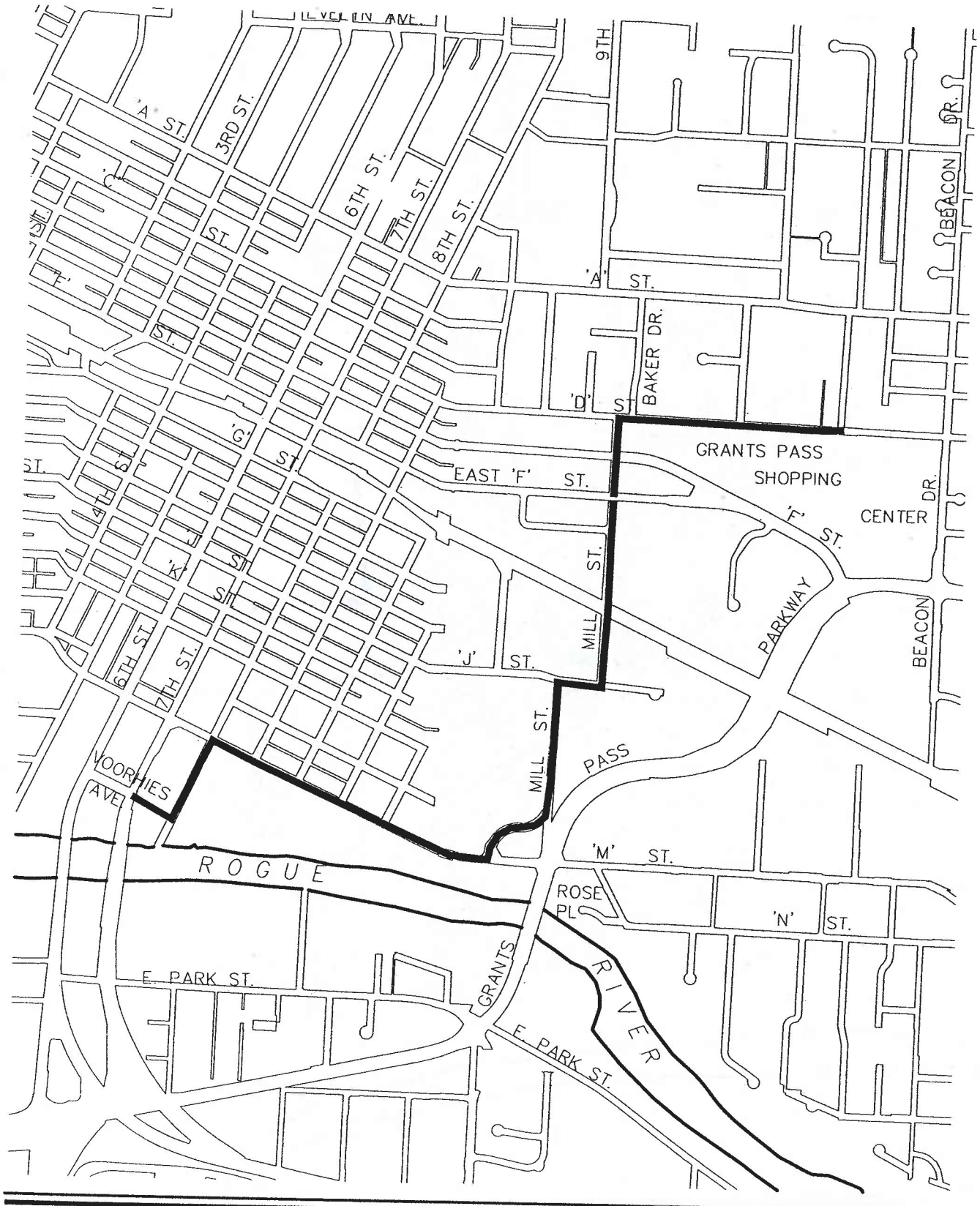

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SCALE IN FEET
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Figure 8-4
Proposed Mill Street
Pipeline Improvement Project

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Parametrix DATE: 09/24/04 09:11am FILE: P3416025F-24

SCALE IN FEET
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Figure 8-5
Proposed 7th Street Relief Project

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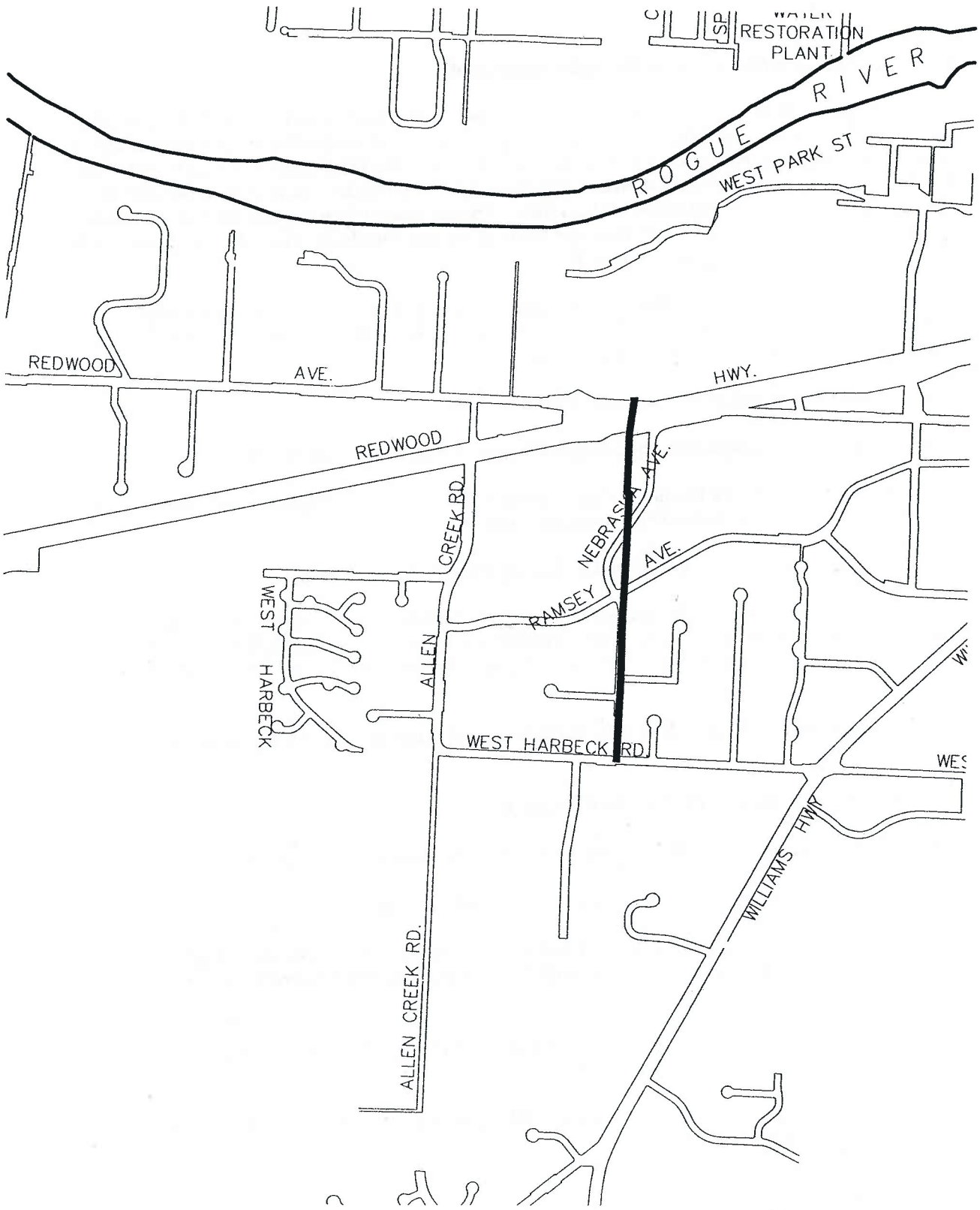

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SCALE IN FEET
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Figure 8-6
Proposed Nebraska Avenue Pipeline Improvement Project
COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON

8.2.2 Maintenance and Reliability Improvements

As described in Section 4, much of the downtown area of Grants Pass is served by very old (greater than 60 years old) small-diameter (often 6-inch-diameter) pipelines. Based on the Maintenance and Reliability Analysis conducted on the wastewater collection system as described in Section 7, six areas of the city that are served by these old small-diameter collection lines need further investigation and will likely require repair or replacement of much of these lines. The six structural repair areas that were identified are shown in Figure 8-7 (page 8-8). A summary figure of the size, length, and age of the pipelines in each of the six areas is shown in Figure 8-8 (page 8-11).

The limits of each area were identified with the assistance and guidance of the City's sewer maintenance staff and using the Maintenance and Reliability Analysis process described in Section 7. Generally, each of these areas exhibit the following identical problems:

- Extensive 6-inch-diameter (very old) pipelines.
- Extensive structural defects, including holes in pipes, broken pipes, and cracks.
- Extensive root encroachment into these pipelines that require very frequent, difficult cleaning and numerous service calls to ensure continued operation.
- Extensive rock removal during routine cleaning and maintenance.

Rather than attempt to identify individual pipelines within each of these areas that require repair/replacement, these areas have been identified as needing more detailed investigations and inspections, after which specific repair/replacement programs for each area can then be identified and implemented.

The following steps should be followed to develop a detailed repair/replacement program for each of these areas:

- Investigate existing database information in detail.
- Conduct further testing and inspections and review all available video inspections.
- Identify specific pipeline sections requiring repair and/or replacement.
- Evaluate alternative repair/replacement technologies, including slip lining, cured-in-place (i.e. in situ form technologies), pipe-bursting, and fold-and-form pipe repair, together with complete pipe replacement.
- Identify alternative technology costs and select repair/replacement process for each specific pipe segment based on cost and other considerations.
- Develop final repair/replacement program and begin implementation (i.e., design, construction award, and construction).

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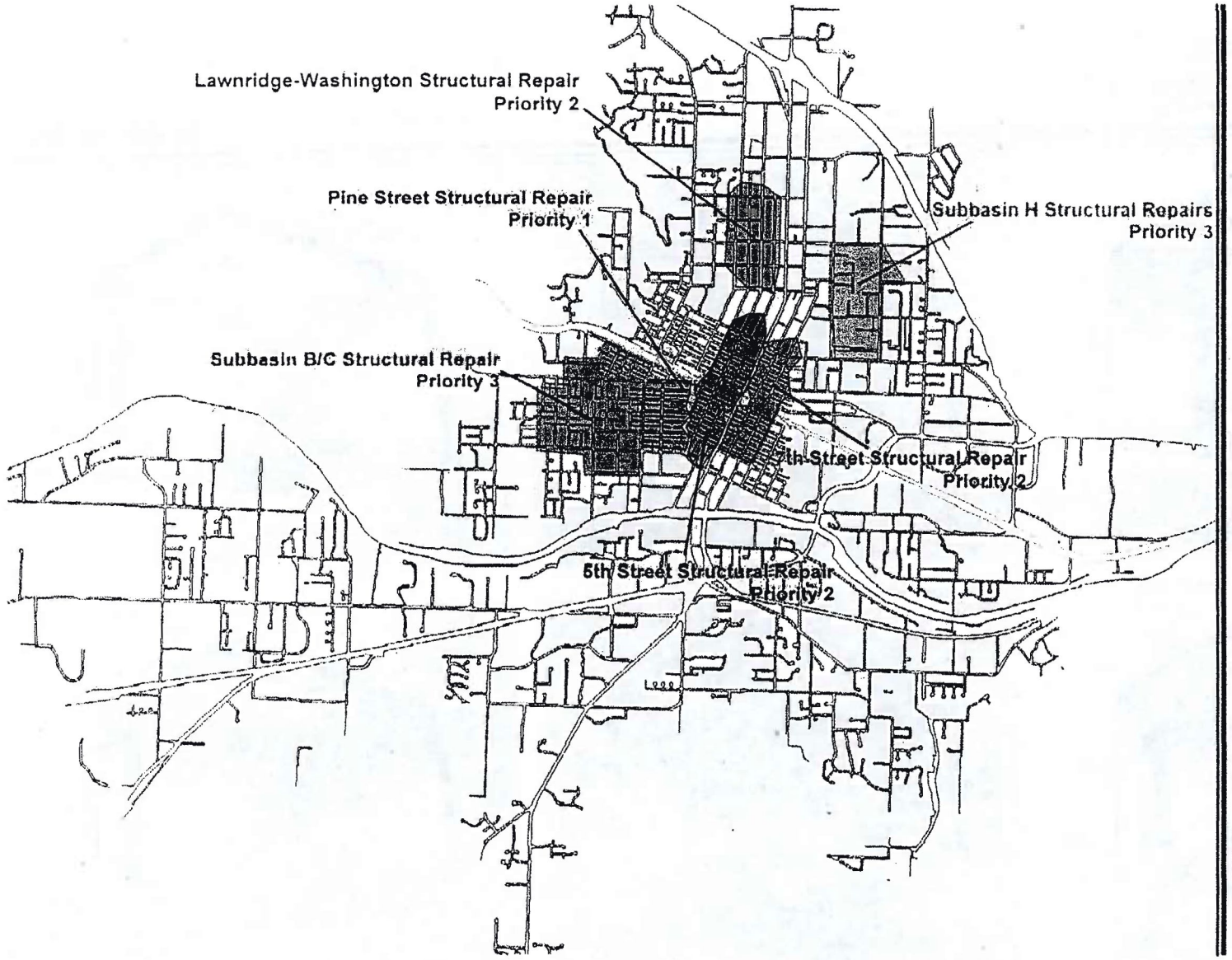

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Project Area by Priority

- Pine Street Structural Repair Priority 1
- 5th Street Structural Repair Priority 2
- 7th Street Structural Repair Priority 2
- Lawnridge-Washington Structural Repair Priority 2
- Subbasin B/C Structural Repair Priority 3
- Subbasin H Structural Repairs Priority 3

SCALE IN FEET

0 1,250 2,500

N

Figure 8 - 7
Proposed Structural Repair Areas

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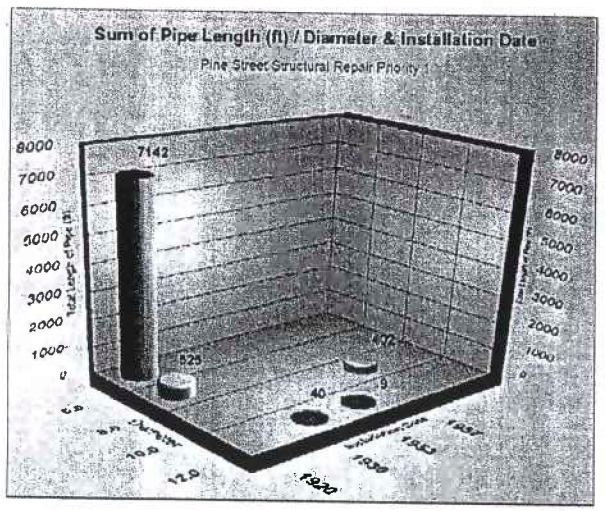

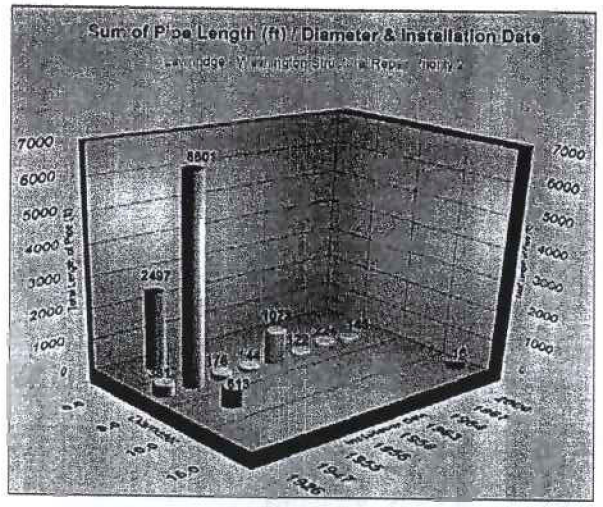

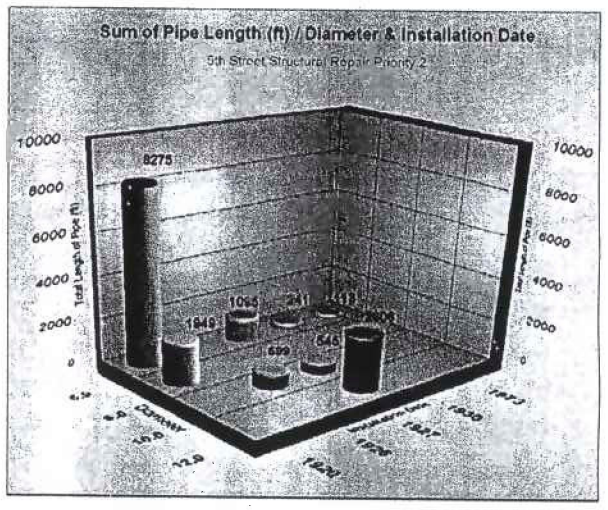

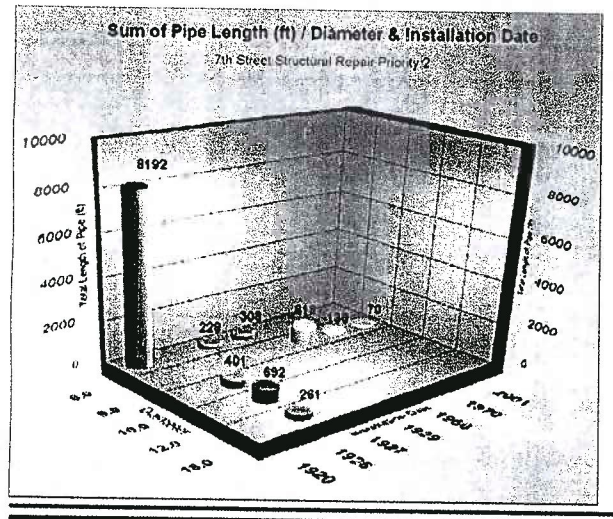

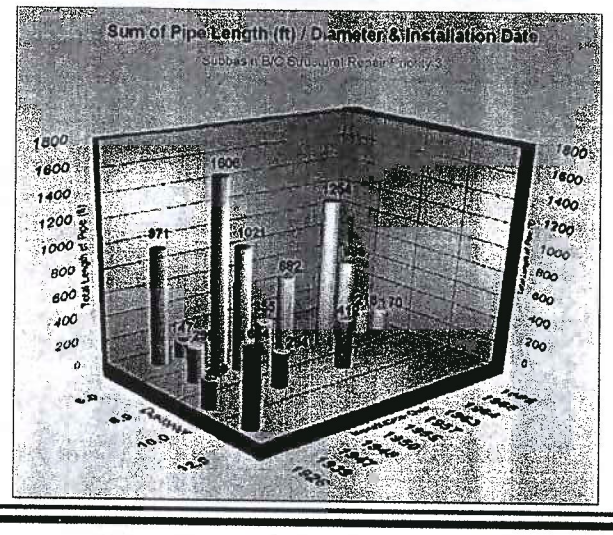

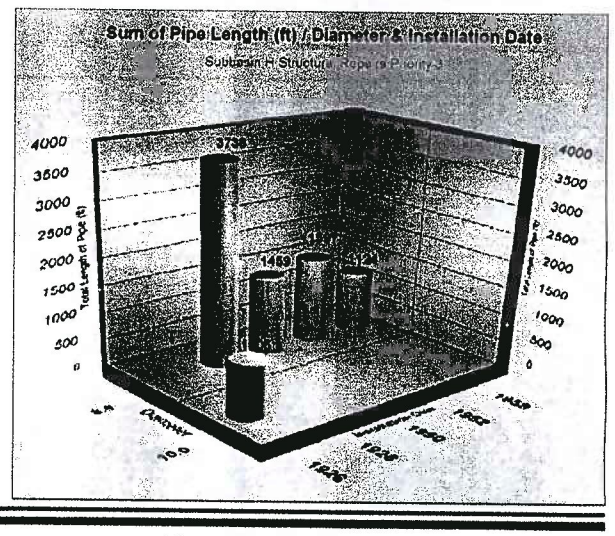

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Figure 8 - 8
Proposed Structural Repair Areas

COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON

Each of these areas is identified in the following six Figures 8-9 through 8-14. The individual pipeline segments in each area, that need further investigation and repair and/or replacement, are highlighted in red. Table 8-1 contains a detailed breakdown of these pipeline sizes and lengths for each area.

Table 8-1. Detailed Pipeline Size and Length for Each Segment

Structural Repair Area Project	Diameter	Total Length (feet)	Total Segments
Pine Street Structural Repair	6.0	7,141.81	18
	8.0	525.00	1
	10.0	402.13	1
	12.0	48.44	2
Project Group Total:		**8,117.38**	**22**
5th Street Structural Repair	6.0	9,723.05	34
	8.0	1,948.84	8
	10.0	1,243.68	4
	12.0	2,606.26	10
Project Group Total:		**15,521.83**	**56**
7th Street Structural Repair	6.0	8,728.34	29
	8.0	598.59	3
	10.0	400.58	1
	12.0	692.28	2
	18.0	260.65	1
	Unknown	427.51	1
Project Group Total:		**11,107.95**	**37**
Subbasin B/C Structural Repair	6.0	1,118.02	4
	8.0	6,291.11	21
	10.0	945.14	3
	12.0	644.20	2
Project Group Total:		**8,998.47**	**30**
Subbasin H Structural Repairs	8.0	7,918.49	32
	10.0	1,005.18	3
Project Group Total:		**8,923.67**	**35**
Lawnridge-Washington Repair	6.0	2,496.55	6
	8.0	8,866.53	34
	10.0	612.90	2
	15.0	15.99	1
Project Group Total:		**11,991.97**	**43**
All Project Groups Total:		**64,661.28**	**223**

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8.3 PRIORITIZATION OF IMPROVEMENTS

In Figure 8-1 (page 8-2), the five hydraulic capacity improvements recommended were prioritized based on the results of the hydraulic model analysis. Improvements based on more immediate hydraulic restrictions (existing surcharging) were identified as priority one improvements. Improvements based on Year 2025 hydraulic restrictions were generally identified as priority two, and those based on Year 2060 hydraulic restrictions identified as priority three.

In Figure 8-7 (page 8-10), the six structural repair areas recommended for further investigation and pipeline repair/replacement were also prioritized based on extent of structural and maintenance related issues identified during the Maintenance and Reliability Analysis. These prioritizations will be used in development of a Capital Improvement Plan for the collection system.

8.4 ESTIMATED COST OF IMPROVEMENTS

Using the gravity sewer pipeline construction cost unit prices developed in Section 5 and shown in Section 5.1, and the total project cost components shown in Table 5-2 (page 5-3), total project costs for each of the recommended pipeline improvement projects were prepared. These costs are developed and presented in Table 8-2.

Table 8-2. Cost of Recommended Pipeline Improvement Projects (in $1M)

Project	Preliminary Length (feet)	Construction Diameter (inches)	Base Cost ($/foot)	Construction Construction Cost	Contingency 40%	Engineering (Legal and Administration) 30%	Total Estimated Project Cost
Pine Street	7,010	24	$200	$1.40	$0.56	$0.59	$2.55
Western Avenue	4,720	18	$170	$0.80	$0.32	$0.34	$1.46
Mill Street	9,140	21	$185	$1.69	$0.68	$0.71	$3.08
7th Avenue	4,530	18	$170	$0.77	$0.31	$0.32	$1.40
Nebraska	2,710	18	$170	$0.46	$0.18	$0.17	$0.81

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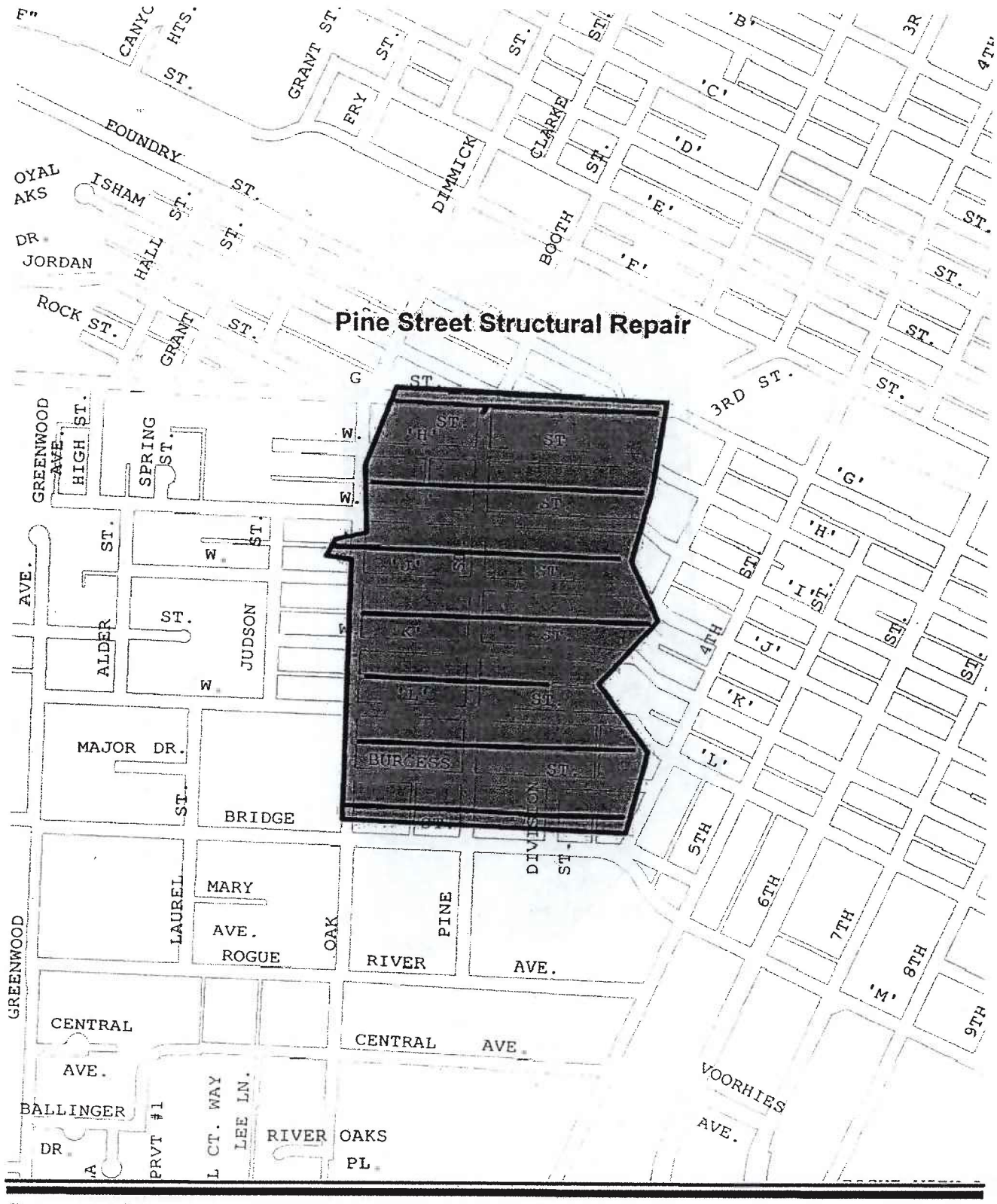

Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-9

SCALE IN FEET
0 300 600

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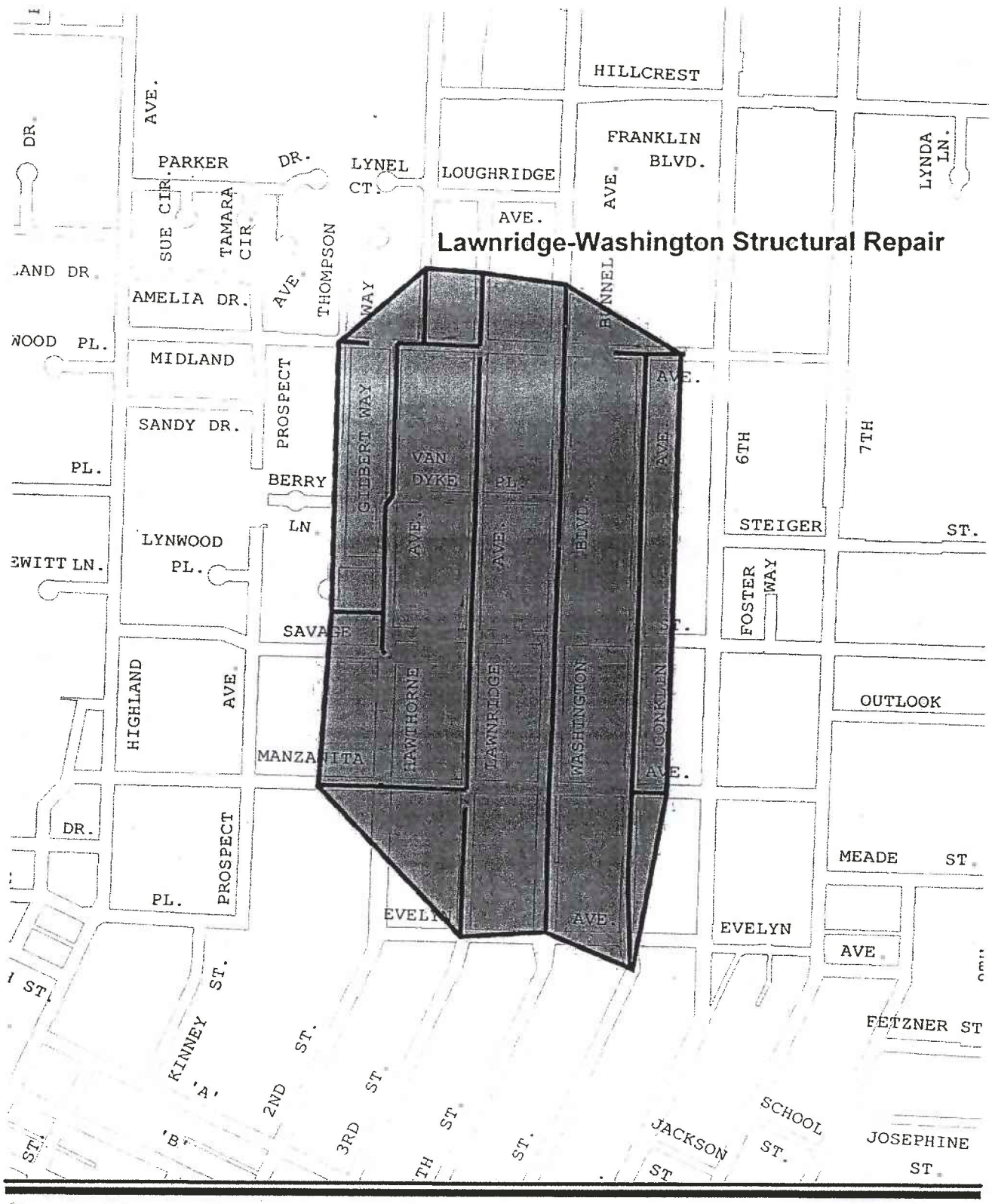
Legend

Pine Street Structural Repair Priority 1

Figure 8 - 9
Pine Street Structural Repair

COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON



Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-10

SCALE IN FEET
0 300 600

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Lawnridge-Washington Structural Repair Priority 2

Figure 8 - 10
Lawnridge-Washington Structural Repair

COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON

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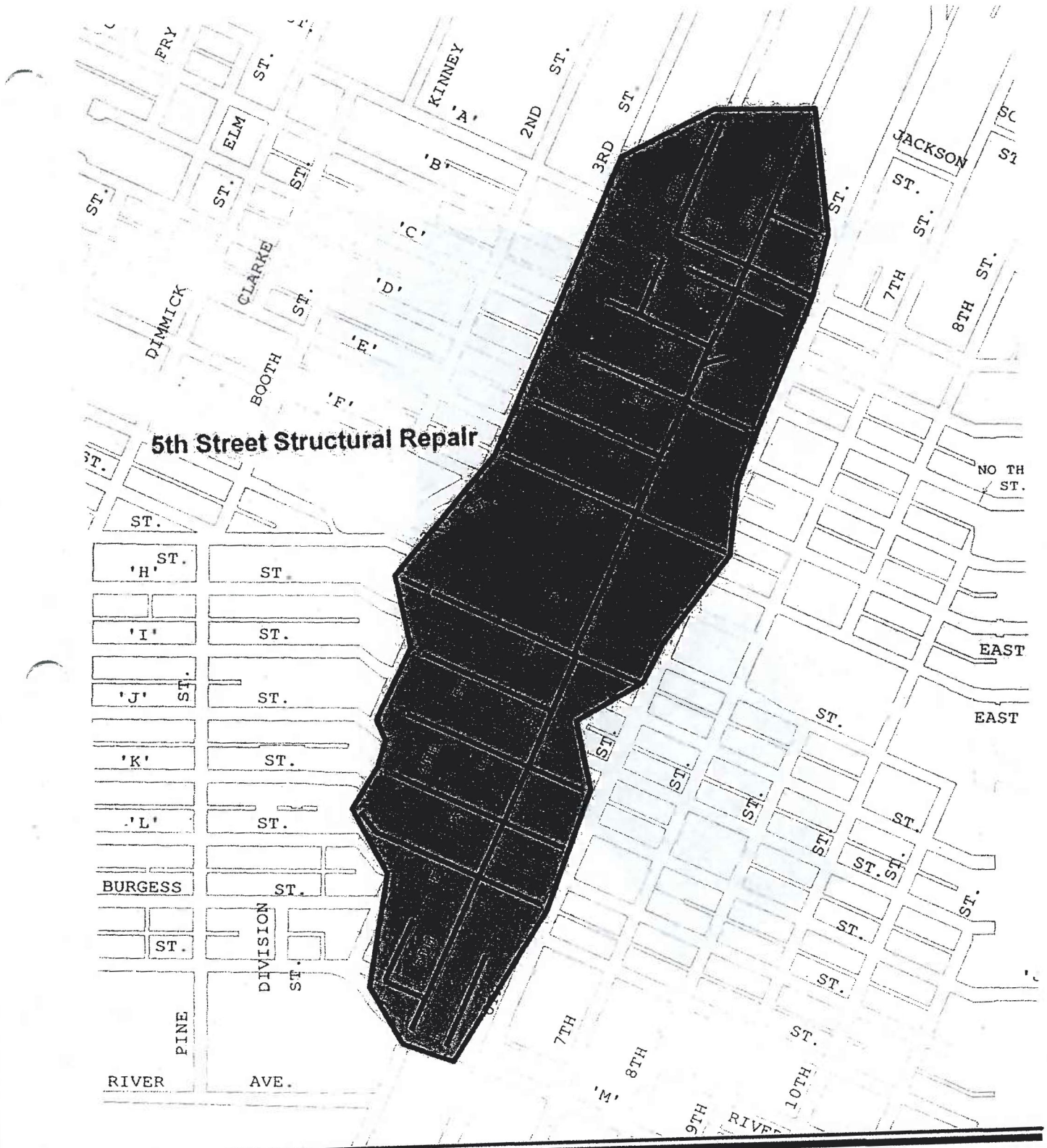

Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-11

SCALE IN FEET
0 300 600

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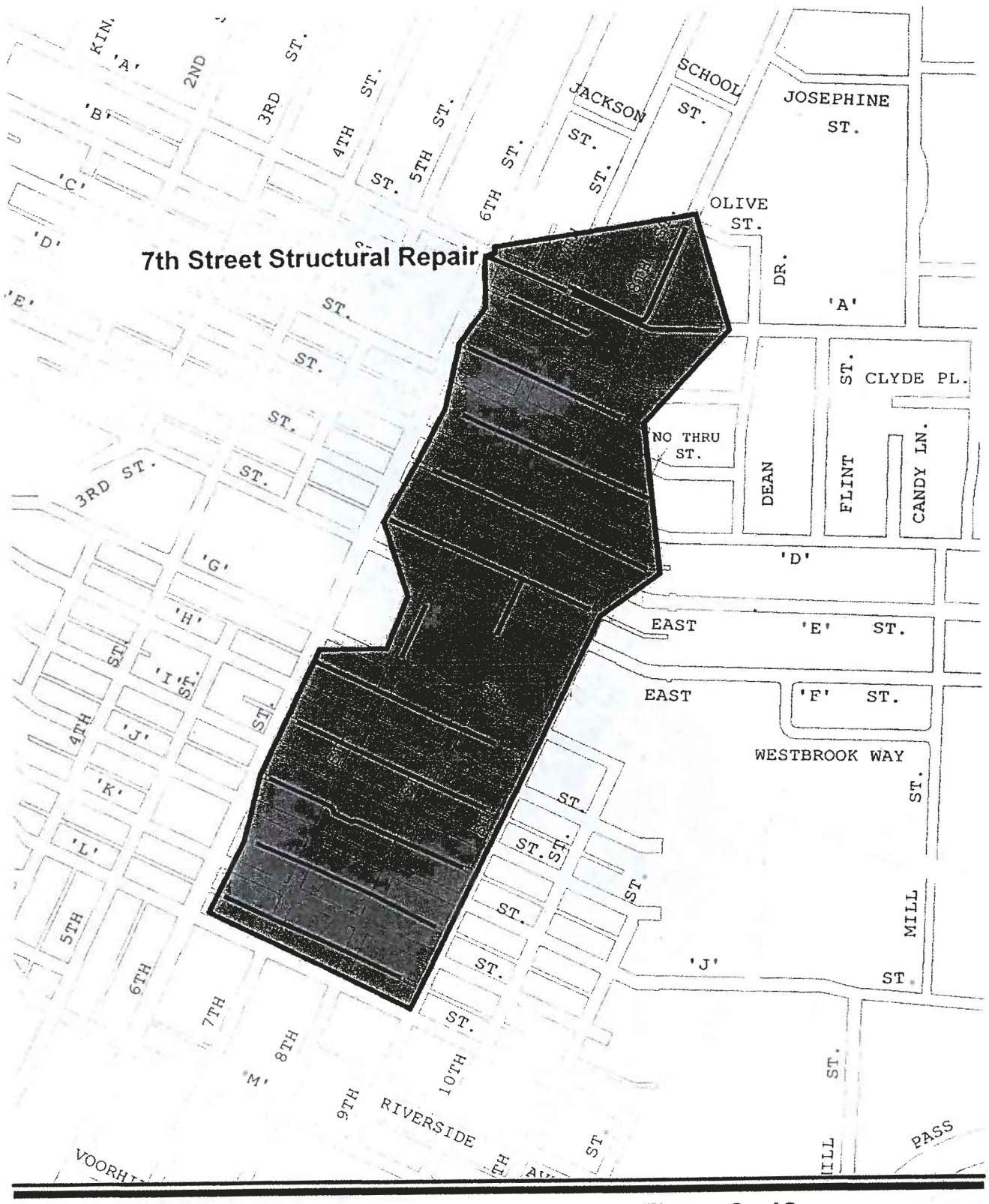
5th Street Structural Repair Priority 2

Figure 8 - 11
5th Street Structural Repair
COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON



Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-12

SCALE IN FEET
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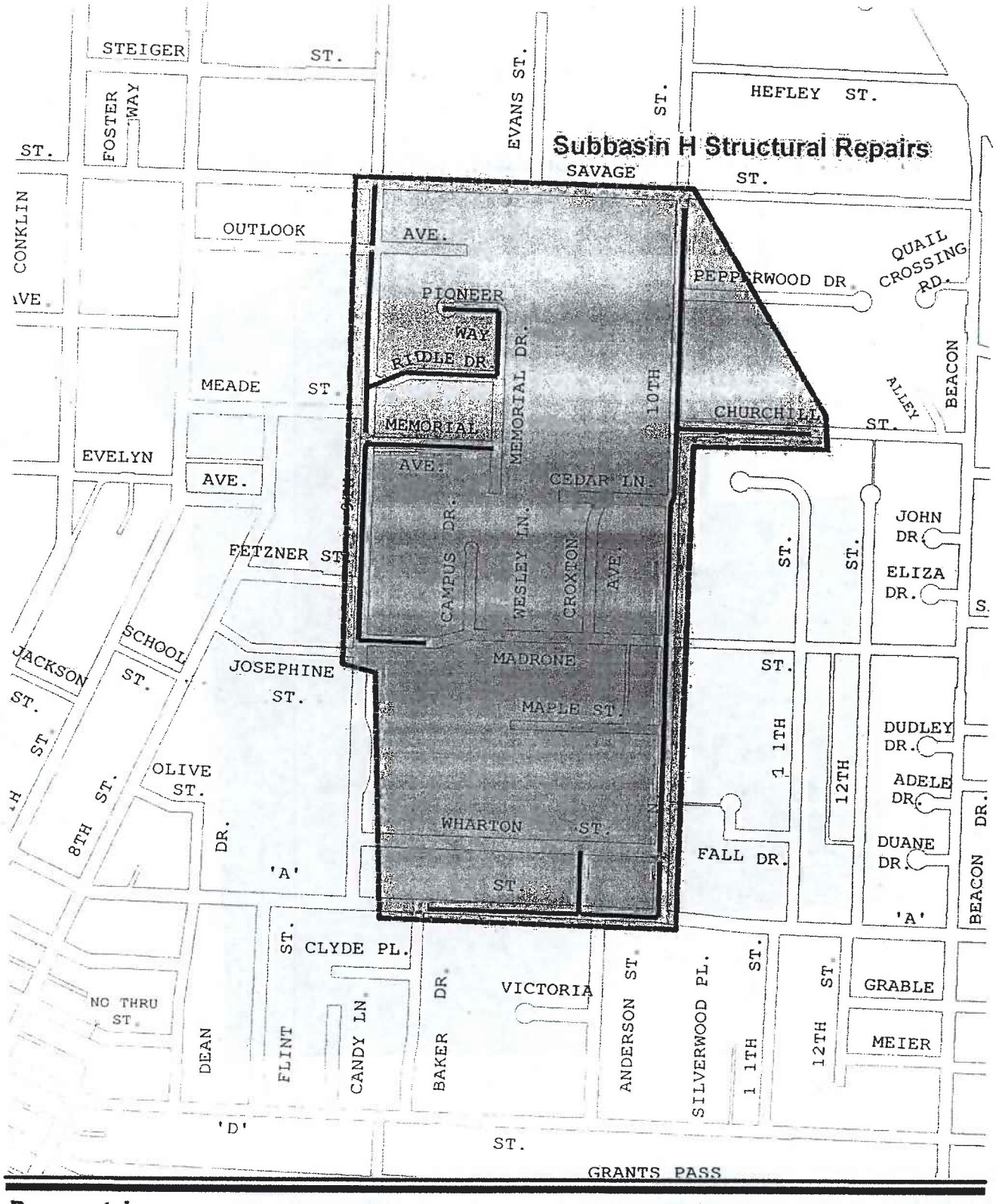
Legend
7th Street Structural Repair Priority 2

Figure 8 - 12
7th Street Structural Repair
COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON



Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-13

SCALE IN FEET
0 300 600

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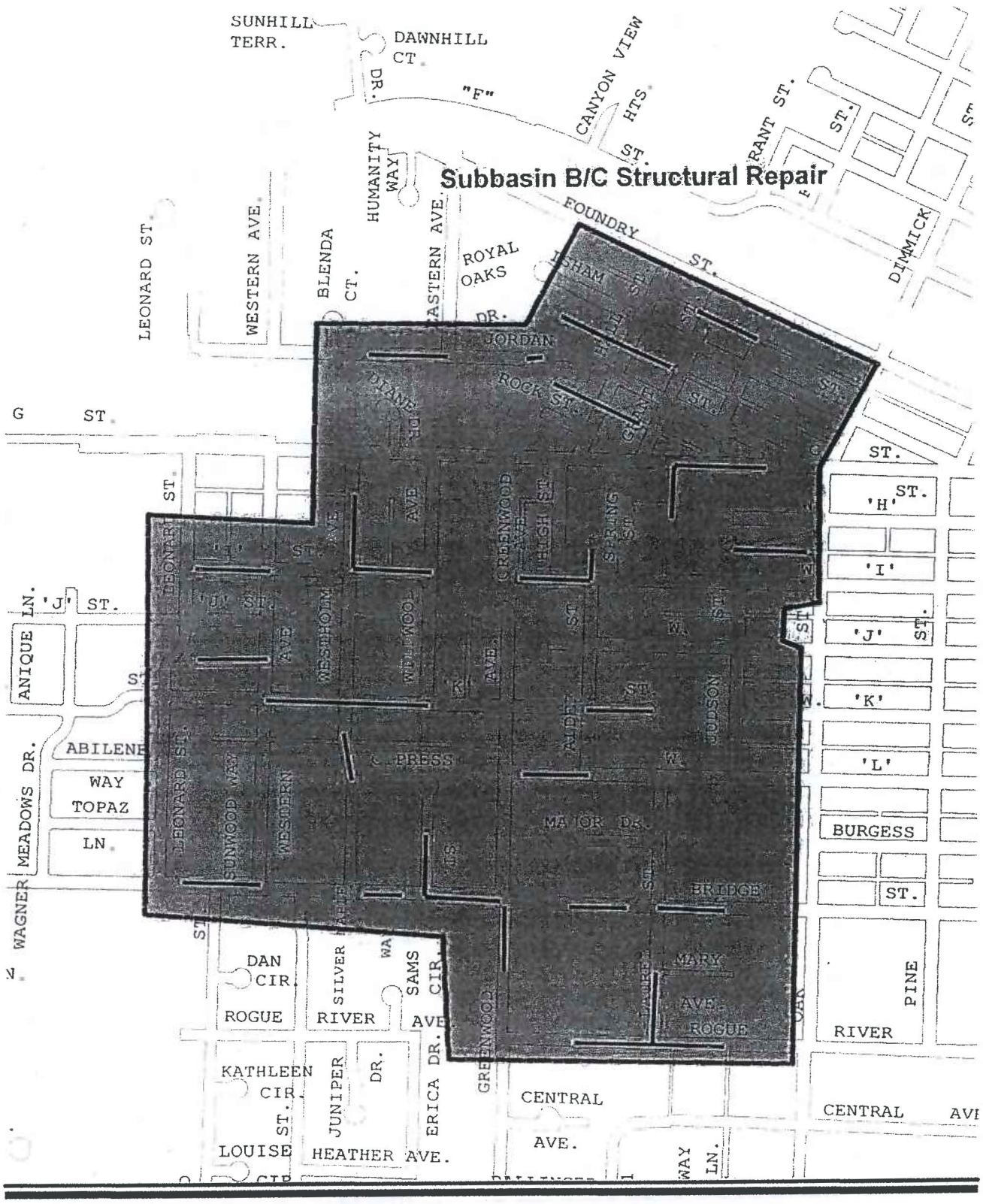
Legend

Subbasin H Structural Repairs Priority 3

Figure 8 - 13
Subbasin H Structural Repairs
COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON



Parametrix DATE: 09/23/04 FILE: K:\GRANTSPASS\MXD\FIGURE 8-14

SCALE IN FEET
0 300 600

N

Legend

Subbasin B/C Structural Repair Priority 3

Figure 8 - 14
Subbasin B/C Structural Repairs
COLLECTION SYSTEM MASTER PLAN
CITY OF GRANTS PASS, OREGON

Similarly, the total project costs for the recommended structural repair areas have been prepared. A minimum diameter of 8-inch sewer pipeline has been assumed to estimate project cost. This is because replacing existing 6-inch-diameter sewer pipe with 6-inch does not meet generally accepted sewer design criteria.

Given that different repair/replacement technologies will be implemented in these areas, rather than just assume replacement of all old pipelines with new pipelines to estimate the total project costs in each of these areas, it was assumed that total cost would equal the cost to replace one-half of all pipelines in these areas. These costs are developed and presented in Table 8-3.

Table 8-3. Cost of Recommended Structural Repair Areas (in $1M)

Structural Repair Area	Pipeline Diameter (inches)	Length (feet)	Construction Cost ($/foot)	Base Construction Cost	Construction Contingency 40%	Engineering (Legal and Administration) 30%	Total[a] Estimated Cost
Pine Street	8	7,667	$150				
	10	402	$150	$1.22	$0.49	$0.51	$1.11
	12	48	$155				
5th Street	8	11,672	$150				
	10	1,244	$150	$2.34	$0.94	$0.98	$2.13
	12	2,606	$155				
7th Street	8	9,326	$150				
	10	400	$150				
	12	692	$155	$1.61	$0.64	$0.67	$1.46
	18	260	$170				
Subbasin B/C	8	7,409	$150				
	10	945	$150	$1.35	$0.54	$0.57	$1.23
	12	644	$155				
Subbasin H	8	7,919	$150				
	10	1,005	$150	$1.34	$0.53	$0.56	$1.21
Lawnridge-Washington	8	11,362	$150				
	10	612	$150	$1.80	$0.72	$0.76	$1.64
	12	16	$155				

[a] Total of base cost, construction contingency, and engineering divided by two.

8.5 CAPITAL IMPROVEMENT PLAN

Using the prioritization of Collection System Improvements described in Section 8.3 and the estimated cost of these improvements presented in Table 8-2 and Table 8-3, a recommended Capital Improvement Plan for the City of Grants Pass collection system has been developed and is presented in Table 8-4.

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Table 8-4. Recommended Capital Improvement Plan
Grants Pass Collection System

Project	Schedule	Cost ($1M)
Pine Street Sewer	2004–2006	$2.55
Western Avenue Sewer	2006–2009	$1.46
Pine Street Structural Repair	2009–2011	$1.11
5th Street Structural Repair	2010–2012	$2.13
7th Street Structural Repair	2013–2015	$1.46
Lawnridge-Washington Structural Repair		$1.64
Mill Street Sewer	2016–2018	$3.08
Subbasin B/C Structural Repair		$1.23
7th Street Relief System	2019–2021	$1.40
Subbasin H Structural Repair		$1.21
Nebraska Avenue Sewer	2022–2024	$0.81
Total Collection System Capital Improvement Plan:		**$18.08**

8.6 IMPLEMENTATION

The City of Grants Pass needs to begin to implement the CIP presented in Section 8.5 as soon as possible. The Pine Street sewer project has already been initiated and is currently in planning and engineering design. Construction is currently planned for 2005–2006 if sufficient funding is available.

Implementation of further improvements is, however, contingent upon developing an adequate funding source. The City should begin an evaluation of their sewer customer rate schedule to provide sufficient funding in the future to implement the remainder of the CIP. This evaluation should include a review of the current sewer rate structure by customer class and consideration of a collection system, system development change (SDC).

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REDWOOD SANITARY SEWER SERVICE DISTRICT
JOSEPHINE COUNTY

ENGINEERING REPORT

Prepared for:

City of Grants Pass
101 Northwest A Street
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and

Josephine County
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PMX #27-2192-05
April 1999
(Revised November 1999)

EXHIBIT 9 ~~813~~
to UAPC Staff Report

CERTIFICATE OF ENGINEER

The technical material and data contained in this document were prepared under the supervision and direction of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.



EXP 5/12/2000

Prepared by: Allan Maas, P.E.

Checked by: Tom Nielsen, P.E.

Approved by: Steven C. Gilbert, P.E.

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Executive Summary

Redwood Sanitary Sewer Service District

Josephine County, Oregon



Parametrix, Inc.

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EXECUTIVE SUMMARY

BACKGROUND

In the late 1960s, the Josephine County Board of Commissioners formed the Redwood Sanitary Sewer Service District (District) in an area 4.8 miles southwest of Grants Pass, Oregon. In 1977-1978, the District constructed a wastewater collection system and treatment plant in the service area. Since then, only minor additions or modifications to both the collection system and/or treatment plant have been made, with exception of a biosolids composting operation that was added to the plant in 1988.

In 1994, Parametrix completed a Wastewater Facilities Plan for the District. The Plan recommended that the plant be upgraded and expanded. However, this action was never implemented due to litigation to shut down the biosolids composting operation at the plant in 1994 and the Oregon Department of Environmental Quality (ODEQ) taking no action to approve the Facilities Plan.

Because the litigation on the composting operation has now resulted in a judgment requiring the compost closure, the existing Facilities Plan is no longer valid. Now the cost of upgrading/expanding the Redwood Wastewater Treatment Plant (Redwood WWTP) has changed. The cost of a new biosolids facility must be added. This means that it now appears that the preferred treatment alternative may no longer be to upgrade/expand the Redwood WWTP, but rather convey wastewater to the Grants Pass Water Restoration Plant (WRP) for treatment. The purpose of this report is to re-evaluate the cost of both of these alternatives.

SERVICE AREA

The current limits of the Redwood Sanitary Sewer Service District's service area encompass an area of approximately 3,480 acres, or 5.4 square miles. The service area lies generally south of the Rogue River, westerly of Allen Creek, northerly of the South Highline Canal of the Grants Pass Irrigation District, and easterly of Rounds Avenue. The service area also includes the Rogue Community College Campus, which lies south of this general area.

The estimated population served by the Redwood wastewater collection system, as of October 1998, is 4,905. The average number of persons per equivalent residential unit (ERU) in the District is 2.42. To estimate future wastewater loads and flows, projections of the population in the service area of the District were developed through a planning period ending in 2020. An annual growth rate of 3.1 percent per year was used based on a consensus between the County and the City of Grants Pass. This resulted in a service area population of 9,600 in 2020.

COLLECTION SYSTEM

The collection system serving the District consists of approximately 16.2 miles of sewers, ranging in size from 8 to 27 inches in diameter. The system experiences very high wastewater flow in the winter. Infiltration and inflow appear to be excessive. The District has agreed to address annual budget allocations for collection system I/I correction as a part of a Sewer System Master Plan to be completed by the District in 2001-02. In the meantime, it is recommended that the District budget $25,000 annually to address infiltration and inflow correction.

TREATMENT PLANT

The Redwood WWTP is located near the intersection of Leonard Road and Rounds Avenue in Josephine County. It is comprised of numerous unit treatment processes, a control/laboratory building, blower/sludge pumping building, and biosolids composting operation.

In 1994, a capacity analysis indicated that the Redwood WWTP's design capacity is 0.6 million gallons per day (mgd) with a peak flow capacity of 1.44 mgd. Because these design flows have been exceeded on occasion, continued operation of the plant requires it be upgraded/expanded.

FUTURE TREATMENT REQUIREMENTS

Based on 2020 population estimates, future wastewater flow and load estimates were developed; these are shown in Table ES-1 and Table ES-2, respectively. Also shown in these tables are flow and load conditions that currently exist at the plant.

Table ES-1
Wastewater Flow Projections, Year 2020

	1996/1998		2020	
	Actual (mgd)	1998 Actual (Gal/cap-day)	Design (mgd)	1998-2020 (Gal/cap-day)
Summer, Max. Month	0.54	120	1.01	100
Winter, Max. Month	1.16	240	1.77	130
Summer Max. Week	0.58	126	1.10	110
Winter Max. Week	1.20	254	1.90	150
Summer, Peak Day	0.76	157	1.43	140
Winter, Peak Day	1.48	364	2.77	275
Summer Average	0.49	99	0.89	85
Winter Average	0.63	129	1.22	125
Annual Average	0.56	114	1.05	105

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Table ES-2 Redwood WWTP BOD and TSS Load (lbs/day)		
	1996-1998 Actual	2020 Projection
BOD		
Summer, Max. 30-day average	1,122	2,400
Winter, Max. 30-day average	1,589	3,100
Annual average	843	2,070
TSS		
Summer, Max. 30-day average	1,398	2,340
Winter, Max. 30-day average	1,116	2,500
Annual Average	879	1,980

To properly evaluate upgrading/expanding the existing Redwood WWTP, an analysis of the facility's wastewater discharge permit limits was also made. Since the Redwood WWTP was built in 1978, wastewater discharge requirements have become more strict, particularly in the Rogue River that is used for salmon spawning and domestic water supply.

The level of treatment required for future wastewater flows and loadings was evaluated based on three different requirements:

- Treatment requirements contained in OAR 340-41-375.
- Treatment requirements contained in OAR 340-41-026.
- Treatment requirements based on receiving water quality criteria for the Rogue River.

These regulations would limit the BOD and TSS limits for the Redwood WWTP as shown in Table ES-3 and Table ES-4.

Table ES-3 Anticipated Future BOD and TSS Effluent Limitations Based on OAR 340-41-375							
	Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge (lbs/day)		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based On Effluent Quality							
Summer, MMDWF	1.06[1]	10	15	—	88	132	176
Winter, MMWWF	1.79[2]	30	45	—	450	675	900

(1) Monthly dry weather flow with 5-year recurrence – MMDWF.
(2) Monthly wet weather flow with 5-year recurrence – MMWWF.

Table ES-4
Anticipated Future BOD and TSS Effluent Limitations
Based on OAR 340-41-026

	Maximum Monthly Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge (lbs/day)		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based On Effluent Quality							
Summer, Dry Weather	1.06[1]	9	14	18	80	120	160
Winter, Wet Weather	1.79[2]	8	12	16	120	180	240

(1) Monthly dry weather flow with 5-year recurrence – MMDWF.
(2) Monthly wet weather flow with 5-year recurrence – MMWWF.

Since the mass discharge limitations based on OAR 340-41-026 are less than those based on OAR 340-41-375, the stricter limits would normally apply. The District could, however, request ODEQ to allow discharge limits based on the OAR 340-41-375 requirements.

Increases have been accepted by ODEQ, provided that:

(1) No other alternatives exist except to lower water quality.

(2) This action is necessary and justifiable for economic or social development benefits and outweighs the environmental costs of lowered water quality.

(3) All water quality standards will be met and beneficial uses protected.

The District would need to formally apply to ODEQ for a permit modification to increase the mass discharge limits to those based on OAR 340-41-375.

In addition to upgrading the plant to meet the projected flow, load, and new BOD and TSS limits, the Redwood WWTP would also need to be modified to achieve ammonia removal and eliminate effluent chlorine discharge.

TREATMENT ALTERNATIVES

To evaluate the best method for meeting the District's wastewater treatment needs to the year 2020, twelve alternatives were analyzed. These alternatives were separated into three groups, the first and second of which use the Redwood WWTP for treatment. The third group uses the City of Grants Pass WRP for treatment.

- Alternatives 1, 2, and 3 would provide secondary treatment at the existing Redwood WWTP with some ammonia removal. These alternatives would meet the effluent limitations of OAR 340-41-375 and would require a special discharge load limit to the Rogue River by ODEQ. These alternatives included:

  - Construct a new contact stabilization with anoxic selectors process
  - Construct a new sequencing batch reactor (SBR) process
  - Add a trickling filter and modify the existing process

- Alternatives 4 and 5 would provide tertiary treatment at the existing Redwood WWTP and would not require a special discharge load limit to the Rogue River. These two alternatives would meet the effluent limitations of OAR 340-41-026. These alternatives included:

  - Construct a new anoxic selector process with effluent filtration
  - Construct a new SBR process with effluent filtration

These five alternatives were the most cost-effective ways to upgrade/expand the Redwood WWTP.

An important consideration when evaluating these treatment alternatives was biosolids treatment. Biosolids are a byproduct of wastewater treatment; they are generated by the process during removal of pollutants from the wastewater. The biosolids generated at the existing Redwood WWTP are currently aerobically digested then further treated in the composting operation.

Because of litigation against the composting operation, however, the District must close operation of the composting facility this year. Therefore, an alternative biosolids treatment method must be included with each of the treatment alternatives using the Redwood WWTP. Because the Grants Pass WRP currently uses anaerobic digesters, this alternative was selected.

Adding new anaerobic digesters to each of the five Redwood WWTP treatment alternatives, however, added significant cost to each of these alternatives. The estimated probable cost of each of these alternatives are shown in Table ES-5.

Table ES-5
Treatment Alternatives 1 through 5
Estimate of Probable Costs (costs in $1,000s)

Wastewater Process	Alt. 1 Contact Stabilization	Alt. 2 SBR	Alt. 3 Trickling Filter	Alt. 4 Complex Mix Anox Select Filter	Alt. 5 SBR Filter
Biofilters	--	--	$719	--	--
Aeration Basin	$1,239	--	--	$1,742	--
Secondary Clarifier	$513	--	$513	$513	--
SBR Basin	--	$1,717	--	--	$1,585
Blower Building	$129	--	--	$129	$123
Primary Sedimentation	--	--	$586	--	--
Package Filtration	--	--	$409	$409	$409
Effluent EQ	--	$95	--	--	$95
Influent Pump Stn Mods	$113	$148	$115	$113	$148
Headworks Mods	$121	$161	$56	$121	$161
UV Disinfection	$315	$315	$315	$315	$315
Site/Civil	$72	$72	$72	$72	$72
Yard Pipe	$72	$72	$72	$108	$108
Electrical	$433	$433	$503	$559	$475
Subtotal	$3,008	$3,014	$3,356	$4,082	$3,491

Table ES-5 (cont.)
Treatment Alternatives 1 through 5
Estimate of Probable Costs (costs in $1,000s)

Wastewater Process	Alt. 1 Contact Stabilization	Alt. 2 SBR	Alt. 3 Trickling Filter	Alt. 4 Complex Mix Anox Select Filter	Alt. 5 SBR Filter
Biosolids Process	$2,590	$2,590	$2,590	$2,590	$2,590
Contingency, 30%	$1,679	$1,681	$1,784	$2,002	$1,824
Subtotal Construction	$7,278	$7,285	$7,730	$8,673	$7,906
Engineering and Admin, 22%	$1,601	$1,603	$1,701	$1,908	$1,739
Total Project Cost[1]	$8,880	$8,890	$9,430	$10,580	$9,650

(1) Rounded to three significant figures.

An estimate of the annual operation and maintenance (O&M) cost of each of these alternatives is shown in Table ES-6, together with the annual O&M of the existing plant.

Table ES-6
Treatment Alternatives 1 through 5
Annual O&M Costs

	Existing Plant	Alt. 1 Contact Stabilization	Alt. 2 SBR	Alt. 3 Biofilter	Alt. 4 Comp Mix Effluent Filter	Alt. 5 SBR with Effluent Filter
Salary and Wages	$55,500	$73,800	$73,800	$73,800	$84,300	$84,300
Employee Benefits	$18,949	$25,250	$25,250	$25,250	$28,850	$28,850
Supply and Material	$24,808	$27,808	$26,400	$29,800	$31,500	$31,000
Services	$55,835	$77,235	$76,800	$80,500	$82,500	$81,500
Interfund and Inter Gov.	$123,254	$136,700	$135,500	$144,000	$146,000	$145,000
Capital Replace/ Improvements	$39,870	$49,370	$49,370	$51,400	$55,400	$55,400
Subtotal	$318,216	$390,163	$387,120	$404,750	$428,550	$426,050
Anaerobic Digestion	$193,760	$193,760	$193,760	$193,760	$193,760	$193,760
Total O&M Cost[1]	$512,000	$584,000	$581,000	$599,000	$622,000	$620,000

(1) Rounded to three significant figures.

- Alternatives 6 through 12 provide for pumping and transmission facilities to convey all the wastewater to the Grants Pass WRP for treatment. This group of alternatives evaluated conveying wastewater to Grants Pass instead of upgrading the existing Redwood WWTP. The Grants Pass WRP has adequate treatment

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capacity available. Each of the conveyance alternatives considered a different sewer force main route between the existing Redwood WWTP and the Grants Pass WRP. These are described as follows:

Alternative 6: Dual 12-inch force mains would be bored under the Rogue River just north of Redwood WWTP, then the force mains would follow the right-of-way of Lower River Road and Western Lane to the Grants Pass WRP.

Alternative 7: A 6-inch force main would be placed in an existing sewer interceptor easement between Redwood WWTP and pump station RI-25. From there dual 12-inch mains would be bored under the Rogue River. From the Lathrop boat landing, the force mains would follow the right-of-way of Lower River Road and Western Lane to the Grants Pass WRP.

Alternative 8: The force main would be placed in the existing sewer interceptor easement between Redwood WWTP and the proposed location of the pedestrian bridge near the Grants Pass WRP. A 6-inch main would be installed to pump station RI-25, then dual 12-inch mains go from there to the Grants Pass WRP. The force mains would be attached to the pedestrian bridge.

Alternative 9: A 6-inch force main would be placed in the existing sewer interceptor easement between Redwood WWTP and pump station RI-25. From there dual 12-inch force mains would follow the right-of-way of Leonard Road and Redwood Avenue to the pedestrian bridge, then to the Grants Pass WRP.

Alternative 10: A 6-inch force main would be placed in the right-of-way of Leonard Road up to pump station RI-25. From there dual 12-inch mains would be bored under the Rogue River and follow the right-of-way of Lower River Road and Western Lane to the Grants Pass WRP.

Alternative 11: A 6-inch force main would be placed in the right-of-way of Leonard Road up to pump station RI-25. From there dual 12-inch force mains would be placed in the existing sewer interceptor easement to the pedestrian bridge, and then to the Grants Pass WRP.

Alternative 12: A 6-inch force main would be placed in the right-of-way of Leonard Road up to pump station RI-25. From there dual 12-inch force mains would follow the right-of-way of Leonard Road and Redwood Avenue to the pedestrian bridge, then to the Grants Pass WRP.

The estimated probable cost of each of these alternatives are shown in Table ES-7, together with their associated force main length and number of anticipated easements required.

Table ES-7
Treatment Alternatives 6 through 12
Estimate of Probable Cost

Treatment Alternative	Pipeline Length	# Ease.	Pipeline	Pump Station	Contingency 30%	Engineering & Admin. 25%	Total[1]
6	26,500	1	$3,240,000	$1,182,000	$1,326,600	$1,105,500	$6,850,000
7	24,000	48	$2,960,000	$1,284,000	$1,273,000	$1,061,000	$6,580,000
8	27,000	77	$2,780,000	$1,284,000	$1,219,200	$1,016,000	$6,300,000
9	29,300	47	$3,000,000	$1,284,000	$1,285,200	$1,071,000	$6,640,000
10	28,600	3	$2,930,000	$1,284,000	$1,264,200	$1,053,500	$6,530,000
11	31,100	32	$3,080,000	$1,284,000	$1,309,200	$1,091,000	$6,700,000
12	33,400	1	$3,230,000	$1,284,000	$1,354,200	$1,128,500	$6,700,000

(1) Rounded to three significant figures.

Also, an estimate of the annual O&M cost of each of these alternatives is shown in Table ES-8 (O&M costs for these alternatives are similar).

Table ES-8
Treatment Alternatives 6 through 12
Annual O&M Costs

Conveyance O&M	Annual Cost
Salary and Wages	$18,000
Employee Benefits	$6,300
Chemicals and Material	$50,000
Power	$11,000
Collection System	$15,000
Subtotal:	**$100,300**
Portion of Grants Pass WWTP O&M	**Annual Cost**
WWTP O&M Cost	$88,500
Billing Services	$22,000
Customer Service	$20,000
Subtotal:	**$130,500**
Grand Total:	**$230,800**

PREFERRED ALTERNATIVE SELECTION

In order to reduce the complexity of evaluating twelve different treatment alternatives for the District, two preliminary screening processes were initially conducted based on cost and impact to residents adjacent to the conveyance pipeline. Treatment Alternatives 1 through 5 were screened to Alternative 1 – Contact Stabilization and Alternative 4 – Complete Mix/Anoxic Selector/Filter. Treatment Alternatives 6 through 12 were screened to Alternative 7 – Easement/Lower River Road and two refinements of Alternative 9, Alternative 9A – Easement/Leonard Road and Alternative 9B – Easement/South River Road. With these last three alternatives, to ensure that the minimum and maximum wastewater flows were properly handled, a dual 12-inch-diameter force main system was included. Two of these alternatives (9A and 9B) included attaching a force main to the proposed pedestrian bridge over the Rogue River near the Grants Pass WRP. Alternative 7 included boring a force main under the Rogue River.

Based on preliminary design criteria and estimated O&M costs, present worth cost estimates for these five remaining treatment alternatives were developed and are shown in Table ES-9.

Table ES-9
Treatment Alternatives 1, 4, 7, 9A, and 9B
Present Worth Cost Estimates (in $1,000s)

	Treatment Alternatives		Conveyance Alternatives		
	Alternative 1 Contact Stabilization	Alternative 4 Complex Mix Effluent Filter	Alternative 7 Easement/ Lower River Road	Alternative 9A Easement/ Leonard Road	Alternative 9B Easement/ South River Road
Subtotal Construction	$7,280	$8,670	$5,520	$5,600	$5,510
Engineering and Admin.	$1,600	$1,910	$1,060	$1,160	$1,150
Total Project Cost	**$8,880**	**$10,580**	**$6,580**	**$6,760**	**$6,660**
Present Worth O&M Cost[1]	$6,870	$7,320	$2,710	$2,710	$2,710
Total Present Worth	**$15,750**	**$17,900**	**$9,280**	**$9,470**	**$9,370**

(1) Present worth based on 6 percent interest for 22 years.

For each of the conveyance alternatives, the City of Grants Pass omitted charging the District a connection charge or System Development Charge (SDC) for treatment at the City's plant. However, even if the City had charged a fee, it would not change the least cost alternative.

As an example, if the City were to charge District customers the SDC currently collected by the District for new developments ($1,966 per ERU), approximately four million dollars would be added to the capital and present worth cost of each conveyance alternative. For Alternatives 7, 9A, and 9B the present worth cost would be $13.28, $13.47, and $13.37 million, respectively. In comparison, the present worth cost of Treatment Alternatives 1 and 4 are $15.75 and $17.9 million, respectively, which are considerably greater. Still, the least cost would be any of the conveyance alternatives.

The next step in the evaluation of these alternatives was to develop a matrix of selection criteria. To help determine the preferred alternative, a list of 22 selection criteria were chosen and then grouped into four categories: Cost Issues, Environmental Issues, Community Issues, and Operation Issues. Two Josephine County representatives, six City of Grants Pass representatives, and the City's attorney made up the committee that ranked each alternative for each of the selection criteria. Through consensus of the participants, the two alternatives (Alternatives 7 and 9B) were then selected as the best alternatives.

These two alternatives were then presented to the Grants Pass City Council, acting as the governing Board for the District, to make their preferred alternative selection. After careful deliberation, the Council selected Treatment Alternative 9B – Easement/South River Road as the preferred alternative.

PREFERRED ALTERNATIVE

The preferred Treatment Alternative 9B – Easement/South River Road consists of the following components which are shown in Figure 1:

- Retrofit the Redwood WWTP influent pump station (RI-0) into a 0.4 mgd capacity submersible pump station and construct a 6-inch force main to RI-25 which would follow the existing interceptor easement route.

- Construct a new pump station north of Darneille Lane (near manhole RI-25) with a capacity of 4.2 mgd and dual 12-inch force mains which would follow South River Road then Leonard Road to the intersection of Leonard Road and Dowell Road. From there, the force mains would generally follow the existing interceptor easement route to the south end of the proposed new pedestrian bridge.

- Attach a single 14-inch force main to the proposed Rogue River pedestrian bridge to be located north of the Josephine County Fairgrounds.

- Construct dual 12-inch force mains from the north end of the proposed pedestrian bridge to the Grants Pass WRP.

To construct this alternative, the total project cost is estimated at $6.7 million. A project schedule to implement this treatment alternative was developed and is shown on Figure 2. The new pump stations and force mains could be operational by mid-October 2000. This is also the projected completion date for the new pedestrian bridge.

FINANCING PLAN

Various project funding scenarios were evaluated to determine the best way for the District to finance this $6.7 million project. After reviewing several scenarios, a preferred alternative was selected. This alternative is to fund the project with approximately $5.4 million of State Revolving Fund (SRF) loan money and approximately $1.3 million of available cash from the District. The District has an additional $0.7 million cash that will be needed for project reserves. With this funding approach, no sewer rates or system development charge increases would be necessary in the District in the next several years.

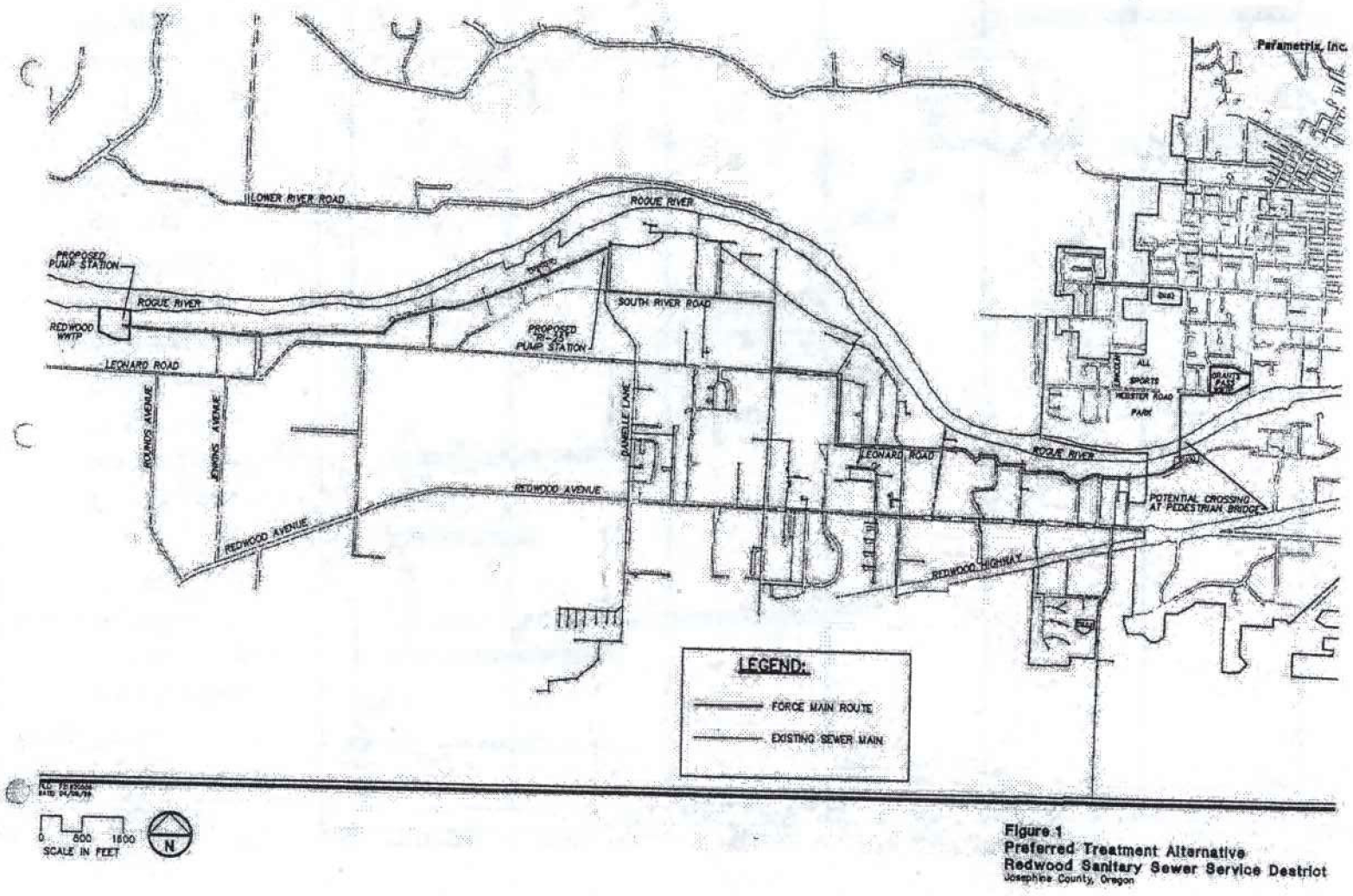

Figure 1
Preferred Treatment Alternative
Redwood Sanitary Sewer Service Destriot
Josephine County, Oregon

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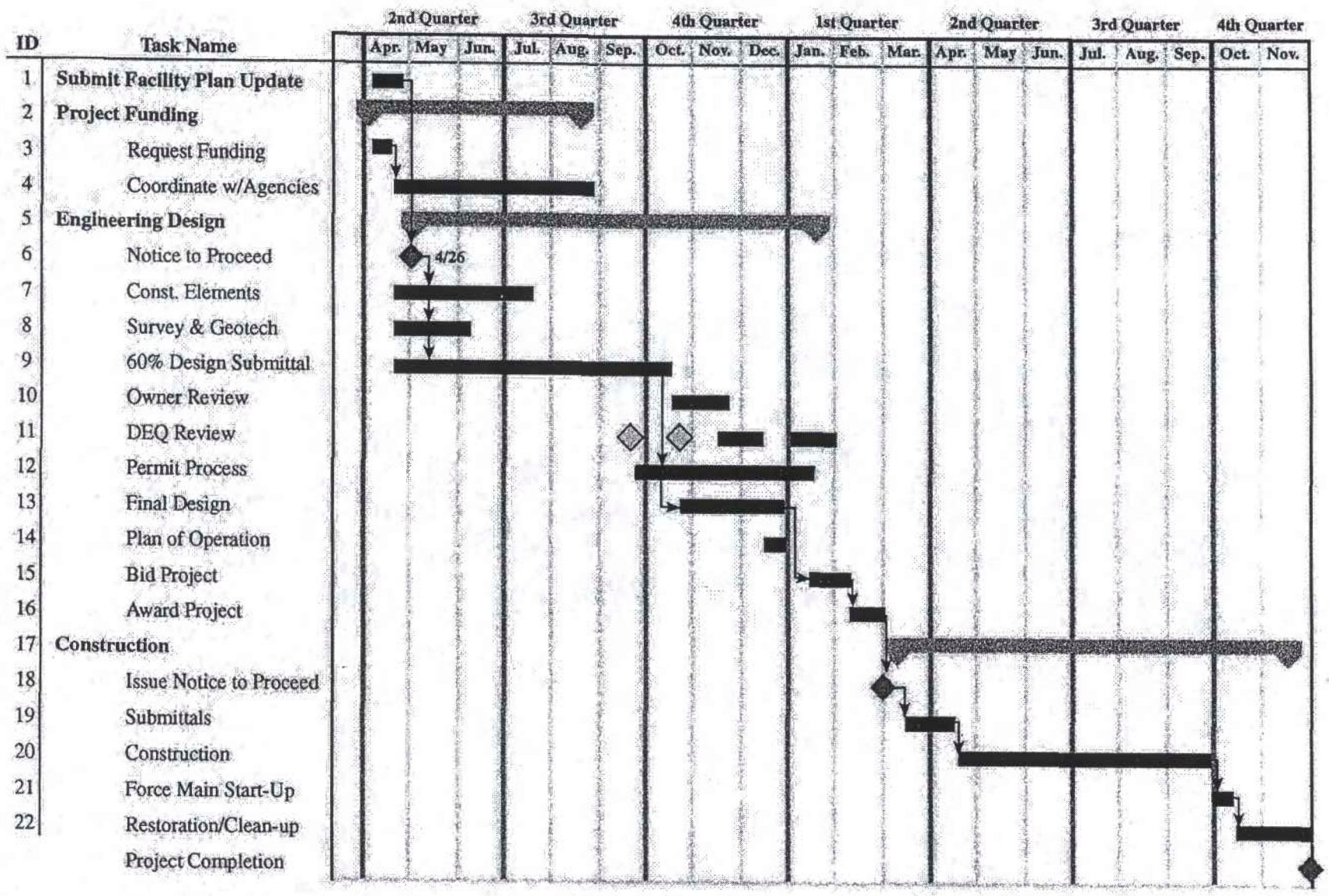
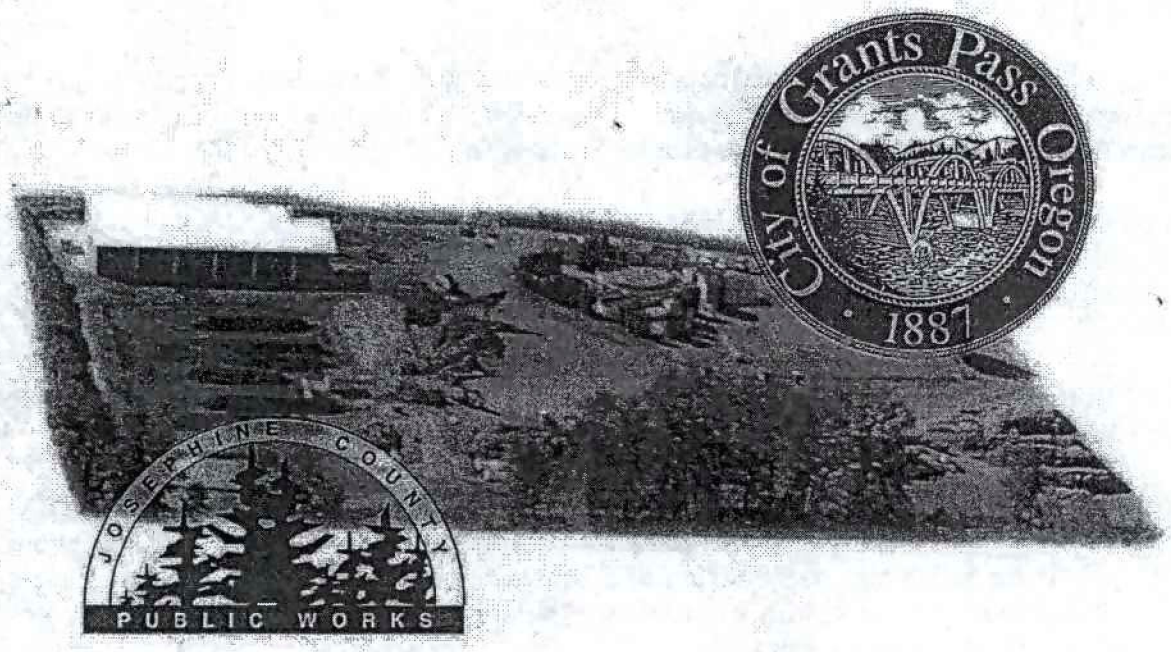


Figure 2
Project Implement Schedule
Preferred Treatment Alternative

Introduction

Redwood Sanitary Sewer Service District

Josephine County, Oregon



Parametrix, Inc.

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1. INTRODUCTION

1.1 BACKGROUND

In the late 1960s, the Josephine County Board of Commissioners formed the Redwood Sanitary Sewer Service District (District) in an area 4.8 miles southwest of Grants Pass, Oregon (Figure 1-1). The Commissioners' action resulted from a publication ("Engineering Report on a Preliminary Study of Sewage Collection and Waste Treatment Facilities for the Redwood Avenue Area") completed by the County in 1966.

In 1977-78, the District constructed a wastewater collection system and treatment facility in the service district to discharge treated wastewater to the Rogue River. Since 1978, only minor additions to the collection system have been made, and the treatment facility has also had only minor changes, except for the addition (in 1988) of a biosolids composting facility, located adjacent to the plant.

In June 1992, Josephine County executed an agreement with Parametrix, Inc. to prepare a Wastewater Facilities Plan for the District. This action was prompted by a condition in the District's National Pollutant Discharge Elimination System (NPDES) Waste Discharge Permit.

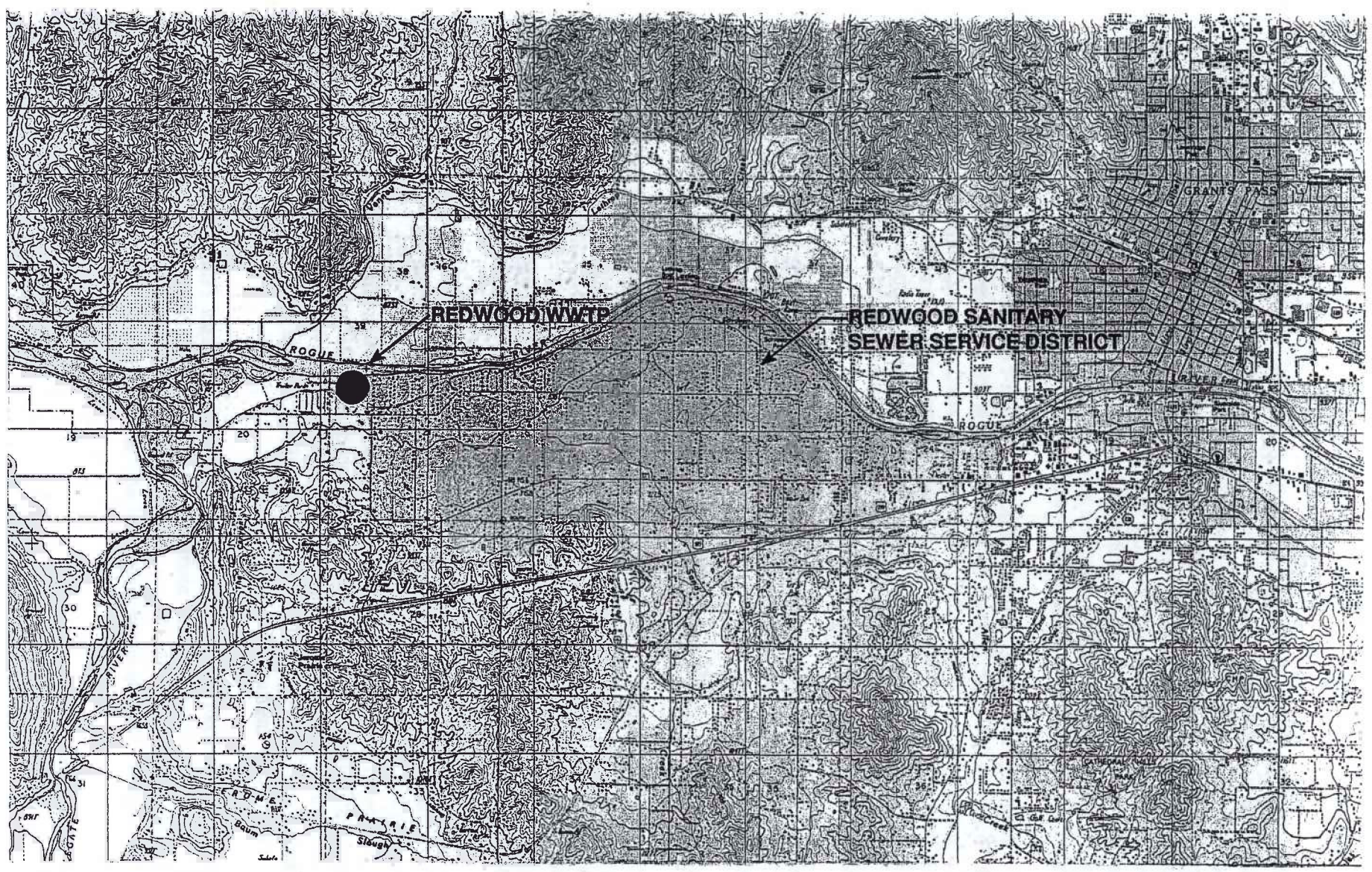
The NPDES permit stipulated that . . . *"As soon as practicable, but not later than two (2) years after permit issuance date, the permittee (District) shall submit a final facility plan report describing growth projections, plant expansion, funding mechanisms, and an implementation schedule."*

The Oregon Department of Environmental Quality (ODEQ) issued Permit Number 100868 on March 24, 1992. A copy is included in Appendix A.

In November 1994, the Wastewater Facilities Plan was completed and submitted for ODEQ approval. The Plan recommended that the plant be upgraded and expanded. These actions would include the following: construction of a new aeration basin and secondary clarifier; a headworks upgrade; the conversion of an existing aeration basin to an aerobic digester; and installation of ultraviolet disinfection.

ODEQ approved this Facilities Plan in July 1995. Following approval, final design of the plant upgrade/expansion began.

In August 1994, neighbors near the treatment plant initiated a court action against the District seeking to shut down the biosolids composting portion of the treatment facility. In 1995, the Circuit Court for Josephine County (Case No. 94-CV-0209) resolved this complaint in a judgment (recorded September 5, 1995) against the District requiring that the composting facility cease operation. However, the District appealed the judgment.



Redwood Waste Water Treatment Plant
#27-2192-05 02/99

Figure 1-1
Redwood Sanitary Sewer Service District and Grants Pass

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Meanwhile, in fall of 1995, the final design of the plant upgrade/expansion was proceeding. In February 1996, a fifty percent complete design review meeting was scheduled; however, at the meeting, instead of reviewing the fifty percent complete design, ODEQ informed the District that approval of the Wastewater Facilities Plan was not possible at this time. This action was prompted by ODEQ's concern for a proposed mass load increase in the Facilities Plan. Having revisited the proposed increase (prior to the meeting), ODEQ believed additional studies were necessary to support approval of the increase rather than their original belief that the plan could be approved without these studies. Based on this decision, the upgrade/expansion design of a Redwood Wastewater Treatment Plant (Redwood WWTP) was stopped.

In 1996, Parametrix completed for the District the *Biosolids Treatment Evaluation* that reviewed biosolids treatment options that could be implemented at the treatment plant if the composting operation were to be closed. In 1997, Parametrix prepared a *Mass Load Increase Evaluation Report* to document for ODEQ the District's required mass load increase case. Final design work was stopped pending the outcome of the District's appeal on compost operation closure and pending ODEQ's approval of the District's request for a mass load increase for their treatment plant and then reapproval of their Facilities Plan.

In the fall of 1998, the District's appeal was denied and compost operation was ordered to close by October 1999. The District's mass load increase request remains to be reviewed and approved by ODEQ.

1.2 AUTHORITY AND PURPOSE

One of the alternatives investigated in the original facilities planning process was abandonment of the existing Redwood WWTP and wastewater conveyance to the City of Grants Pass Water Restoration Plant (WRP). (This alternative was evaluated in a separate report entitled, *"Feasibility Analysis, Wastewater Conveyance to City of Grants Pass Wastewater Treatment Plant,"* prepared by Parametrix, Inc. and dated June 1994.) This concept was not considered feasible at that time because it was more costly than upgrading/expanding the Redwood WWTP.

Because composting is no longer permitted at the Redwood WWTP site, a completely new biosolids treatment and disposal process needs to be included in the Redwood WWTP upgrade/expansion cost and estimates. Addition of a new biosolids facility to the Redwood WWTP increases the cost of the WWTP upgrade/expansion alternative substantially. It now appears that the preferred alternative may be to abandon the Redwood WWTP and convey all wastewater to the City of Grants Pass WRP. Abandonment of the Redwood WWTP and conveyance of the wastewater to the City of Grants Pass WRP would likely become the least cost alternative and the best alternative to implement.

The purpose of this report is to update the Wastewater Facilities Plan for the District and reevaluate the cost of a conveyance alternative compared to the cost to upgrade/expand the existing Redwood WWTP. The conveyance alternative would include abandonment of the existing Redwood WWTP and construction of a new sewer force main and pump station system

to convey wastewater to the City of Grants Pass WRP. This system would generally consist of two pump stations and pipelines as originally identified in the above-stated 1994 "Feasibility Analysis" report as "Alternative 3D."

The purpose of this report is to also present a detailed predesign study of the preferred alternative to clearly identify and resolve any and all potential predesign issues.

1.3 PREVIOUS STUDIES AND REPORTS

During the engineering analysis conducted in this study and during preparation of this document, the following studies and reports were used.

- Parametrix, Inc., August 1997. *Mass Load Increase Evaluation Report*. Sumner, Washington
- Parametrix, Inc., October 1996. *Biosolids Treatment Evaluation*. Sumner, Washington
- Parametrix, Inc., January 1996. *Redwood Wastewater Treatment Plant Improvements*. Redwood Sanitary Sewer District, Josephine County, Oregon.
- Parametrix, Inc., November 1994. *Redwood Sanitary Sewer Service District Wastewater Facilities Plan*. Sumner, Washington.
- Parametrix, Inc., November 1992. Report - Task I Priority Engineering/ Financial Analysis. Sumner, Washington.
- CH2M, June 1996. Engineering Report on a Preliminary Study of Sewage Collection and Waste Treatment Facilities for the Redwood Avenue Area.
- CH2M, February 23, 1970. Letter to Josephine County Board of County Commissioners regarding Redwood Sanitary Sewer Service District.
- CH2M, July 1976. Contract documents for the Construction on the Redwood Sanitary Sewer Service District Wastewater Collection System and Treatment Plant.
- ADS Environmental Service, 1993. Redwood Sanitary Sewer Service District Assessment of I/I for Facility Planning for Josephine County, Oregon.
- Redwood Wastewater Treatment Plant, 1989-1993. Daily Monitoring Report Forms and Wastewater Flow Chart.

- Brown and Caldwell, December 1992. Facilities Plan, City of Grants Pass – Water Restoration Plant.

- Redwood Sanitary Sewer Service District, 1989. Sludge Management Plan. Josephine County, Oregon.

- Preliminary Feasibility Report for a Yard and Wood Waste Recycling Program in Josephine County, Oregon, 1990.

- Federal Emergency Management Agency, June 1982. Floodway-Flood Boundary and Floodway Map, Panel 217 of 500. Josephine County, Oregon.

- Josephine County Public Works, July 1985. Redwood Service District, Sanitary Sewer Collection System Map.

1.4 ACKNOWLEDGMENTS

The authors gratefully acknowledge the courtesy and assistance received from the Josephine County Public Works staff and the entire staff of the City of Grants Pass who provided information essential to the preparation of this report. They also acknowledge the assistance received from the District's staff, particularly the treatment plant operators.

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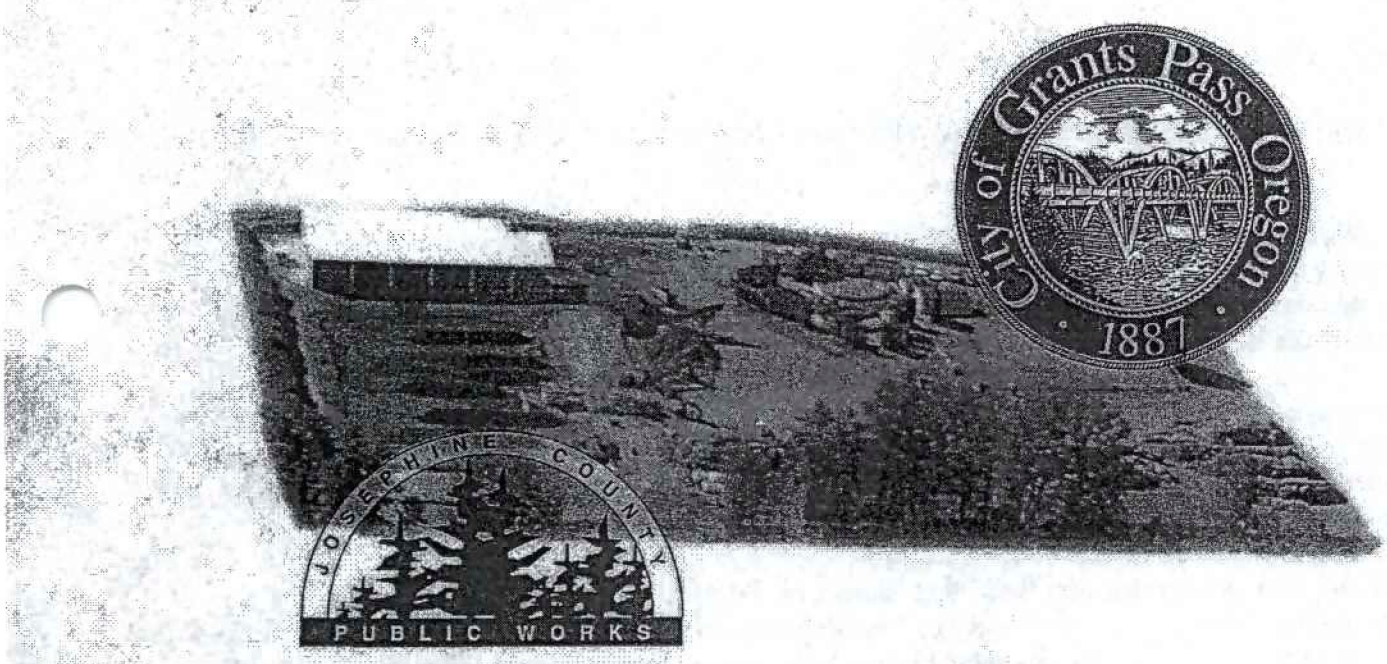
Service Area

Redwood Sanitary Sewer Service District

Josephine County, Oregon



Parametrix, Inc.

2. SERVICE AREA

2.1 INTRODUCTION

Service area characteristics strongly influence the quantity and quality of the wastewater generated in that area. Physical characteristics such as the extent of precipitation and groundwater influence the quantity of wastewater requiring treatment, and they influence the concentration of pollutants in the wastewater.

Land uses and the types of customers in a service area also influence the type and quantity of pollutants in the wastewater. Socioeconomic factors can not only determine whether a service area requires expansion to serve additional area and customers, but also when that expansion should occur.

2.2 DESCRIPTION OF AREA

The District's service area encompasses approximately 3,480 acres, or 5.4 square miles (Figure 2-1).

The service area lies south of the Rogue River, west of Allen Creek, north of the South Highline Canal of the Grants Pass Irrigation District, and east of Rounds Avenue. The service area also includes the Rogue Community College campus, which lies southwest of this general area and south of the Redwood Highway.

In the same drainage basin as the service area but to the south and west are an additional 3,000 acres, or approximately 4.7 square miles. This area does not have sewer service at present and service is not expected in the foreseeable future. The region is not part of the service area at this time.

In addition to the service area boundary shown on Figure 2-1, two other boundaries can be found on this figure. Josephine County's Planning Department Urban Growth Boundary Limits are also shown as is the service area of the existing wastewater collection system. This latter boundary defines the area of the original assessment district. In 1977, properties within this area were assessed to finance the original wastewater collection system and treatment plant built in 1977-78.

2.3 TOPOGRAPHY

The District lies in the Rogue River Valley, a broad and relatively flat valley that slopes an average of 1 to 2 percent towards the River. Elevations within the district range between 880 and 1,000 feet above mean sea level. The Rogue River traverses the valley in a general east-west direction with an average gradient of 6 feet per mile.

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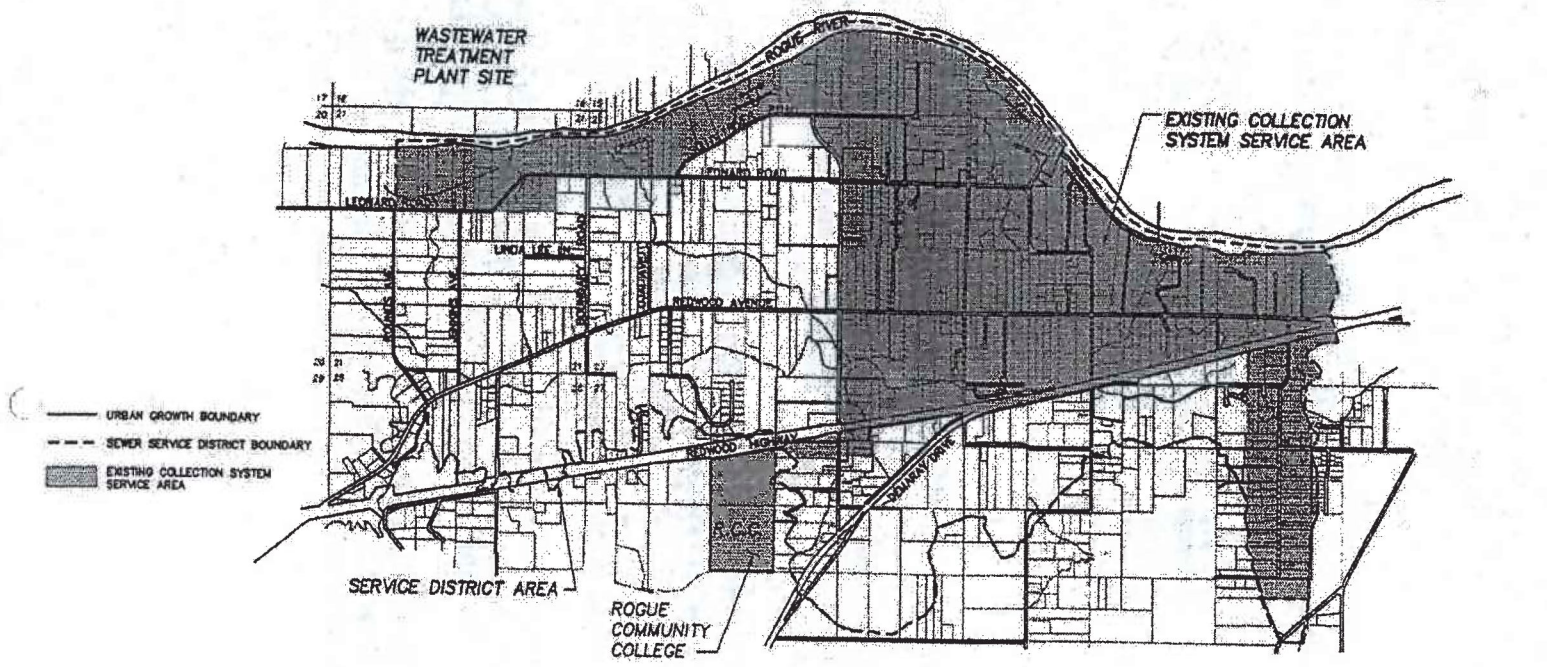

Figure 2-1
Service Area

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2.4 PRECIPITATION

The nearest National Oceanographic and Atmospheric Administration (NOAA) station to the District is located in the City of Grants Pass. This station records daily rainfall and temperature data; it also stores the data gathered for over 90 years. Annual precipitation rates at this station during the past 13 years appear in Figure 2-2; they indicate that nearly 75 percent of the annual rainfall occurs between November and March of each year (Figure 2-3).

Additionally, historical data evaluated by NOAA shows what the following statistical storm events occurring in the Grants Pass area will generate in inches of precipitation:

2-year, 24-hour	3 inches
5-year, 24-hour	3.5 inches
10-year, 24-hour	4 inches
25-year, 24-hour	5 inches
50-year, 24-hour	5.5 inches
100-year, 24-hour	6 inches

These data are provided to help evaluate wastewater flows in the collection system during storm events.

2.5 GRANTS PASS IRRIGATION DISTRICT

The District is located within the Grants Pass Irrigation District. This is significant because the irrigation district annually floods the irrigation canals within the sewer district, and this action affects local groundwater levels.

The Grants Pass Irrigation District was formed in the early 1920s to supply irrigation water to land located between the town of Rogue River and the confluence of the Applegate and Rogue Rivers. Currently, about 7,700 acres of agricultural and residential lands are irrigated. The irrigation district has water rights to divert up to 150 cfs from the Rogue River during the irrigation season, which normally falls between April 15 and October 1 of each year.

During this time, canals constructed by the irrigation district are flooded with water from the Savage Rapids Dam. Many sewer service area residents use the canal water to irrigate their land and garden areas. The canals also convey stormwater away from these adjacent lands.

The net effect of these irrigation practices is to significantly raise the groundwater table in the area of the canals; however, the extent of any irrigation impacts on the groundwater table is not defined. Generally, the groundwater table in this area is higher than normal during the irrigation season.

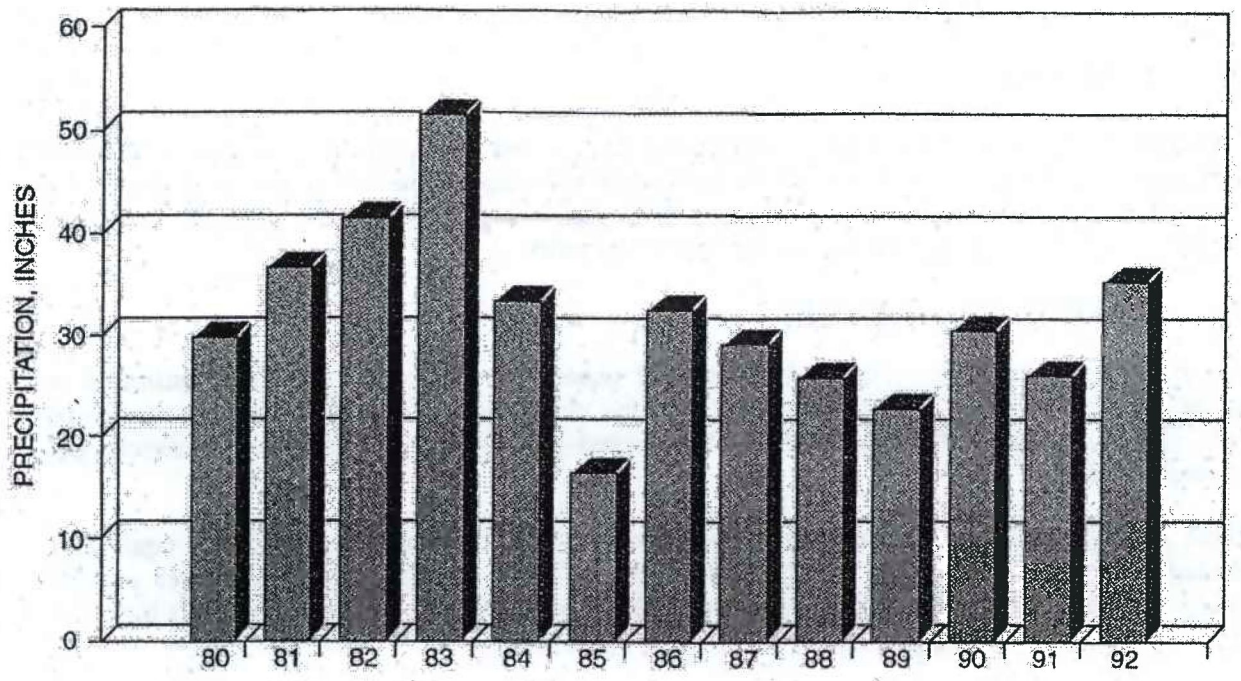

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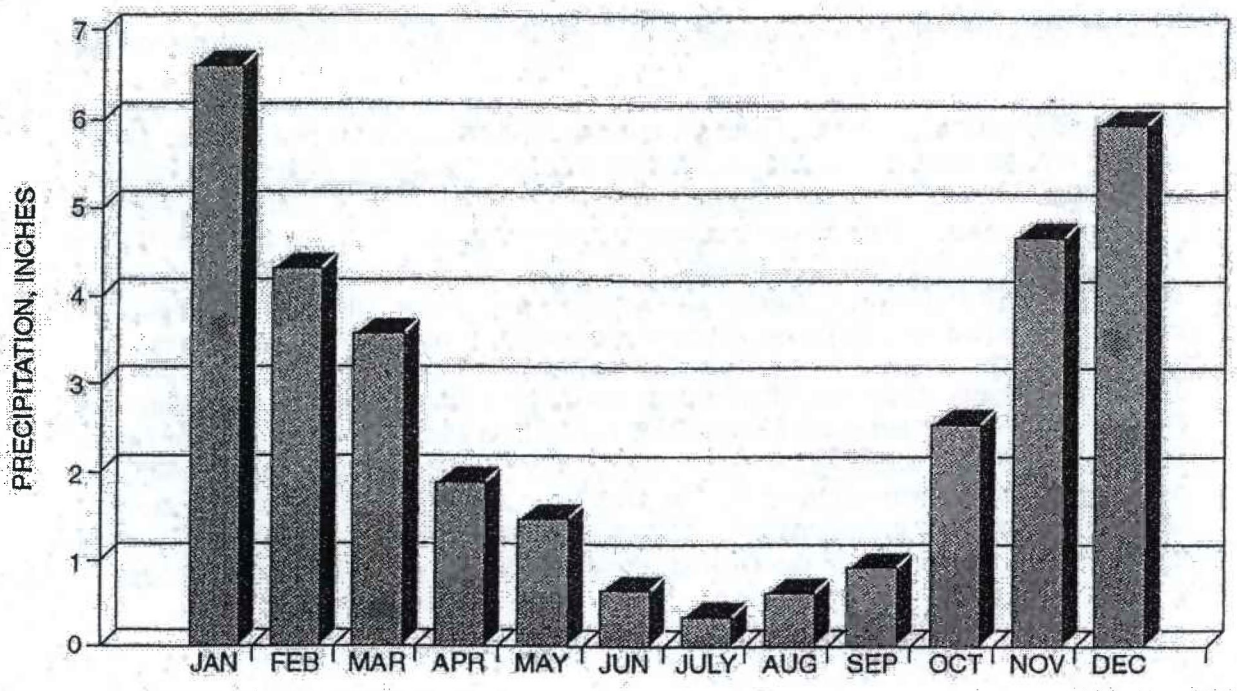
Figure 2-2
Yearly Average Precipitation



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NOTE: MONTHLY AVERAGE PRECIPITATION BASED ON NOAA (NATIONAL OCEANOGRAPHIC & ATMOSPHERIC ADMINISTRATION) RECORDS FROM 1980 TO 1992

Figure 2-3
Monthly Average Precipitation

2.6 LAND USE

The District currently consists of predominately single- and multi-family residential homes and agricultural land use. Such land use is consistent with current land use planning documents within the area. Therefore, no change in land use is anticipated other than the continued development of single- and multi-family residential units.

2.7 EXISTING POPULATION

The current population served by the Redwood wastewater collection system is estimated at 4,905 (October 1998). This figure was calculated using the number of equivalent residential units (ERUs) or single-family dwelling units served and the average number of persons per residential unit.

Since its origination, the District has issued sewer permits for approximately 2,027 equivalent residential units as of October 1998. This figure is based on a review of all sewer permits issued. It also includes the Rogue Community College, which was assigned 48 ERUs based on one ERU per 18 fixture units.

Based on census data from the Josephine County Planning Department for the Redwood District area, the average number of persons per residential unit in the District is 2.42.

2.8 POPULATION PROJECTIONS

To estimate future wastewater loads and flow, population projections in the Redwood District service area were developed through a planning period ending 2020 (Figure 2-4). Also, projections of ERUs in the service area were developed using 2.42 persons per residential unit.

Population and ERU projections for four scenarios were made, based on the following varying growths:

The District's Estimate – The District's estimate has been based on two criteria: (1) the number of new sewer connections made each year for the last 11 years was evaluated, and (2) the increase in student enrollment in School District No. 7 over the last several years was evaluated. Data on these criteria appear in Appendix B. The average increase in sewer connections was 5.4 percent each year. The average increase in school enrollment since 1993 was calculated at 3.9 percent each year. The District's population estimate was based on an average of these two numbers, or 4.9 percent each year.

Josephine County Planning Department Estimate – In response to comments in a December 10, 1998, letter from the ODEQ to William Peterson of the City of Grants Pass regarding the District's growth rate, the County Planning Department developed its own population growth estimate for the District. A copy of the County's estimate worksheet is included in Appendix B. This estimate, based on GIS records documenting the number of new homes in the District, projected growth at 3.41 percent per year.

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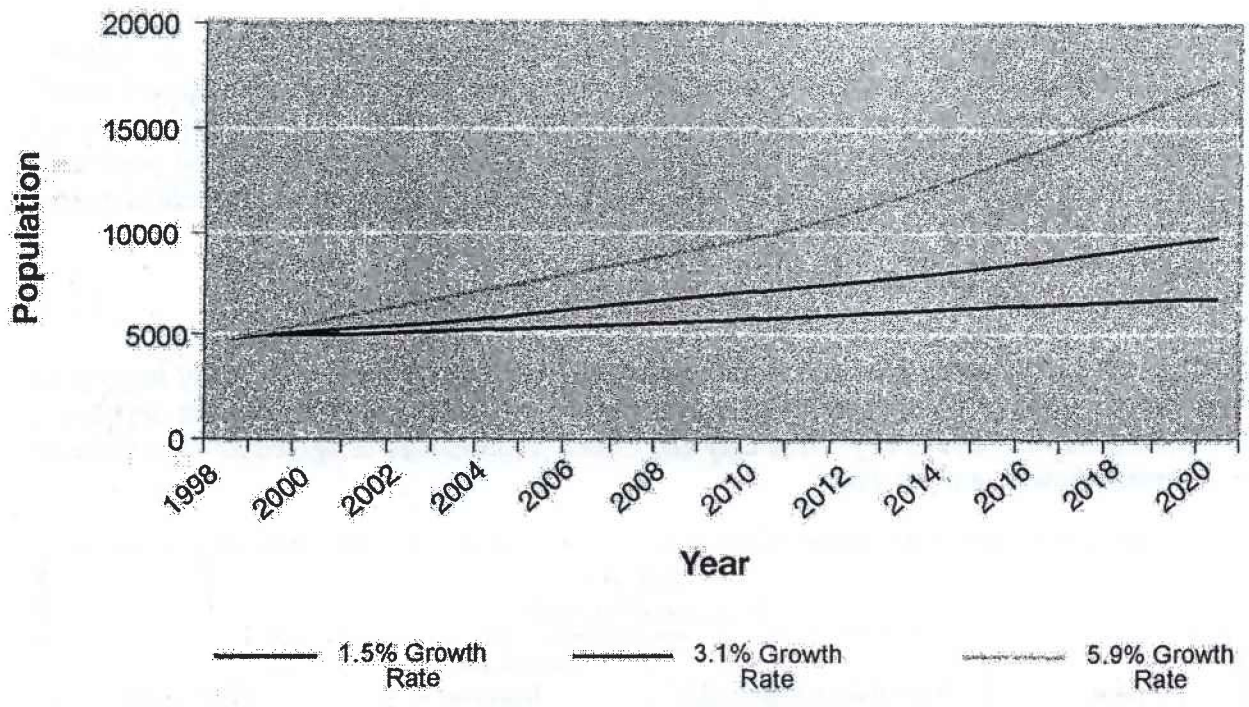

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Figure 2-4
Redwood District
Population Projections

Although this estimate includes the entire District area, not just the sewered area, it should accurately represent the District's growth based on GIS records.

City of Grants Pass Comprehensive Plan Estimate – Although the City's comprehensive plan does not evaluate growth projections specific to the District, it does contain projections for the urban growth area (UGA). According to the City's 1992 Comprehensive Plan, the city's population growth for 1980 to 1990 was 1.5 percent each year. The plan's population growth projection to year 2010 was 1.5 percent per year for the UGA and 1.4 percent for the existing city limits. Approximately one-half of the Redwood District is included in the UGA.

State of Oregon Estimate – The State set the County's population growth rate at 1.0 percent per year for the years from 2000 to 2020. The County and City are obligated to use this number for planning or they need to formally appeal to the State to change the estimate. According to Oregon's Department of Land Conservation and Development (Griffin 1998 Personal Communication), 1.0 percent per year growth is to be used as a countywide average. If plans for one area of the County are based on a growth rate greater than 1.0 percent, then growth must be reduced from somewhere else in the County. ODEQ has stated that before the District can proceed with the Wastewater Facility Plan, the City and County need to agree on a District growth rate.

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The historical district growth since 1987 is tabulated in Table 2-1, and these data indicate that recent growth in the district has been significantly greater than the other estimates quoted above. Therefore the high-growth scenario, or the recent rate of sewer connections, could probably not be sustained over a long planning period. In addition, since a good portion of the population served is made up of retired persons, the rate of student population growth in the school district may not be indicative of increases in sewered population.

Conclusion

Because the City Comprehensive Plan and the District estimates are not specifically targeted to population growth in the District, the County estimate would probably be the best population predictor available. In January 1999, City and County representatives agreed to use a value of 3.1 percent growth rate per year.

Table 2-1
Historical Growth

Year	Annual Increased ERUs	Annual Percent Increase	Total ERUs
1987	---	---	1,083
1988	53	4.9%	1,136
1989	57	5.0%	1,193
1990	75	6.3%	1,268
1991	62	4.9%	1,330
1992	140	10.5%	1,470
1993	126	8.6%	1,596
1994	119	7.5%	1,715
1995	92	5.4%	1,807
1996	132	7.3%	1,939
1997	63	3.2%	2,002
1998	25	1.3%	2,027
Average Growth Rate – 5.9%			

NOTE: Table based on information from Josephine County

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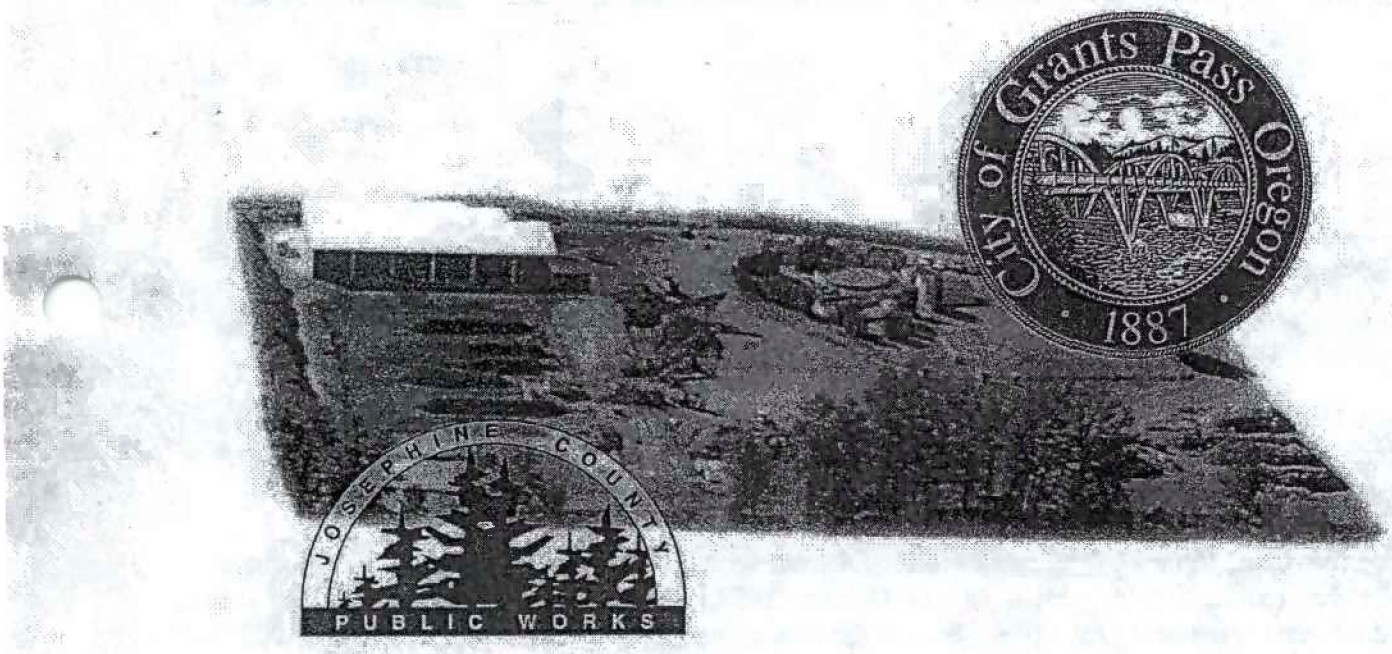
Existing Collection System

Redwood Sanitary Sewer Service District

Josephine County, Oregon



Parametrix, Inc.

3. EXISTING WASTEWATER COLLECTION SYSTEM

3.1 DESCRIPTION OF FACILITIES

In 1977-78, the District constructed a wastewater collection system and treatment facility. The collection system when completed, consisted of approximately 16.2 miles of concrete sewers ranging from 8 to 27 inches in diameter. Specifically, the sewers built consisted of the following components (Table 3-1):

Table 3-1
As-built Components of the District's Existing Wastewater Collection System[1]

Component	Interceptor System[2]	Collection System[2]
Manholes	143	132
8-inch sewer	2,606/ft.	37,825/ft.
10-inch sewer	10,041/ft.	—
12-inch sewer	12,315/ft.	—
15-inch sewer	5,076/ft.	—
18-inch sewer	5,673/ft.	—
24-inch sewer	11,646/ft.	—

(1) As shown in the final pay estimate for sewage collection and treatment facilities, Tab 302 and Tab 310, September 1978.
(2) As identified by either Schedule I – Interceptor System or Schedule C – Collection System in Contract Documents CH2M Hill, May 1974.

Since 1978, the only changes and modifications that have been made to the system added approximately 16,800 feet of mostly 8-inch-diameter PVC sewers. These sewers were added to serve new residential housing developments.

Also since then, some existing homes in the District, which were not originally connected to the sewer system, have been added. As of October 1998, with these new connections and the original system connections, the system now serves an estimated service population of 4,905.

The existing wastewater collection system serves residential units almost exclusively. Essentially, no industry discharges to the system. A collection system layout, showing pipeline routes and manholes, is provided in Figure 3-1.

3.2 MOST RECENT FLOW MONITORING

Since 1978, only flow monitoring in the wastewater collection system has been performed at the wastewater treatment plant. These data were collected by the plant effluent flow meter and recorder (Appendix B). These flow data are presented later in this report.

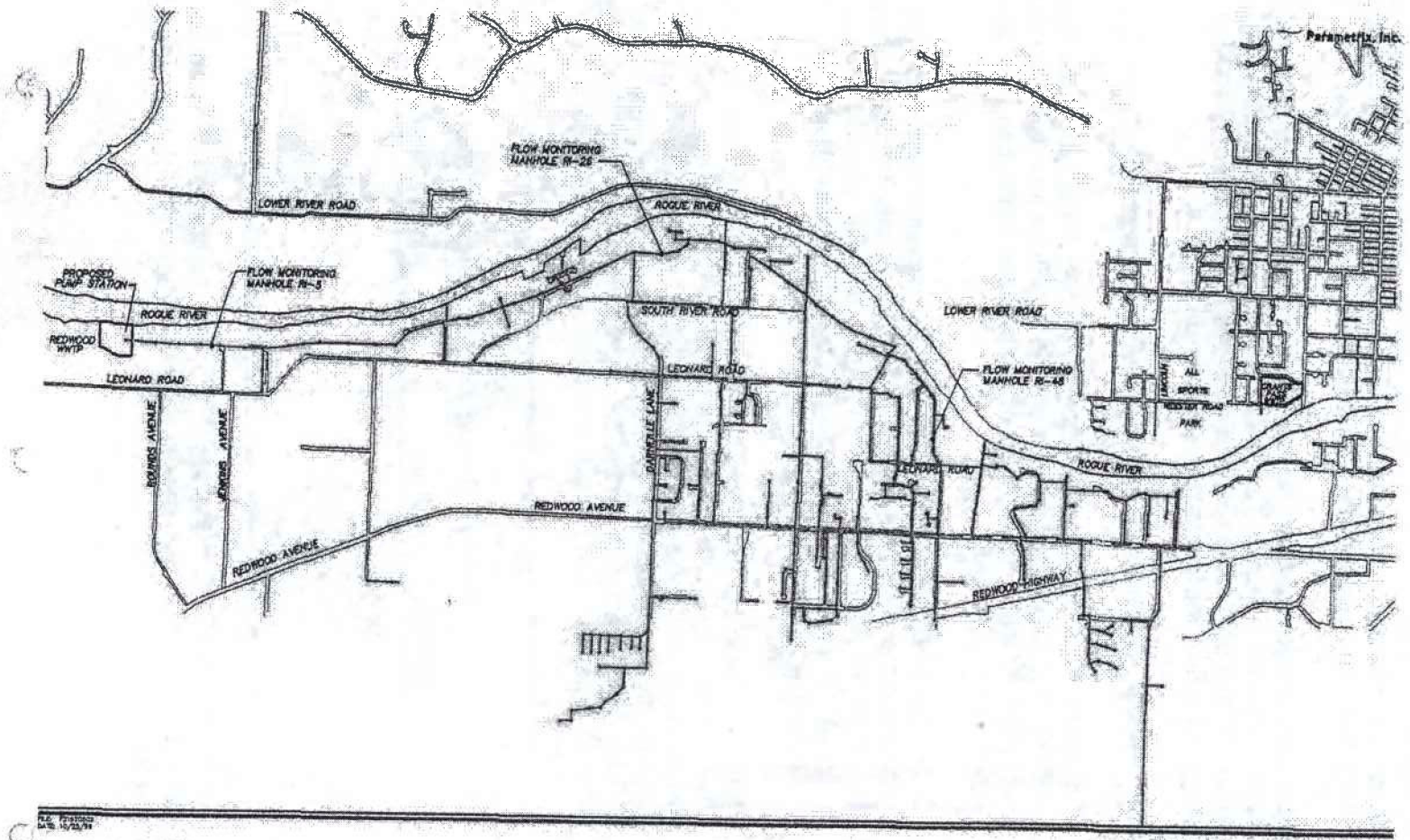


Figure 3-1
Existing Redwood Sanitary Sewer Service District Wastewater Collection System

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To better assess the conditions for the wastewater collection system, more detailed flow monitoring was conducted. In September, October, and December 1992 and January 1993, continuous wastewater flow was measured in three different manholes of the collection system. ADS Environmental Services, Inc. measured these flows.

The flow measurements assessed the wastewater collection system inflow and infiltration (I&I) conditions. On each occasion, flow was measured during different groundwater and rainfall parameters. Differences in the observed wastewater flows could then be used to identify general infiltration and inflow sources to the collection system.

Generally, flow measurements at each occasion were made to assess the following conditions.

In September 1992, flow was monitored because it was anticipated that rainfall would be unlikely and that the groundwater level in the service area would be high due to irrigation and canal leakage. The impact of groundwater infiltration into the collection system could then be assessed.

In October 1992, wastewater flows were also measured because rainfall was unlikely and groundwater levels were expected to be low. At this time, irrigation had been stopped for about 30 days; the canals were dry, and the groundwater levels had probably dropped. Wastewater flows at this time would most likely indicate the base sewage flow in the system unaffected by infiltration and inflow.

Although it was believed that October 1992 would have little or no rainfall during the flow monitoring period, this did not occur. In fact, a severe rainstorm affected the later stages of the flow measurement. Specifically, rain began on October 28 and continued through the remaining flow monitoring days. Comparing these data to the historical rainfall data presented in Section 2.4 indicates that this storm was less than a 2-year, 24-hour storm event. The rainfall during this period is shown in Table 3-2.

Table 3-2
Rainfall During Flow Monitoring Period

1992 Monitoring Dates	Rainfall (Inches)
October 24-27	0.00
October 28	0.02
October 29	0.06
October 30	1.48
October 31	0.66

From December 1992 to January 1993, flow was monitored because it was anticipated that rainfall would be high and the impact of inflow into the collection system could then be assessed.

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During this flow monitoring, significant rainfall events did, in fact, occur. The seven-day period with the most precipitation was selected for inflow analysis. The rainfall during this period is shown in Table 3-3.

Table 3-3
Rainfall During Flow Monitoring Period

1992 - 1993	Rainfall (inches)
December 27	0.75
December 28	1.37
December 29	0.77
December 30	0.35
December 31	1.73
January 1	0.28
January 2	0.04

The three manholes chosen to measure flow in the collection system were selected so as to divide the system roughly into thirds. This approach could then identify the condition(s) in particular portions of the system, if possible.

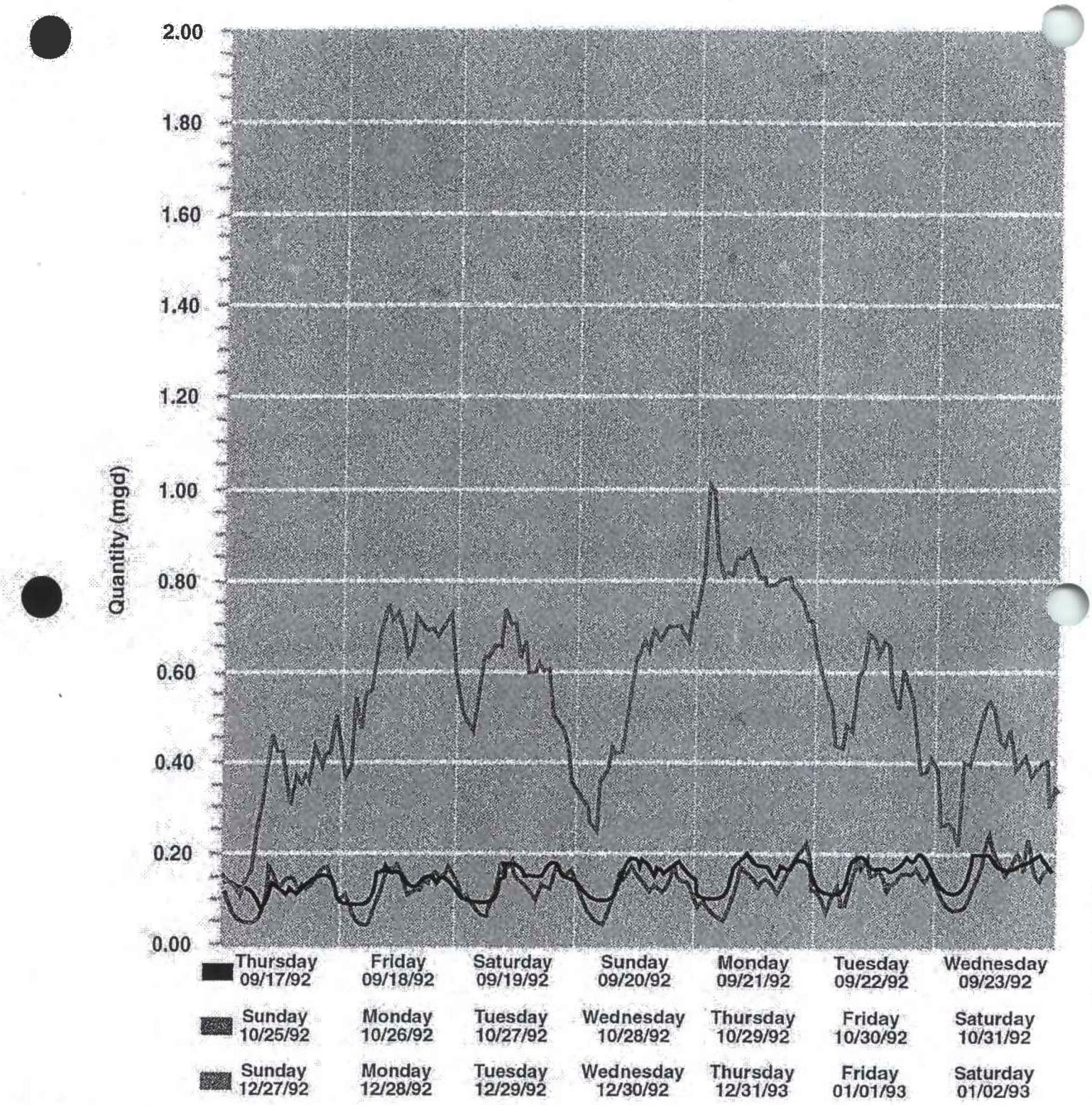
Flow in each manhole represents roughly the following:

Manhole RI-48 - Flow measured at point furthest from the treatment plant to represent roughly one-third of the collection system.

Manhole RI-26 - Flow measured at a point representing roughly two-thirds of the system.

Manhole RI-5 - Flow, measured at a point in the collection system closest to the treatment plant, represents roughly the entire system.

The collection system flow monitoring results are shown in Figures 3-2, 3-3, and 3-4. Flows in each manhole for each occasion are shown separately. The complete data files appear in the ADS Environmental Services, Inc. report, *Redwood Sanitary Sewer Service District Assessment of I/I for Facility Planning*, 1993.



Redwood Wastewater Treatment Plant
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Figure 3-2
Flow Monitoring Results
Manhole RI-48

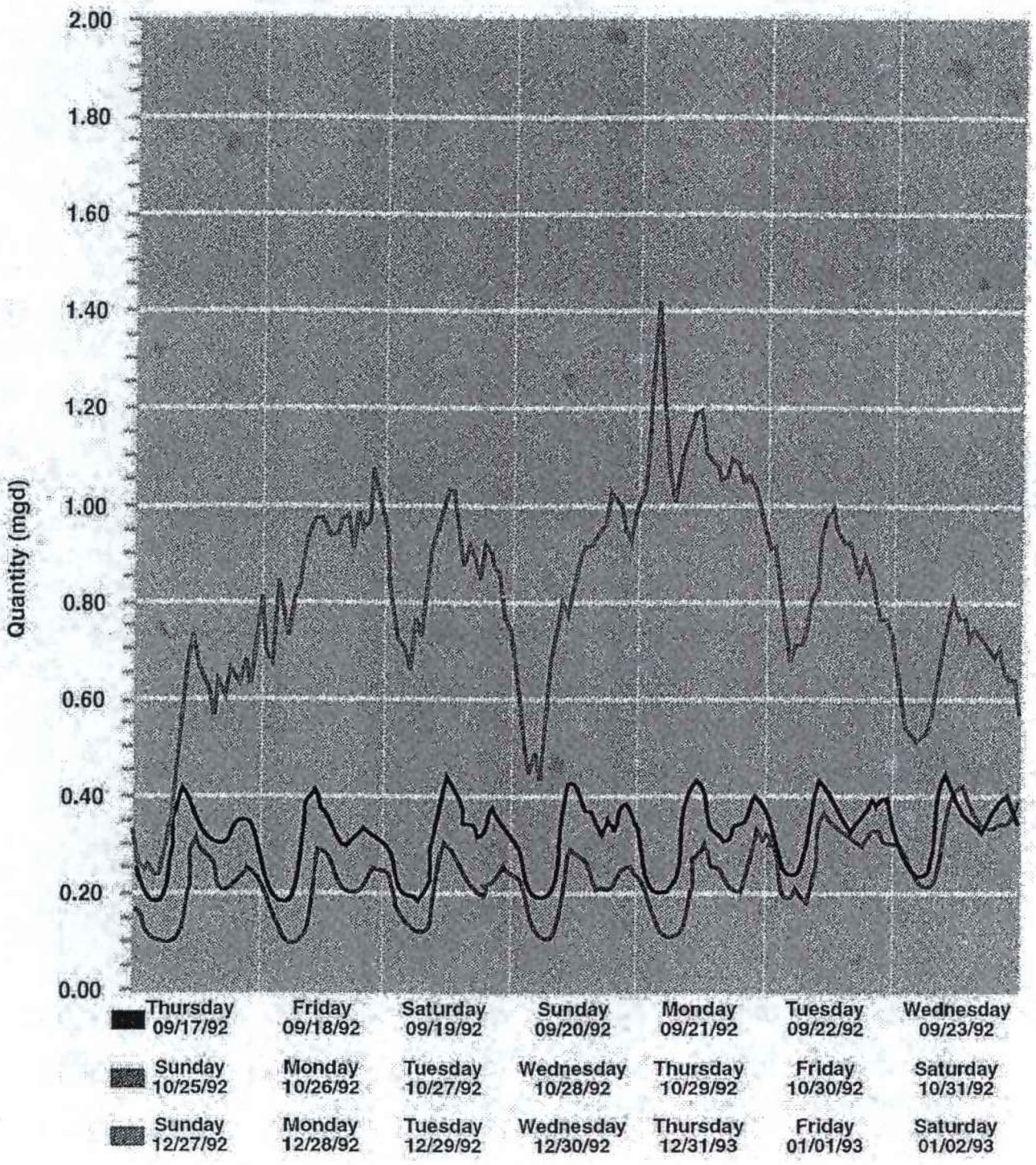
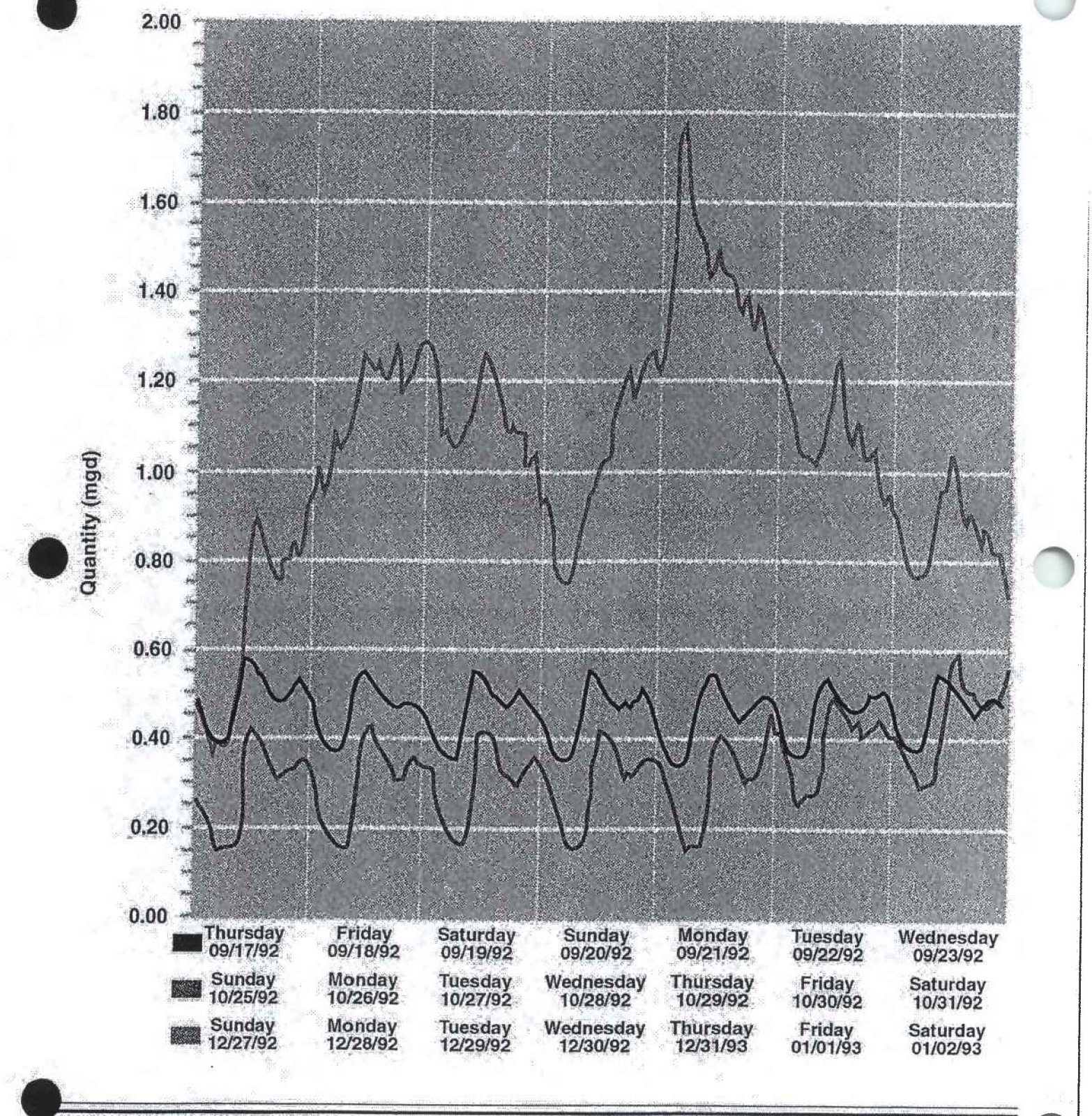


Figure 3-3
Flow Monitoring Results
Manhole RI-26



Redwood Wastewater Treatment Plant
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Figure 3-4
Flow Monitoring Results
Manhole RI-5

3.3 INFILTRATION AND INFLOW ASSESSMENT

3.3.1 Infiltration

The difference in flow measured at each manhole during September and October is significant, particularly at Manholes RI-26 and RI-5. It likely indicates that groundwater infiltrates into the collection system during the irrigation season and during extended rainfall events. This is apparent, based on the following observations:

- The lowest flows measured between 3:00 and 5:00 a.m. at Manhole RI-48 changed from ±0.13 mgd in September to ±0.06 mgd in October. The October flows probably represent a base flow without infiltration. The September flows probably represent flow with infiltration. Since the September flows are double the October flows, the difference is probably only infiltration from high groundwater conditions owing to irrigation.

- Flows measured at Manhole RI-26 were consistently almost 0.10 mgd higher in September than in the first four days of October flow measurement. Flow conditions at this point in the system were significantly affected by the high groundwater conditions in September, but not by rainfall, since no rainfall occurred until October 28. The 0.10 mgd difference in flow is therefore probably due to infiltration into the collection system.

- Flows measured at Manhole RI-5 were consistently almost 0.20 mgd higher in September than in the first four days of October flow measurement. As with Manhole RI-26, flow conditions at Manhole RI-5 were also significantly affected by the high groundwater conditions in September. The 0.20 mgd difference in flow is probably attributable to infiltration.

- During the latter days of October, flow measurements increased as a result of rainfall (October 28 to October 31). Flow increased appreciably on October 30, the highest rainfall event. Groundwater levels were presumably rising during each day of rainfall. This resulted in October flow measurements approaching September measurements during October 30 and 31. These flows resulted from infiltration of groundwater into the collection system.

Collection system infiltration is significant. Flow measurements to date indicate that infiltration into the collection system may amount to as much as 0.20 mgd during the highest groundwater levels. Highest groundwater levels occur either during the irrigation season (summer) because of irrigation and canal leakage, or during the winter rainy season. Low groundwater levels may occur only during the spring, before irrigation, or the fall, after irrigation ends but before significant rainfall occurs.

3.3.2 Inflow

The difference in flow measured at each manhole during October and December/January is also significant. Substantial differences occurred at all three manholes, likely indicating inflow into the collection system during prolonged heavy rainfall.

Figure 3-5 presents October, December, and January flow measurements at Manhole RI-5 and the daily rainfall records. Generally, the greater the daily rainfall, the greater the daily flow measurements. Inflow into the collection system is also significant.

Rainfall increased wastewater flow at all three manholes. All three portions of the collection system appear to be equally impacted by rainfall.

3.4 COLLECTION SYSTEM CONDITIONS

3.4.1 Non-Excessive I&I Definition

The U.S. Environmental Protection Agency (EPA), in a publication entitled, *I&I Analysis and Project Certification* dated May 1985 (EPA 1985), sets numerical limits for both non-excessive infiltration and inflow based on results of national surveys of sewerage agencies.

The EPA criteria for non-excessive I&I are as follows:

- If the average daily flow per capita (excluding major industrial and commercial flows greater than 50,000 gpd each) measured during 7- to 14-day dry periods during seasonal high groundwater is less than 120 gallons per capita day (gpcd), the collection system infiltration shall be considered non-excessive.

  The EPA criteria for determining non-excessive infiltration includes the following flow components:

  a. 70 gpcd is considered the domestic wastewater base flow.

  b. 50 gpcd is considered the non-excessive infiltration flow.

  The total flow standard of 120 gpcd is based upon these two components.

- If the average daily flow per capita (excluding major industrial and commercial flows greater than 50,000 gpd each) measured during periods of significant rainfall during which the treatment plant does not experience hydraulic overloads is less than 275 gpcd, the collection system inflow shall be considered non-excessive.

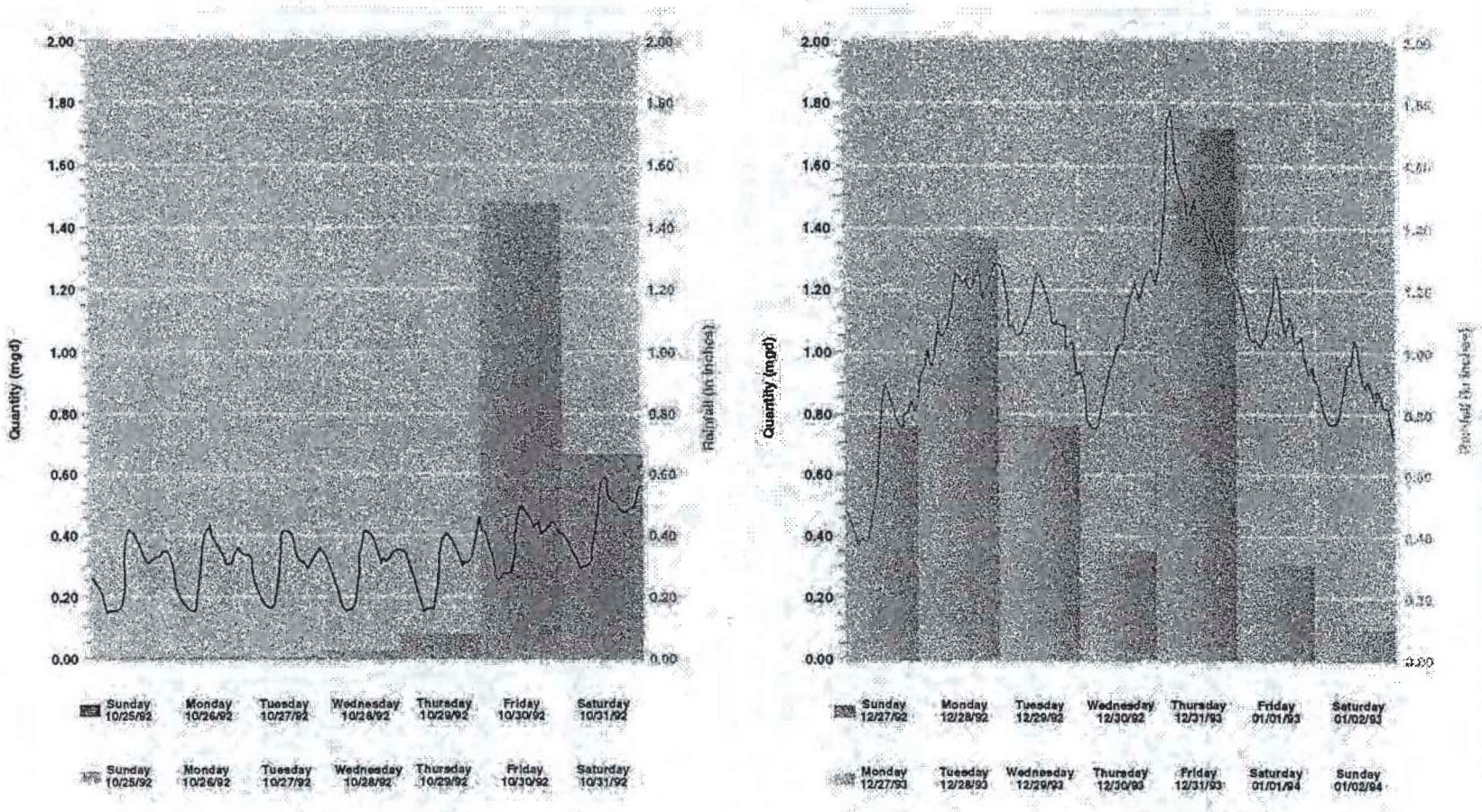

Figure 3-5
Flow Monitoring Results and Rainfall Data Manhole RI-5

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The EPA criteria for determining non-excessive inflow includes the following two flow components:

a. 70 gpcd is considered the base domestic wastewater flow.

b. 205 gpcd is considered the level of non-excessive inflow flow.

The total flow standard of 275 gpcd is based upon these two components.

If excessive infiltration and/or inflow is determined, further studies must quantify the excesses and evaluate corrective measures. Using the results of these studies, the most cost-effective solution is then selected. Possible solutions could include a sewer rehabilitation program, sizing of the treatment plant to handle inflow and infiltration, or some combination of these two approaches.

3.4.2 Comparison to Measured Flows

Infiltration

During September 1992, the measured seven-day average daily flow at Manhole RI-5 was 0.464 mgd. If the population served by the plant at that time was 3,775, this flow equates to 122 gpcd and is greater than the EPA 120 gpcd criteria.

It is greater than the criteria, however, only by 2 gpcd. At less than two percent of the standard, it is doubtful whether EPA would conclude the Redwood sanitary sewage collection system has excessive infiltration.

Inflow

The highest measured instantaneous flow occurred during December/January at Manhole RI-5 (1.86 mgd) where average daily flow was 1.07 mgd. This flow occurred on December 31, 1992.

The highest measured flow equates to 493 gpcd and the average daily flow equates to 283 gpcd using a service area population of 3,775. Both of these values are more than the EPA criteria for excessive inflow. The District's sanitary sewer does appear to have excessive inflow based on these data, particularly if rainfall on this date (of the highest measured flow) is considered.

Even though highest instantaneous flows occurred on December 31, 1992, only 1.73 inches of rainfall was measured. This is not a significant event. The 2-year, 24-hour rainfall event is 3.0 inches (Section 2.4), almost twice the rainfall on December 31, 1992. Had the rainfall been greater, undoubtedly the average daily flow at the plant would have been higher.

Historical plant flow records report flow exceeding 275 gpcd twice during the last five years of data; however, these data were recorded by an old flow meter.

3.5 INFILTRATION AND INFLOW RECOMMENDATION

The collection system exceeded EPA criteria for both I&I during the flow monitoring periods. The infiltration criteria was exceeded by only a modest amount, 2 gpcd over the criteria. However, the inflow criteria were exceeded during an event less than a 2-year, 24-hour storm event. This requires action(s) to correct inflow.

We recommend that the District establish an I&I correction program because (1) evidence indicates that inflow is excessive, whereas infiltration is only marginally excessive; and (2) we understand that the State of Oregon has an understanding with small municipalities and districts that if inflow problems are addressed and corrected in wastewater collection systems, the infiltration problems, unless severe, do not require correction. Correcting infiltration is generally not cost-effective.

3.5.1 Cost

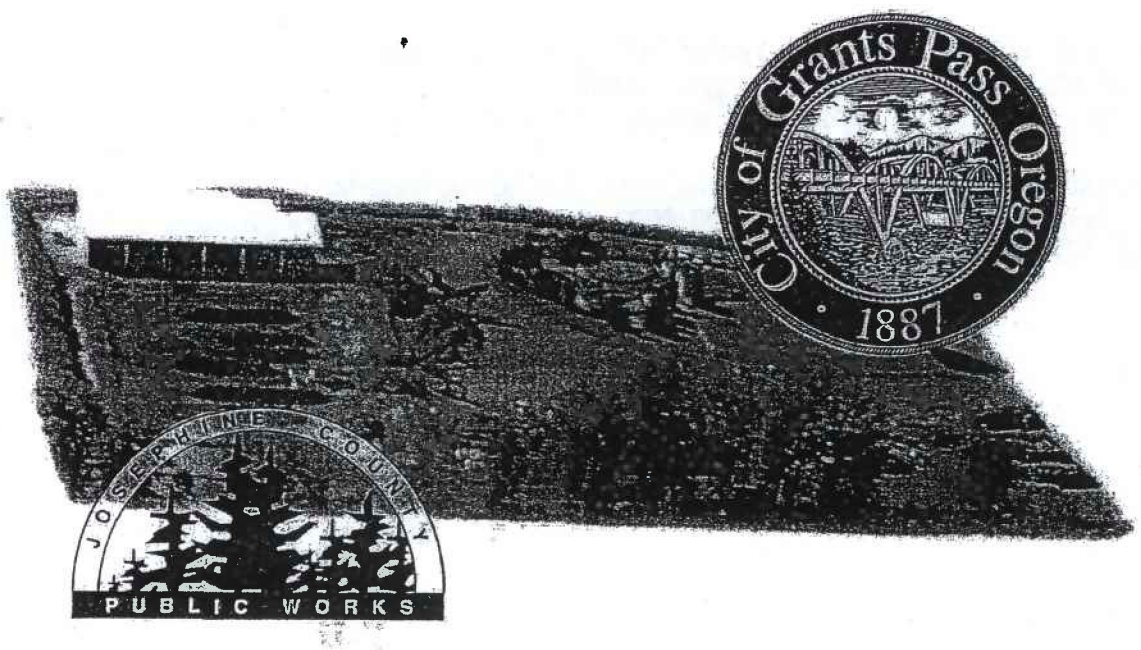
To correct excessive collection system I&I, it is recommended that the District institute an annual I&I reduction program budget. Annual expenditures should be integrated into the District's financing for capital improvements and operations and maintenance.

It is recommended that an annual expenditure of $25,000 be budgeted for district I&I correction. This amount should be budgeted until 2003, at which time another evaluation of I&I in the system should be completed.

Existing Treatment System

Redwood Sanitary Sewer Service District

Josephine County, Oregon



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4. EXISTING WASTEWATER TREATMENT SYSTEM

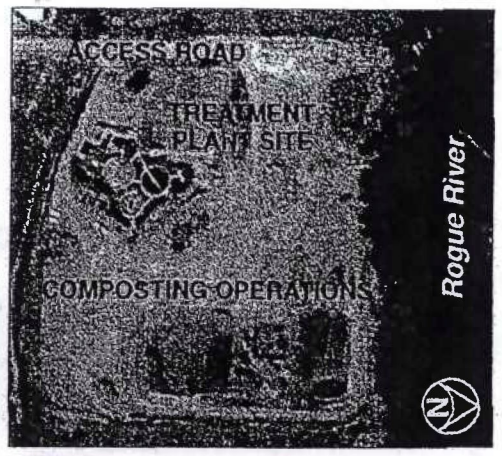
4.1 EXISTING WASTEWATER TREATMENT PLANT

4.1.1 Description of Facilities

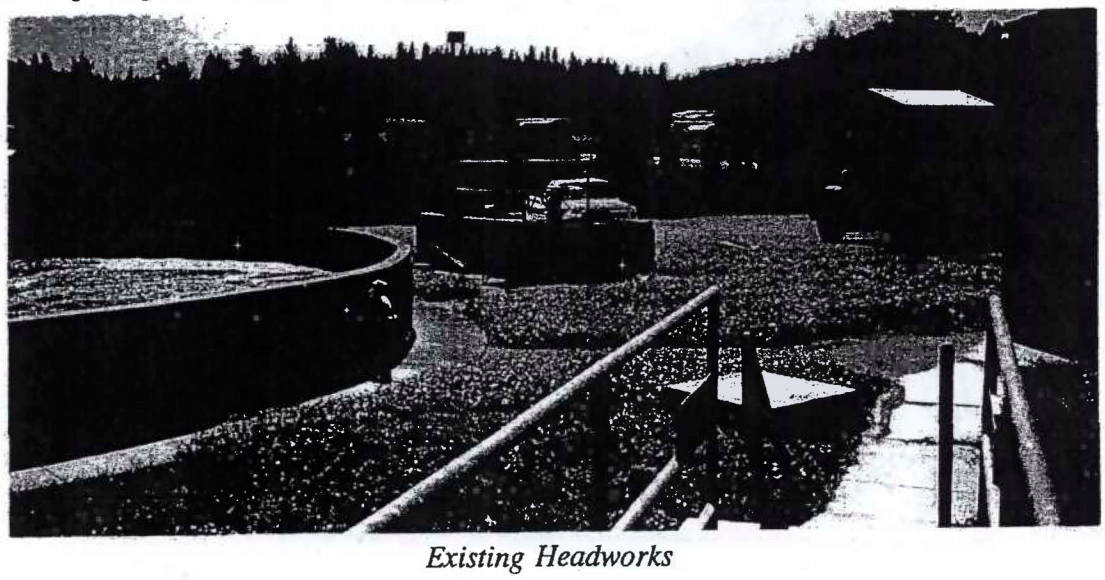
The existing Redwood WWTP is a conventional activated sludge plant with no primary sedimentation (Figure 4-1). The plant is located at the west end of the District's service area (see Figure 3-1).

Located at the plant site are the unit treatment processes, a control/laboratory building, the blower/sludge pumping building, and biosolids composting operations. The unit treatment processes include the following components:

- Influent pump station
- Headworks with comminutor
- Aeration basin
- Secondary clarifier
- Chlorine contact chamber
- Effluent outlet structure and outfall
- Aerobic digester
- Biosolids compost facility



The wastewater enters the treatment plant at the influent pump station, and is pumped to the headworks where the solids and debris are shredded by a comminutor or screened through a ¾-inch opening bar screen. This completes primary treatment at the plant.



Existing Headworks

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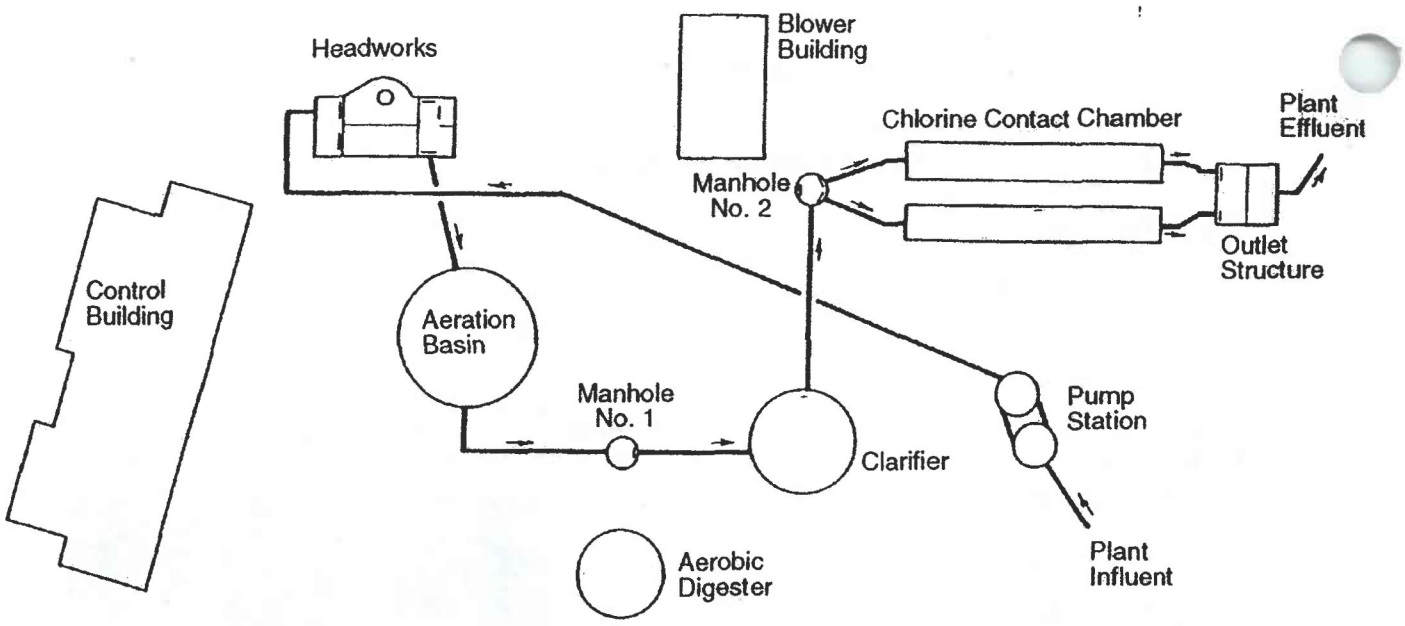

PROCESS SCHEMATIC

SITE PLAN

Redwood Wastewater Treatment Plant
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Figure 4-1
Process Schematic and Site Plan

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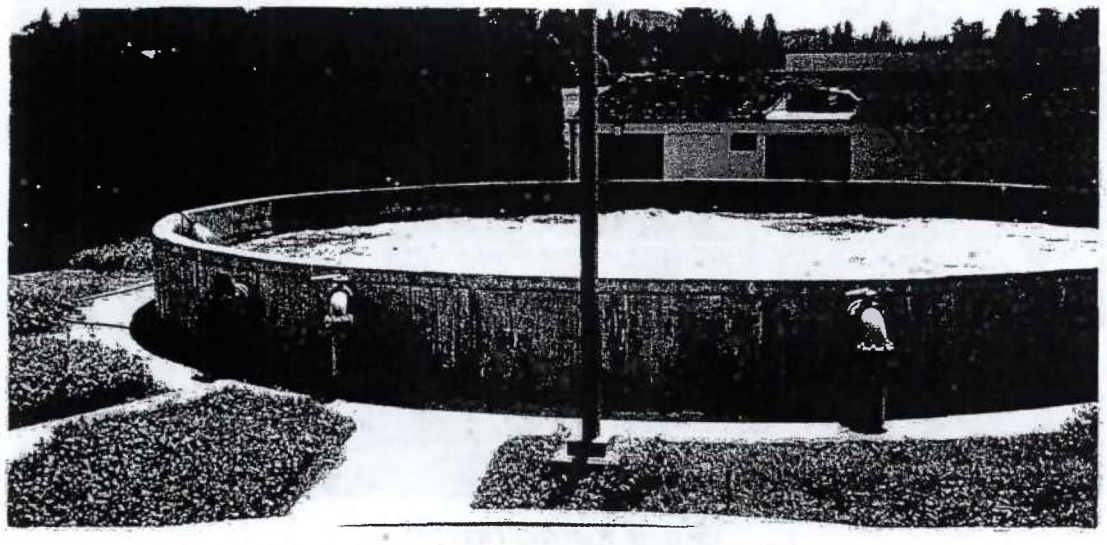
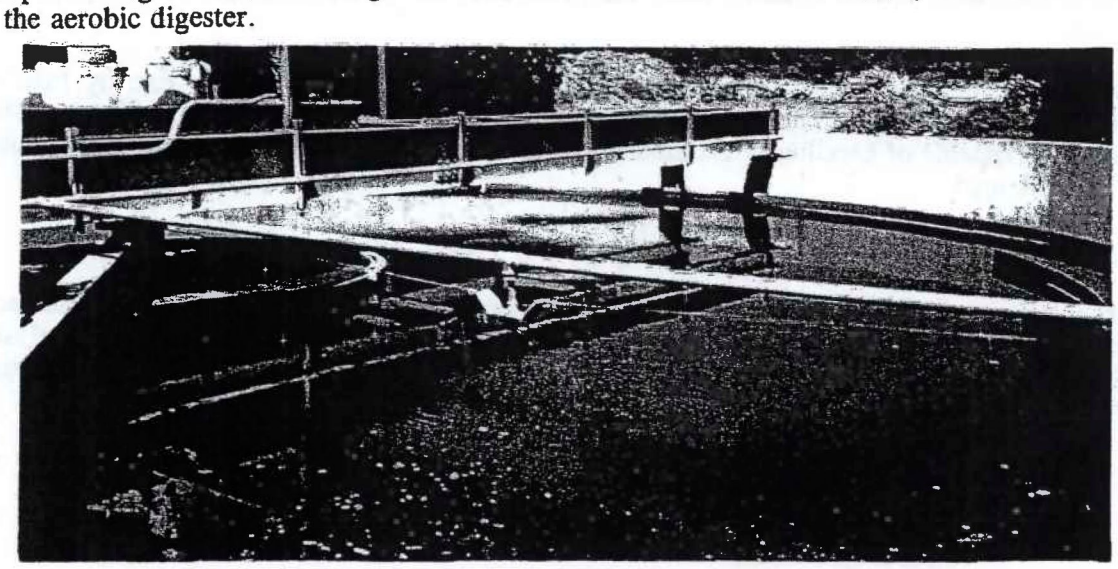
From the headworks, wastewater is directed to the aeration basin, where it undergoes biological treatment which results in activated sludge. Wastewater in the basin is aerated using positive displacement blowers situated in the blower building. Coarse bubble diffusers are located on three (3) air diffuser pipelines submerged in the tank. The wastewater in the aeration basin is referred to as "mixed liquor."



Existing Aeration Basin and Control Building

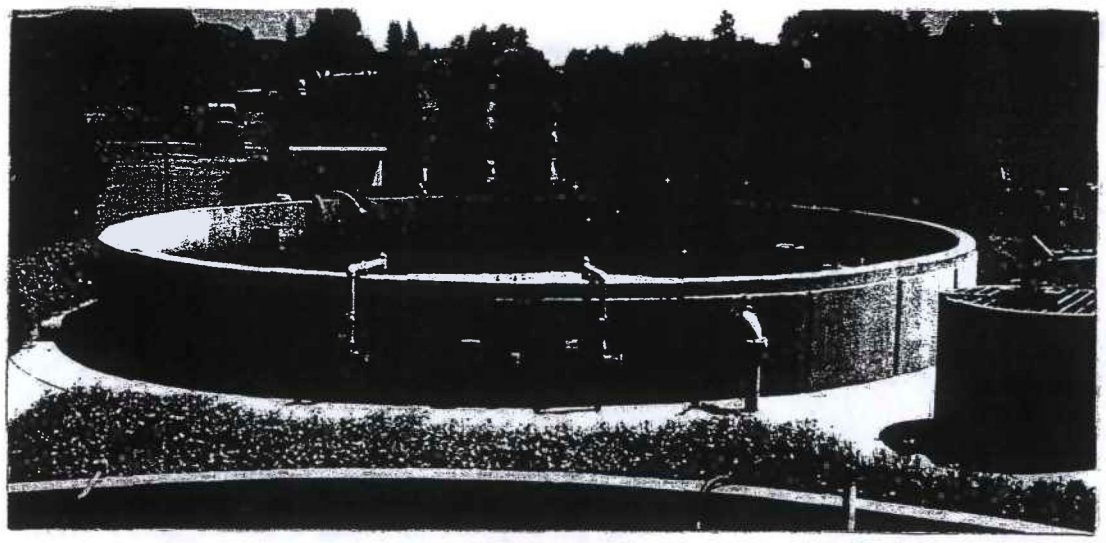
The mixed liquor from the aeration basin is then transferred to the secondary clarifier where the biological floc and other solids are separated from the treated wastewater.

A portion of the solids (referred to as return activated sludge) collected at the bottom of the clarifier are recycled back to the headworks to maintain a sufficient level of microorganisms of an optimum age to reduce the organic load in the wastewater. The remaining solids are directed to the aerobic digester.



Secondary Clarifier

The solids are treated further in the aerobic digester before being sent to the compost facility for final disposal. Supernatant, the clarified liquid that forms on top of the solids in the digester, is sent back to the influent pump station for treatment.



Existing Aerobic Digester

Treated wastewater is disinfected with chlorine before discharge to the Rogue River. A contact chamber provides the time needed for the chlorine to reduce the potential pathogenic bacteria to an acceptable number. The effluent is measured prior to discharge. A weir is used to control the level of release and the flow rate.

The plant treats wastewater which is generated from residential homes. There are no known industrial discharges to the system; also, the plant does not accept septage from septic tank pumpers.

A detailed plant capacity analysis included in the 1994 District's Wastewater Facility Plan is included in Appendix D. The capacity of each portion of the process was analyzed. The facilities are capable of handling a maximum month average flow of 0.6 mgd and a peak hour flow of 1.44 mgd.

4.2 WASTEWATER FLOW AND LOAD

Before the Redwood WWTP capacity requirements can be determined and wastewater treatment alternatives evaluated, existing flow and load needed to be reviewed. The flow/load analysis was based on wastewater data collected at the Redwood WWTP from September 1995 to August 1998.

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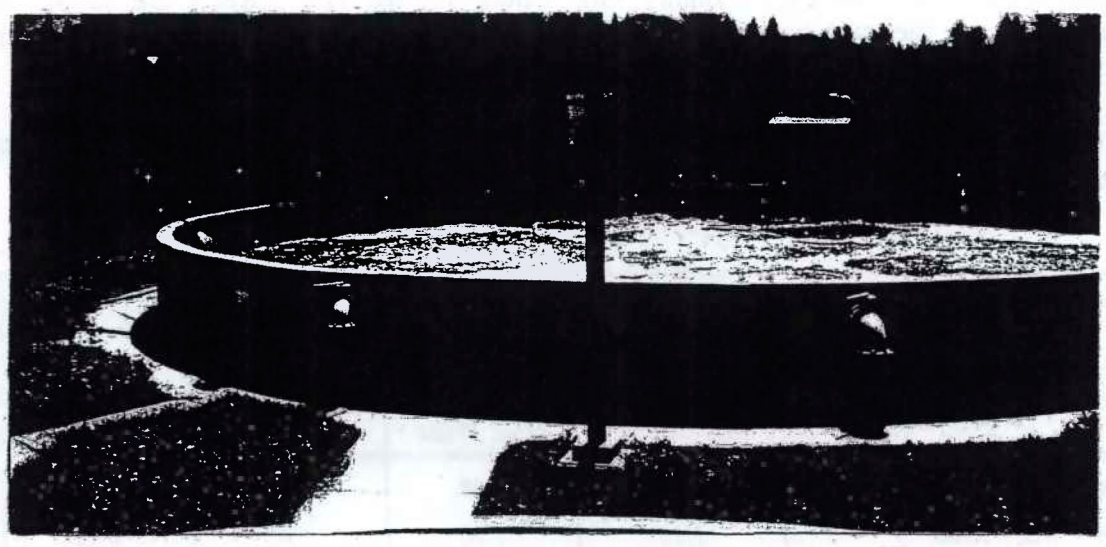
The solids are treated further in the aerobic digester before being sent to the compost facility for final disposal. Supernatant, the clarified liquid that forms on top of the solids in the digester, is sent back to the influent pump station for treatment.



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4.2.1 Existing Wastewater Flow

Effluent wastewater flow measurements have been taken at the Redwood WWTP since 1978. These measurements are based on the water level measured upstream of a 60° V-notch weir at the effluent structure. Average monthly flow values for the last 3 years are shown in Figure 4-2. A very thorough flow evaluation, made in 1994 as part of the District's Wastewater Facility Plan, is included in Appendix E.

Based on average flow and peak day flow, a summary table of flow conditions was prepared (Table 4-1).

Table 4-1
1998 Plant Flow Conditions
Million Gallons Per Day (mgd)

	Max. Month	Max. Week	Peak Day	Average
Summer – Dry Weather	0.54	0.58	0.76	0.49
Winter – Wet Weather	1.16	1.20	1.48	0.63
Annual	---	---	---	0.56

4.2.2 Existing Wastewater Load

Influent and effluent wastewater Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS) are measured twice per week to quantify the load of pollutants in the wastewater. BOD and TSS are normally measured in mg/L of water. Using the BOD concentration and wastewater flow measurements, plant influent pounds of BOD per day were calculated. Three years of 30-day average BOD lbs/day calculation results are shown on Figure 4-3. Using monthly average and peak day BOD lbs/day, a summary table of BOD load conditions was prepared (Table 4-2) from data dated January through December of 1995 to 1998.

Table 4-2
1998 Plant BOD Load Conditions (lbs/day)

	Max. Month	Max. Week	Peak Day	Average
Summer – Dry Weather	1,122	1,345	2,488	716
Winter – Wet Weather	1,589	1,731	4,428	969
Annual	---	---	---	843

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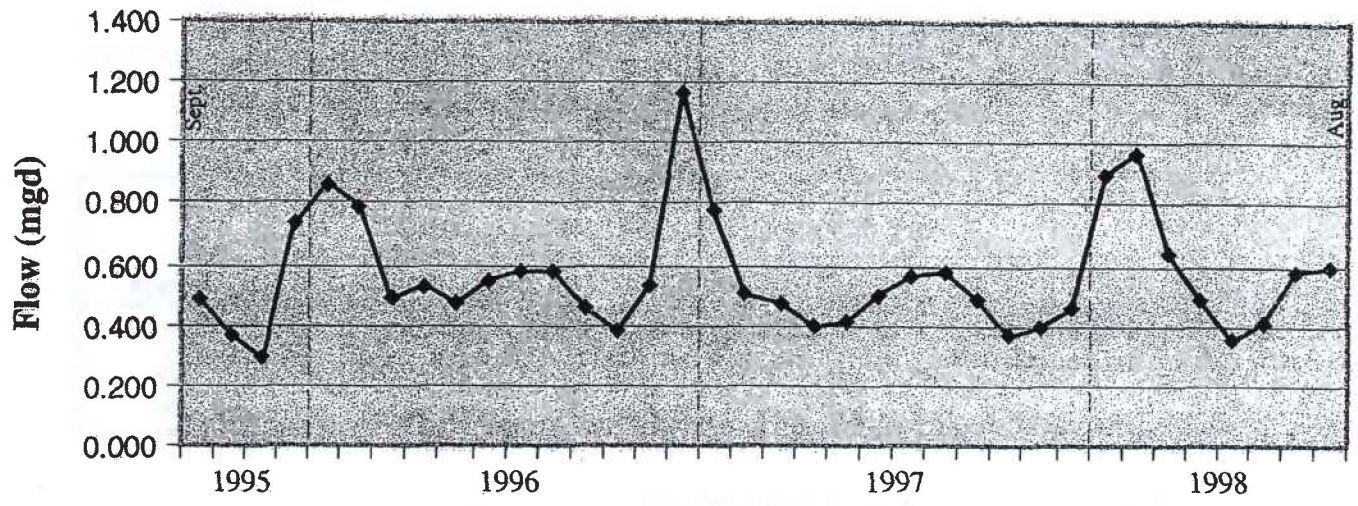

Redwood Wastewater Treatment Plant
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Figure 4-2
Redwood WWTP Average Daily Flow/Month

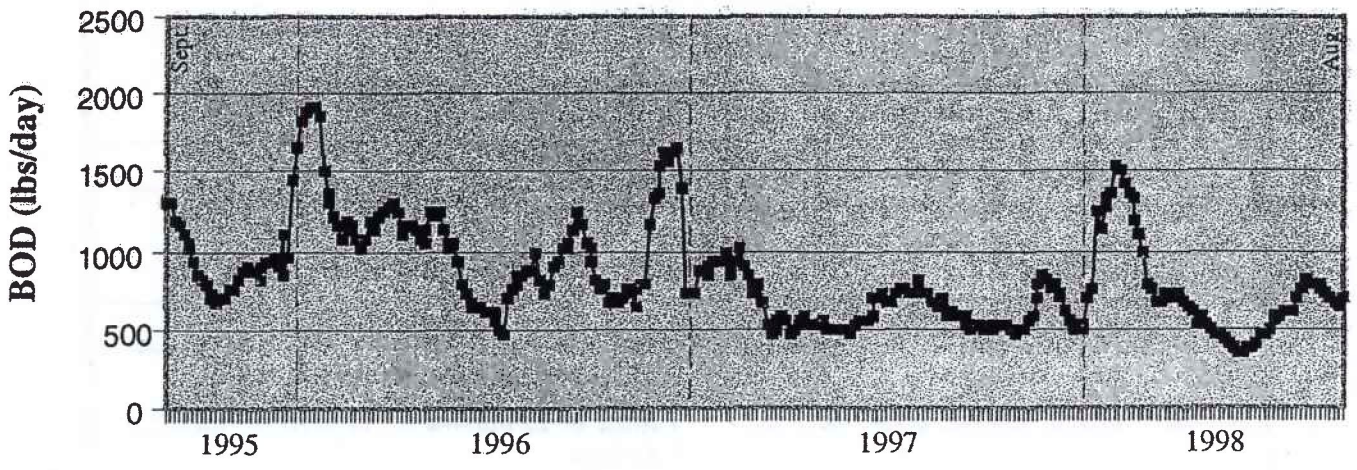

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Figure 4-3
Redwood WWTP 30-Day Average BOD

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Using the TSS concentration and wastewater flow measurements, plant influent pounds of TSS per day were calculated. Table 4-3 is a summary of TSS load conditions. This table used data from January through December of 1995 to 1998 to calculate the maximum month and week, peak day and average.

Table 4-3
1998 Plant TSS Load Conditions (lbs/day)

	Max. Month	Max. Week	Peak Day	Average
Summer - Dry Weather	1,398	1,576	2,455	930
Winter - Wet Weather	1,116	1,404	3,473	829
Annual	---	---	---	879

4.3 PLANT REGULATORY LIMITATIONS

The regulatory limitations for the Redwood WWTP are documented in the plant's NPDES permit. The effluent limitations in this permit vary by time of year (Table 4-4). In addition, the plant flow is limited to an average monthly value of 0.48 mgd.

Table 4-4
Existing Redwood WWTP NPDES Limits

Parameter	Average Effluent Concentrations		Monthly Average lb/day	Weekly Average lb/day	Daily Maximum lbs
	Monthly	Weekly			
May 1 – October 31					
BOD	20 mg/L	30 mg/L	80	120	160
TSS	20 mg/L	30 mg/L	80	120	160
FC* per 100 ml	200	400			
November 1 – April 30					
BOD	30 mg/L	45 mg/L	120	180	240
TSS	30 mg/L	45 mg/L	120	180	240
FC* per 100 ml	200	400			
Other Parameters (year-round)			**Limitations**		
pH			Must be within the range 6.0 – 9.0		
Average percent removal (BOD & TSS)			Must be minimum of 85 percent, monthly average		
Average dry weather flow to the treatment facility			0.48 mgd		

* *Fecal Coliforms*

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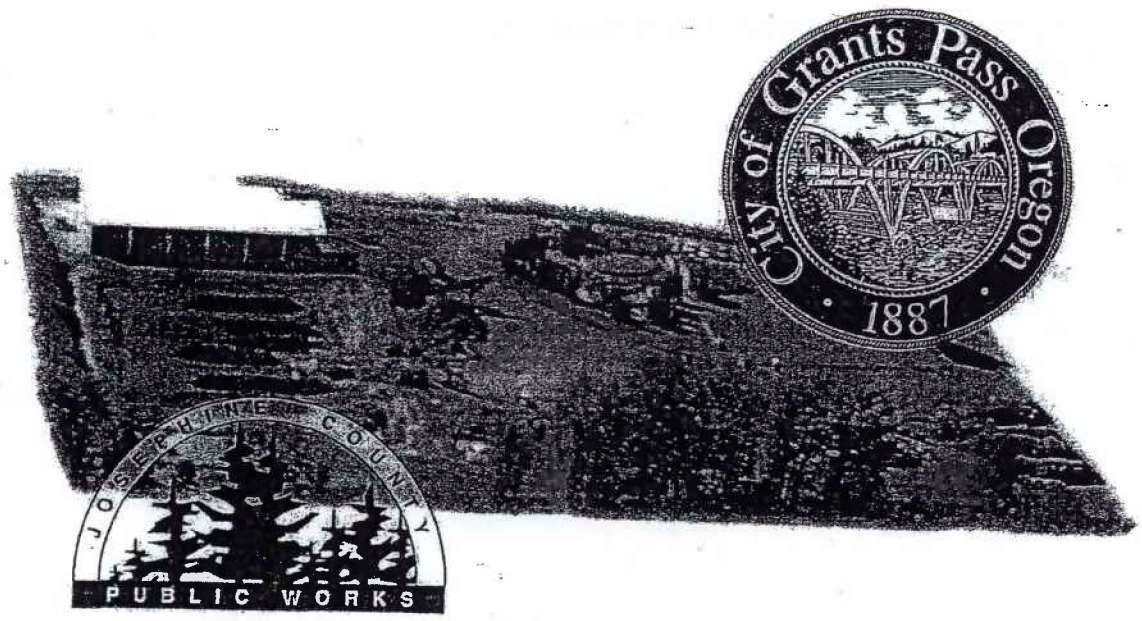
Future Treatment Requirements

Redwood Sanitary Sewer Service District

Josephine County, Oregon



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INTRODUCTION TO CHAPTER 5

FUTURE TREATMENT REQUIREMENTS

Since the preparation of the *1994 Redwood Facilities Plan Report*, several water quality standards have been revised. In 1996, new temperature, bacteria, pH, and dissolved oxygen standards were affected.

For the Facilities Plan Update, these parameters would have been reevaluated in Section 5.3 had a treatment plant upgrade been the preferred alternative. Since the outcome of this document is to convey the wastewater to Grants Pass WRP and abandon the Redwood WWTP, it is immaterial to review the new water quality standards. Therefore, Chapter 5 has been extracted from the *1994 Facilities Plan Report* and has not been updated to include these revisions.

However, changes have been made to wastewater flow and load projections to update them from year 2012 to year 2020.

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5. FUTURE TREATMENT REQUIREMENTS

5.1 WASTEWATER FLOW AND LOAD PROJECTIONS

Regardless of whether the District's future wastewater treatment occurs at the Redwood WWTP or the Grants Pass WRP, the District's projected waste stream flow and load had to be determined. To determine wastewater treatment capacity needs for the District, the existing Redwood WWTP flow, load, and population were analyzed. Based on this analysis, wastewater flow/load design criteria for year 2020 were developed for presentation in this section of the report. Water conservation measures are also evaluated in this section.

5.1.1 Flow Projections

Based on the population projections in Subsection 2.8, and existing flow data presented in Subsection 4.1, flows were projected for year 2020 (Table 5-1). Flow projections are based on existing wastewater flow plus a typical flow per capita assumed for all future connections. Although the existing wastewater flow per capita is relatively high, the per capita flow assumed for future connections is typical for newer construction. If new sewer pipes are properly installed and inspected, these pipes should not allow the amount of I&I flow the District's sewers currently experience.

Table 5-1
Wastewater Flow Projections, Year 2020

	1996/1998		2020	
	Actual (mgd)	1998 Actual (Gal/cap-day)	Design (mgd)	1998-2020 (Gal/cap-day)
Summer, Max. Month	0.54	120	1.01	100
Winter, Max. Month	1.16	240	1.77	130
Summer Max. Week	0.58	126	1.10	110
Winter Max. Week	1.20	254	1.90	150
Summer, Peak Day	0.76	157	1.42	140
Winter, Peak Day	1.48	364	2.77	275
Summer Average	0.49	99	0.89	85
Winter Average	0.63	129	1.22	125
Annual Average	0.56	114	1.05	105

Projected flows were developed based on the following assumptions:

- Future flows for maximum month during the summer dry weather were calculated using the current maximum month dry weather flow (MMDWF) of 0.59 mgd and adding 100 gallons/person/day for future population.

- Future flows for maximum month during the winter wet weather were calculated using the current MMWWF of 1.18 mgd and adding 130 gallons/person/day for future population.

Projected flows for summer and winter conditions are shown on Figure 5-1.

5.1.2 Impact of Water Conservation

In recent years, two factors have influenced Americans' water use habits. This has included, on one hand, drought-stricken communities restricting the use of potable water and/or escalating water rates in response to increased regulations to protect public health. The District can be impacted by both factors in the future.

To evaluate any potential impacts, it is necessary to review where potable water is used, the source of water into the sanitary system, and the Redwood WWTP design criteria. A large portion of the potable water is used for irrigation, and this water does not enter the wastewater system. Additionally, a significant part of the wastewater flow is inflow and infiltration and this is not impacted by water conservation.

The treatment plant design criteria are based on a number of conditions, both hydraulic and organic loads, which occur at different times of the year. Organic loading will not be impacted by water conservation. A reduction in wastewater flows does not significantly impact peak flows which are highly influenced by inflow and infiltration into the system.

In the western United States, water conservation programs have been implemented in California and Seattle due to drought conditions. California conservation efforts (Table 5-2) had the following results:

Table 5-2
Water Conservation Efforts vs. Sewer Flow

Location	Reduction in Water Demand (%)	Reduction in Sewer Flows (%)
Marin County	20 to 60	15 to 40
Santa Barbara	37	34
Santa Clara	19	17

An equal wastewater flow reduction cannot be expected in the District because of inflow and infiltration. In these California communities inflow and infiltration was negligible; therefore, the conservation measures had a greater impact on wastewater flows. In the District, inflow and infiltration make a much larger contribution to total wastewater flow.

A more relevant example of water conservation's effect on wastewater flows can be seen in Seattle from 1991 to 1992. The Seattle Water Department, which imposed water restrictions for the first time in almost 40 years, witnessed a 17 percent reduction in total system water

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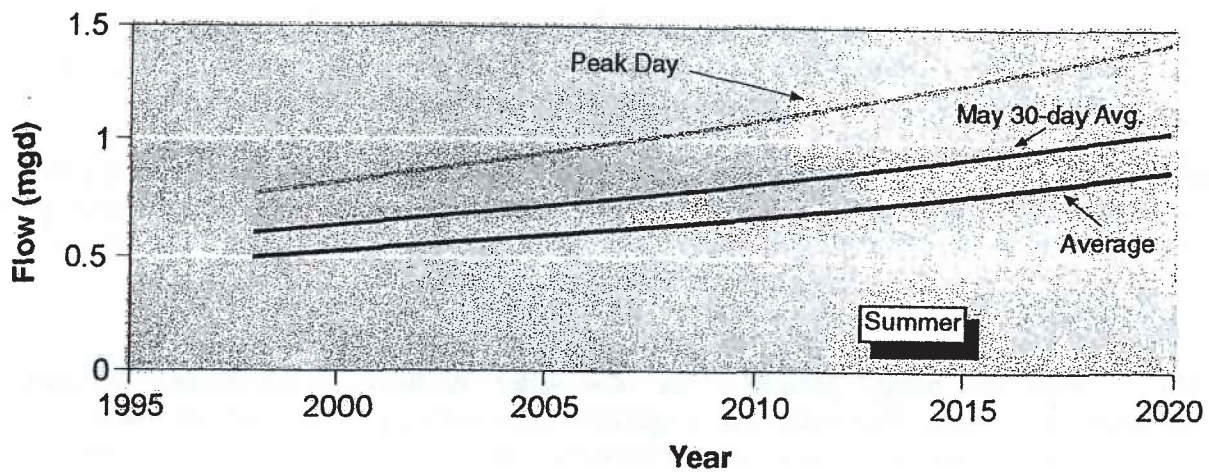

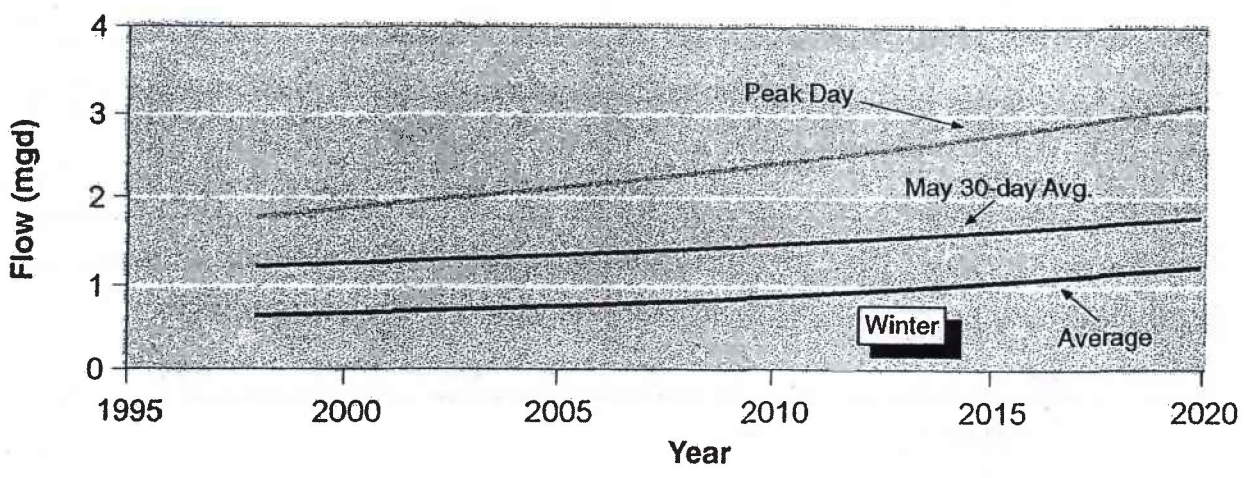

Redwood Wastewater Treatment Plant
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Figure 5-1
Wastewater Flow Projections

demand between 1991 and 1992. The summer demand in the three highest consumption months was reduced by a dramatic 33 percent. The sewage flow into Metro's West Point treatment plant, however, dropped only 5.8 percent during dry-weather flow. During winter months, water consumption dropped only 8 percent. This reduction in winter water usage did not appear to have a significant impact on the sewage treatment plant flow or operation.

Our review suggests that the District could anticipate a reduction in wastewater flows up to 10 percent annually if an aggressive water conservation program was instituted. To institute such a conservation program requires individual water meters, which the District lacks.

The District is currently served by either individually owned small community water systems or by individual wells serving single or multiple residences. Since none of these systems measure water usage, an aggressive water conservation program was not considered feasible.

5.1.3 Load Projections

Another important design parameter for wastewater facilities is wastewater pollutant concentrations or load. Wastewater load projections are based on existing wastewater load plus a typical load per capita assumed for all future connections. A summary of existing and future pollutant load estimates is shown in Table 5-3.

Table 5-3
Redwood WWTP BOD and TSS Load (lbs/day)

	1996-1998 Actual	2020 Projection
BOD		
Summer, Max. 30-day average[1]	1,122	2,400
Winter, Max. 30-day average[2]	1,589	3,100
Annual average[3]	843	2,070
TSS		
Summer, Max. 30-day average[4]	1,398	2,340
Winter, Max. 30-day average[5]	1,315	2,500
Annual Average[6]	879	1,980

(1) Existing BOD load plus 0.27 lbs/day-capita.
(2) Existing BOD load plus 0.32 lbs/day-capita.
(3) Existing BOD load plus 0.2 lbs/day-capita.
(4) Existing TSS load plus 0.20 lbs/day-capita.
(5) Existing TSS load plus 0.25 lbs/day-capita.
(6) Existing TSS load plus 0.20 lbs/day-capita.

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5.2 REGULATORY TREATMENT CRITERIA

To evaluate upgrading the existing Redwood WWTP, future NPDES discharge limits needed to be determined. This report subsection presents future discharge requirements for the Redwood WWTP as set by Oregon Administrative Rules. *Because the design flows vary only slightly from those used in the 1994 report, the analysis in this section was not revised.*

The existing NPDES permit effluent limitations for the Redwood WWTP are shown in Section 4.3 of this report.

The treatment level for future wastewater flows and loadings has been evaluated based on three (3) requirements:

- Treatment Requirements contained in Oregon Administrative Rules (OAR) 340-41-375.
- Treatment Requirements contained in OAR 340-41-026.
- Treatment Requirements based on receiving water quality criteria for the Rogue River.

5.2.1 OAR 340-41-375

Future criteria for the treatment plant are subject to the regulations outlined in OAR 340-41-375. A summary of those requirements is listed here:

- Monthly average for BOD and TSS shall not exceed 10 mg/L during "low stream flows" (summer period).
- Treatment shall meet secondary requirements during "high stream flows" (winter period).
- Effluent BOD concentrations in mg/L divided by the dilution factor (ratio of receiving stream flow to effluent flow) is not to exceed one (1) unless otherwise authorized by ODEQ. *This will not apply, however, since the minimum river flow is 1,100 times the peak day discharge.*
- Sixty (60) minutes of contact time will be provided to ensure adequate disinfection with 1 mg/L residual.

Based on these regulations, the anticipated future BOD and TSS effluent limitations for the Redwood plant would be as shown in Table 5-4.

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Table 5-4
Anticipated Future BOD and TSS Effluent Limitations
Based on OAR 340-41-375

	Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge (lbs/day)		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based On Effluent Quality							
Summer, MMDWF	1.05	10	15	—	88	132	176
Winter, MMWWF	1.80	30	45	—	450	675	900
Permit Limits Based On 85 Percent Removal							
Winter							
AWWF	1.30	(1) 24			260		
MMWWF	1.80	30			450		
Max. Week	1.85		44			675	
Max. Day	2.52			43			900
Summer							
ADWF	.94	10			(2) 78		
MMDWF	1.05	10			88		
Max. Week	1.12		14			132	
Max. Day	1.47			14			176

(1) Limit determined by 85 percent removal requirement and minimum influent load condition.
(2) Load limitation required by maximum concentration.

5.2.2 OAR 340-41-026

In addition to the regulations noted above, OAR 340-41-026 further requires that growth and development be accommodated by increased waste treatment efficiency and effectiveness rather than by increased permitted mass discharges. Based on these requirements, the anticipated future BOD and TSS effluent limitations for the Redwood WWTP would be as shown in Table 5-5.

Table 5-5
Anticipated Future BOD and TSS Effluent Limitations
Based on OAR 340-41-026

	Maximum Monthly Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge (lbs/day)		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based On Effluent Quality							
Summer, Dry Weather	1.05(1)	9	14	18	80	120	160
Winter, Wet Weather	1.80(2)	8	12	16	120	180	240

(1) Monthly dry weather flow with 5-year recurrence - MMDWF.
(2) Monthly wet weather flow with 5-year recurrence - MMWWF.

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Since the proposed mass discharge limitations shown in Table 5-3 exceed those listed in Table 5-4, ODEQ approval for increased permitted mass discharges for the Redwood WWTP would be required for these effluent limitations to apply to any plant expansion. Otherwise, Table 5-4 effluent limitations would apply for any Redwood WWTP expansion.

Increases have been accepted by ODEQ, provided that:

(1) No other reasonable alternatives exist except to lower water quality.

(2) This action is necessary and justifiable for economic or social development benefits and outweighs the environmental costs of lowered water quality.

(3) All water quality standards will be met and beneficial uses protected.

The suggested increase in the summer period (Table 5-5) is only from 80 to 88 lbs/day, or a 10 percent increase. This should be acceptable to ODEQ provided the provisions for exceptions are met; these will be reviewed in Section 6. This mass increase is based on the maximum month dry weather flow. The mass discharge for the average summer flow would not likely increase above existing NPDES permit levels. Since the proposed increase is small and would only occur for a brief period, it is also not likely to impact water quality (see Section 5.3).

During the winter period, a significant increase is proposed, due to increases in wastewater flows, while maintaining only secondary treatment standards. Allowable mass discharges are controlled, however, by the 85 percent removal requirement, as demonstrated in Table 5-5. If minimum load is experienced at or near average flows, the mass discharge limitations for the plant would be restricted to only 260 lbs/day, just over double the current NPDES permit monthly limit.

5.3 RECEIVING WATER QUALITY CRITERIA

5.3.1 Introduction

The Redwood WWTP discharges secondary effluent to the Rogue River, which is a valuable resource to the region, with beneficial uses downstream of the sewage treatment plant (as listed in OAR 340-41-362):

- Public Domestic water supply
- Private Domestic Water Supply
- Industrial Water Supply
- Irrigation
- Livestock Watering
- Anadromous Fish Passage
- Salmonid Fish Rearing and Spawning
- Resident Fish and Aquatic Life Habitat

- Fishing, Boating, and Water Contact Recreation
- Aesthetic Quality
- Commercial Navigation and Transportation

Each of these beneficial uses depends, to varying degrees, on water quality. Local and state agencies and the public recognize the need to protect the Rogue River from further water quality degradation, both from increased point and non-point discharges. The ODEQ has indicated that *"An expansion of the treatment plant could prompt tighter effluent limits, increased monitoring, and possible restrictions on discharge to surface waters (Belsky, D., ODEQ, 1991 Personal Communication)."*

Receiving water quality standards are promulgated in *Management Plan, Beneficial Uses, Policies, Standards, and Treatment Criteria in the Rogue River Basin*, OAR 340-41-362 and OAR 340-41-365. This section evaluates the ability of the existing and future discharge to meet Oregon's receiving water quality standards, thereby assuring protection of beneficial uses.

5.3.2 Discharge Description

Figure 5-2 shows the Redwood WWTP outfall from effluent outlet structure to discharge location in the Rogue River at river mile 97.7. From the chlorine contact chamber to Manhole #3, the outfall consists of 300 feet of 18-inch concrete pipe. From Manhole #3 to the discharge point, the outfall consists of 62 feet of 18-inch corrugated metal pipe (CMP). The open-ended discharge is located approximately 25 feet from the south bank at ordinary low river flows. Water depth from water surface to outfall crown is approximately 1.5 feet at ordinary low river flows.

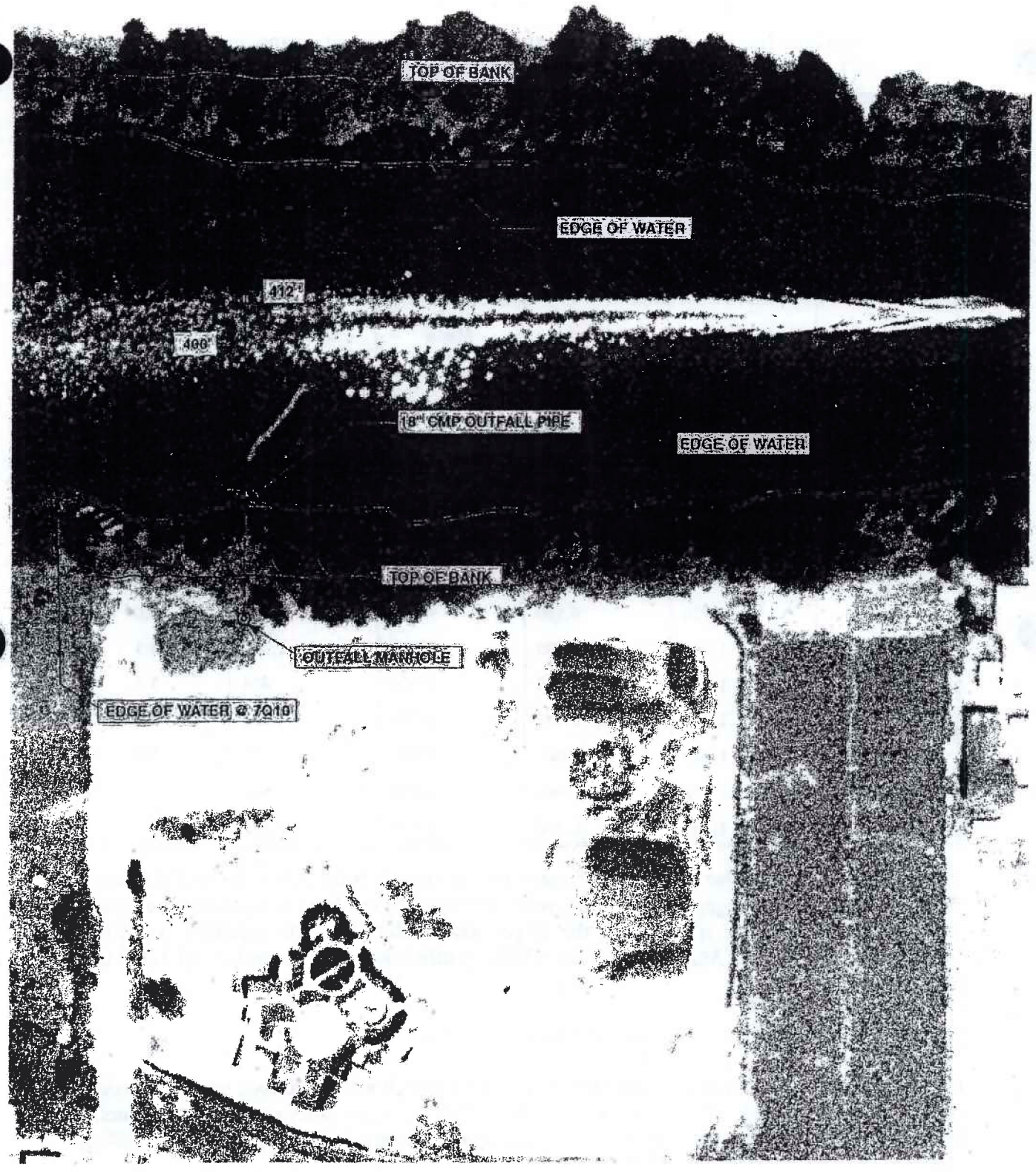
The invert of the 18-inch open-ended discharge pipe is six inches off the river bottom, facing offshore and downstream at a horizontal angle of 45 degrees. A riprap apron protects from scouring 10 feet downstream of the outfall. Appendix F shows the as-built outfall (CH2M Hill May 1974).

5.3.3 Receiving Water Conditions

Receiving water conditions, including river flows, channel characteristics, current speed, and ambient water quality, are discussed in the subsections that follow.

5.3.3.1 River Flow

The United States Geological Survey (USGS) operates a number of river flow gaging stations on the Rogue River, including a nearby station at Grants Pass, 0.6 miles upstream from the Highway 99 bridge at river mile 101.8. Since February 1977, Rogue River flow has been regulated by Lost Creek Dam with slight regulation by Fish Lake and Emigrant Lake. There are many water diversions from the Rogue River and its tributaries, the largest being 5.5 miles upstream for Savage Rapid Dam of the Grants Pass Irrigation District (USGS 1990).



Redwood Wastewater Treatment Plant
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Figure 5-2
Outfall and Vicinity
Redwood Sanitary Sewer Service District

River flow statistics (monthly, minimum, maximum, and mean flows) compiled by USGS from 1978 to 1987 are listed in Table 5-6. As Table 5-6 indicates, the season of lowest river flows is late summer to early autumn; during this low-flow period, ambient mixing is least, and receiving water quality most impaired. In this report, receiving water impacts will be evaluated at these critical low-flow conditions.

Table 5-6
Statistical Flow Summaries for Rogue River
After Regulation of Lost Creek Lake (1978-1987)

Month	Minimum (cfs)	Maximum (cfs)	Mean (cfs)	Standard Deviation (cfs)	Percent of Annual Runoff (%)
October	1,420	2,280	1,720	305	4.1
November	1,580	7,670	3,400	1,800	7.9
December	2,020	14,000	6,200	4,330	14.9
January	1,710	7,750	4,300	1,900	10.3
February	2,150	11,000	5,740	3,260	12.6
March	1,510	8,120	4,340	2,150	10.4
April	1,750	6,850	4,310	1,920	10.0
May	1,960	5,590	3,440	1,370	8.3
June	1,810	4,520	2,530	838	5.9
July	1,850	3,130	2,340	355	5.6
August	1,880	3,080	2,260	379	5.4
September	1,330	2,640	1,930	382	4.5
Annual	**1,930**	**5,270**	**3,530**	**1,160**	**100.0**

The regulatory low-flow for evaluation of water quality impacts is the 7Q10, or the 7-day low-flow that occurs, on average, once every 10 years (USEPA 1991). Data records are inadequate to compute this statistical measure for the Rogue River following flow regulation in 1977; however, the *Rogue River Management Plan* (1972) specifies that a minimum flow of 1,200 cfs be maintained.

5.3.3.2 Channel Characteristics and Current Speeds

Channel dimensions and water surface elevation near the outfall were measured by cross-section survey on November 30, 1993 (Gary Wicks 1993). The surveyed cross section is reproduced in Figure 5-3. River flow during the survey was approximately 1,360 cfs (Miller, J., USGS 1993).

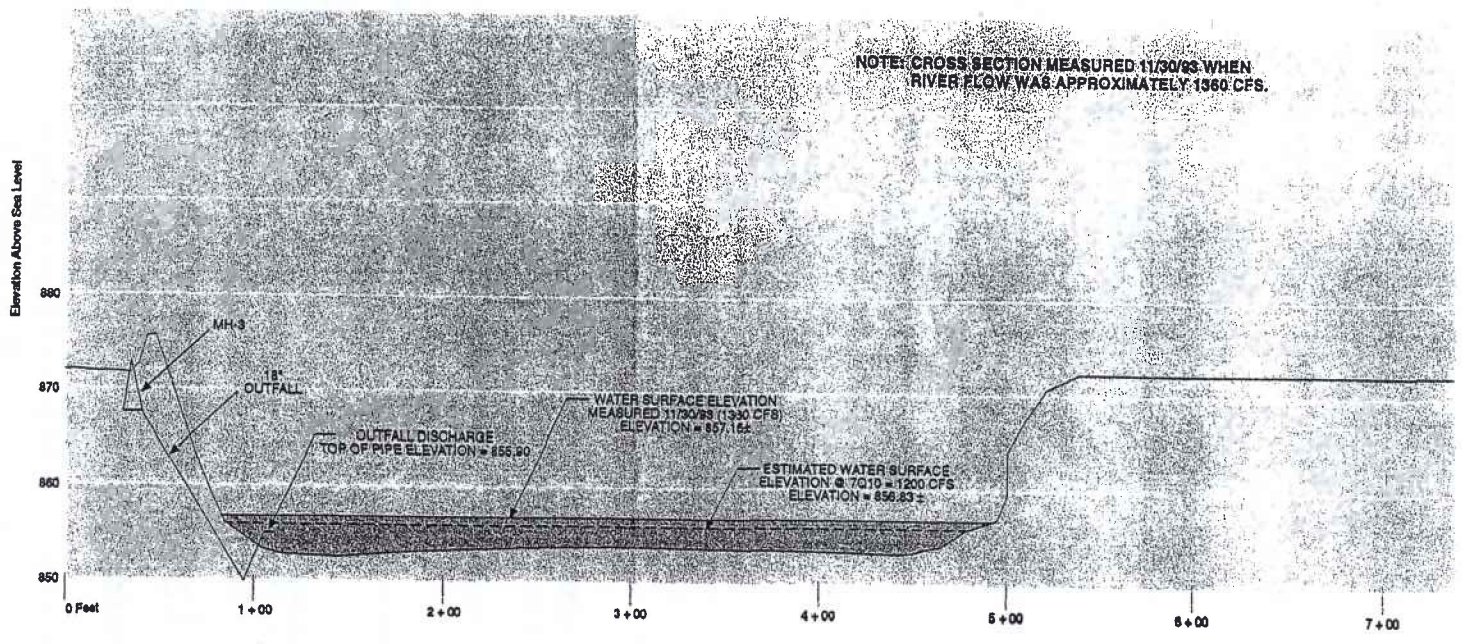

Figure 5-3
Rogue River Cross Section at Outfall
Scale: 1" Horiz = 50 ft
1" Vert = 10 ft

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Manning's Equation was used to estimate average river depth and velocity at the regulatory low river flow of 1,200 cfs:

$$U = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad \text{(Equation 5.1)}$$

where:

U = Average River Velocity
n = Manning's Roughness Coefficient
R = Hydraulic Radius (Area/Wetted Perimeter)
S = River Slope

R is approximately equal to the river depth, and Equation 5.1 can be rearranged to yield:

$$d = \left[\frac{Qn}{1.49\, W S^{1/2}} \right]^{3/5} \qquad \text{(Equation 5.2)}$$

where:

d = Average River Depth
Q = River Discharge
W = River Width

Table 5-7 summarizes channel characteristics and current speeds at field survey conditions and estimated at regulatory low-flow conditions.

Table 5-7
River and Channel Characteristics

Flow Condition	River Discharge	River Width	River Depth		Average Current Speed
			Average	At Outfall	
Nov. 30, 1993 Survey	1,380 cfs	412 ft	2.6 ft	2.8 ft	1.29 ft/s
Regulatory Low	1,200 cfs	400 ft	2.4 ft	2.6 ft	1.25 ft/s

5.3.3.3 Ambient Water Quality

The USGS maintains a database of Rogue River water quality measurements. Approximate monthly records for two stations were obtained and analyzed to determine statistical measures of water quality parameters (Marxer, L., November 1993, Personal Communication). Sample

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stations are located 2.5 miles west of Grants Pass, upstream of the Redwood outfall, and at the Robertson Bridge, approximately 11 miles downstream of the Redwood outfall.

A larger record is available at the Robertson Bridge station; however, the Robertson Bridge data must be interpreted with caution if used to represent conditions near the Redwood plant because the Applegate River contributes to the Rogue River flow upstream of the Robertson Bridge sampling station and downstream of the outfall. The annual mean flow of the Applegate River is 716 cfs (river mile 7.6), or 20 percent of the Rogue River mean flow. The influence of the Applegate River diminishes during the season of lowest river flows. Mean flow of the Applegate River is 26 cfs in August and 40 cfs in September, or less than 4 percent of the Rogue River flow during these months (USGS 1990). Table 5-8 summarizes water quality statistics for the Rogue River at both stations for the period of record since flow regulation (August 1977 – March 1993).

5.3.4 Water Quality Standards

In September 1991, ODEQ completed its most recent triennial review of State Water Quality Standards, as required by the Clean Water Act. The resulting amendments to OAR 340, Chapter 41, pertaining to receiving water quality, have profound effects on municipal and private dischargers. Amendments include changes in policies involving:

- Antidegradation
- Bacteria
- Mixing Zones
- Toxic Substances
- Biological Criteria
- Turbidity

ODEQ is implementing new policies to be used both to detect and address potential toxics problems. The steps of this policy framework will be executed through the NPDES permitting program and will include:

- *Basic Testing and Reporting Program* – Includes both chemical analysis and biomonitoring. Frequency will be determined through key factors such as dry weather design flow, industrial inputs to the treatment system, and quantity of sludge produced.

- *Evaluation of Receiving Water* – Analyses dilution at the point of discharge and at the mixing zone boundary to determine compliance with instream water quality standards.

- *Accelerated Testing Program* – Initiated basic monitoring program if results indicate that water quality standards violations could occur.

- *Effluent Limits* – Established when discharges cause an adverse effect on beneficial uses by violating water quality standards outside of an authorized mixing zone.

- *Implementation Plans and Schedules* – When needed to eliminate toxicity problems.

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Table 5-8
Ambient Water Quality Statistics for Rogue River Near Redwood Wastewater Treatment Plant

Conventionals	Units	Number of Samples		Maximum		Minimum		Mean		5th Percentile		50th Percentile (Median)		95th Percentile	
		G.P.[1]	R.B.[2]	G.P.	R.B.	G.P.	R.B.	G.P.	R.B.	G.P.	R.B.	G.P.	R.B.	G.P.	R.B.
Temperature	°C	31	113	22.0	23.5	4.0	4.0	12.1	12.7	5.0	5.0	11.0	12.5	20.2	20.7
pH	SU	31	113	8.2	9.0	6.2	7.0	7.4	8.0	6.3	7.3	7.5	8.0	8.2	8.6
Dissolved Oxygen	mg/ℓ	31	109	13.9	14.6	9.9	9.3	11.7	11.5	10.2	9.9	11.7	11.5	13.3	13.4
Turbidity	HACH FTU	31	112	56.0	46.0	1.0	1.0	6.9	4.6	1.0	2.0	3.0	2.0	29.0	12.5
Total Hardness	mg/ℓ as $CaCO_3$	31	72	48	53	28	28	38	38	31	29	38	39	47	47
Total Ammonia	mg/ℓ as N	31	113	0.210	0.150	0.030	0.020	0.091	0.051	0.050	0.020	0.080	0.040	0.140	0.104
Fecal Coliforms	MPN/100 mℓ	31	106	4,600	2,400	30	<2	368	232	<30	23	91	<93	940	930
Metals[3]															
Arsenic	µg/ℓ	8	4	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5
Cadmium	µg/ℓ	8	4	<1	4	<1	<1	<1	<1.75	<1	<1	<1	<1	<1	4
Chromium	µg/ℓ	8	4	<50	<50	<2	<2	<20	<38	<2	<2	<2	<50	<50	<50
Copper	µg/ℓ	8	4	<50	<50	<2	<2	<21	<36	<2	<2	3	<50	<50	<50
Lead	µg/ℓ	8	4	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Mercury	µg/ℓ	8	4	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5	<0.5
Silver	µg/ℓ	5	4	<10	<10	<1	<1	<1	<1	<1	<1	<1	<1	<10	<10
Selenium	µg/ℓ	4	4	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5
Zinc	µg/ℓ	8	4	40	39	3	<10	<14	<17	3	<10	11	10	40	39

(1) G.P. – 2.5 miles downstream of Grants Pass.
(2) R.B. – Robertson Bridge, 11 miles downstream of Redwood WWTP.
(3) Total metals, including dissolved and particulate fractions.

The Rogue River, downstream of the confluence with the Applegate River, is designated as a Wild and Scenic River under the *1968 Wild and Scenic Rivers Act* (Public Law 90-542). The Wild and Scenic Rivers Act stipulates that the Rogue River must remain in a free-flowing condition without water quality degradation.

The Bureau of Land Management (BLM) and the U.S. Forest Service administer the Rogue River Wild and Scenic Rivers Act. In 1972, they published a joint *Rogue River Management Plan* for the development, operation, and management of the Rogue River. The management plan outlines the responsibilities of all local, county, and state jurisdictions. The ODEQ is responsible for ensuring that water quality and waste treatment standards are met.

The ODEQ has responded to the need for water quality protection in the Rogue River by promulgating OAR 340-41-362 and OAR 340-41-365, outlining specific receiving water criteria to be met in the Rogue River Basin. These criteria are discussed here.

5.3.4.1 Anti-Degradation

Notwithstanding the water quality standards described below, the highest and best practicable treatment and/or control of wastes, activities, and flows shall in every case be provided to maintain dissolved oxygen and overall water quality at the highest possible levels and water temperatures, coliform bacteria concentrations, dissolved chemical substances, toxic materials, radioactivity, turbidities, color, odor, and other deleterious factors at the lowest possible levels.

5.3.4.2 Dissolved Oxygen (DO)

DO concentrations shall not be less than 90 percent of saturation at the seasonal low flow or less than 95 percent of saturation in salmonid fish spawning areas during spawning, incubation, hatching, and fry stages.

5.3.4.3 Temperature

No measurable temperature increases shall be allowed outside the assigned mixing zone, as measured relative to a control point immediately upstream from a discharge when stream temperatures are 58°F or greater; or more than 0.5°F increase due to a single-source discharge when receiving water temperatures are 57.5°F or less; or more than 2°F increase due to all sources combined when stream temperatures are 56°F or less.

5.3.4.4 Turbidity

The measurement standard for turbidity is the Nephelometric Turbidity Unit (NTU) reflecting current analytical procedures. No more than a 10 percent cumulative increase in natural stream turbidities is allowed, as measured relative to a control point immediately upstream of the turbidity causing activity. Some dredging and limited duration activities are exempt.

5.3.4.5 pH

The pH levels must not fall outside the 6.5 - 8.5 range.

5.3.4.6 Bacteria

Effluent limitations for fecal coliform will remain in effect until permit renewal, or until the ODEQ reopens existing permits to include an effluent limit and compliance schedule for enterococci [OAR 340-41-365(2)(e)(D)].

The Redwood WWTP is currently monitoring under the fecal coliform standard. The NPDES limitation is a monthly geometric mean of 200 fecal coliforms per 100 mL and weekly geometric mean of 400 fecal coliforms per 100 mL. The fecal coliform standard is an "end of pipe" limitation. The new enterococci bacteria standard in the Rogue River is a monthly average geometric mean of 33 per 100 mL, with no sample exceeding 61 per 100 mL.

5.3.4.7 Total Dissolved Solids

Total dissolved solids are not to exceed 500.0 mg/L.

5.3.4.8 Toxic Substances

Acute and chronic numeric criteria for a wide range of toxicants must be met within the zone of immediate dilution (ZID) and authorized mixing zone. Numeric criteria are listed in Table 20 of OAR 340-41, which is based on *Quality Criteria for Water* (EPA 1986). Where the natural quality parameters of waters of the Rogue River basin are outside the numerical limits of the above-assigned water quality standards, the natural water quality shall be the standard.

5.3.4.9 Ammonia

Recently, a standard has been added to the toxicant list in Table 20, OAR 340-41, to acknowledge the need for aquatic life protection from un-ionized ammonia. Allowable ammonia concentrations are dependent on receiving water temperature and pH, which are most critical during the summer low-flow period.

5.3.5 Mixing Zone Evaluation

This section outlines the approach and results of hydrodynamic model mixing predictions. Section 5.3.5.1 discusses the Redwood WWTP mixing zone.

5.3.5.1 Authorized Mixing Zone

The ODEQ has allowed a mixing zone for the Redwood discharge. A mixing zone is defined in OAR 340-41-365 as a "designated portion of a receiving water to serve as a zone of dilution for wastewaters and receiving waters to mix thoroughly." The department may suspend all or part of the water quality standards or set less restrictive standards in the authorized mixing zone.

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Redwood WWTP's mixing zone is described in NPDES Permit #100868 as having a radius of 100 feet from the point of discharge.

In determining the size of the mixing zone, the department may use appropriate mixing zone guidelines to assess the biological, physical, and chemical character of the receiving waters and effluent and the most appropriate placement of the outfall to protect instream water quality, public health, and other beneficial uses. The mixing zone must also:

- Be as small as possible.
- Avoid overlap with any other mixing zones to the extent possible and be less than the total stream width as necessary to allow passage of fish and other aquatic organisms.
- Be free of materials in concentrations that cause acute and chronic toxicity to aquatic life as measured by an approved bioassay.

Acute and chronic toxicity is defined in OAR 340-41-365(4) as follows:

> *Acute Toxicity* – Acute toxicity is lethality to aquatic life as measured by a 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 due to ammonia and chlorine only when it is demonstrated on a case-by-case basis that immediate dilution of effluent within the mixing zone reduces toxicity below lethal concentrations. The department may, on a case-by-case basis, establish a ZID if appropriate for other parameters.
>
> *Chronic 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 test species life cycle.

The Redwood WWTP NPDES permit does not define the ZID; however, EPA guidance suggests a distance 10 percent of the distance to the mixing zone boundary is appropriate. This translates to 10 feet with the current Redwood WWTP mixing zone (USEPA 1991).

Redwood WWTP's current authorized mixing zone may be too restrictive. For comparison, the City of Grants Pass WRP, several miles upstream of the Redwood plant, has a mixing zone three times that allotted for the Redwood plant, although average discharge is ten times greater than the Redwood plant. The Grants Pass mixing zone is defined in the Grants Pass NPDES Permit #100989 as follows:

> The allowable mixing zone is that portion of the Rogue River from 10 feet above the point of discharge to 300 feet downstream of the discharge point. In addition,

the ZID shall not exceed 10 percent of the defined mixing zone in any direction from the point of discharge.

The current average dry weather design flow for the Grants Pass plant is 4.7 mgd (Brown and Caldwell 1992).

OAR 340-41-365(4) suggests that the size of Redwood WWTP's mixing zone be reevaluated. Prior to adjusting mixing zone dimensions, acute and chronic bioassay testing should be completed as required for assessing toxicity in the mixing zone. The mixing zone should not occupy greater than 25 percent of the river width to allow passage of fish and other aquatic organisms. Additional monitoring recommendations are given in Section 5.3.8.

5.3.5.2 Mixing in Rivers

A submerged river discharge has two distinct mixing regions: near-field and far-field. Mixing in the near-field is vigorous from rapid dissipation of jet momentum. Near-field mixing is primarily a function of port size and outlet velocity. Dispersion of effluent in the far-field is driven by turbulence in the receiving waters which is a function of current speed and channel characteristics. Far-field mixing rates are greatly reduced from near-field mixing rates.

5.3.5.3 Model Methodology

Several EPA-approved river mixing models are available to predict acute and chronic mixing ratios. Those used in the Redwood analysis included:

- PLUMES (Baumgartner, D.J.; Frick, W.E.; Roberts, P.J.W.; and Bodeen, C.A.; June 1993)
- Turbulent Advective Dispersion Equation (Fischer 1979)

Each model is discussed in the sections that follow.

PLUMES

The PLUMES Model was used to predict near-field mixing ratios. PLUMES is an interface linking the near-field model UM (Updated UMERGE) and a far-field model based on Brooks Equation (Fischer 1979). PLUMES is normally run in marine environments; however, the model does support shallow river discharges. In the shallow discharge mode, PLUMES is constrained to co-flowing jets and weakly or unstratified receiving waters. The Redwood discharge fits these criteria.

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Turbulent Advective Dispersion Equation

The turbulent advective diffusion equation is a conservative estimate of dilution since there is no consideration for near-field mixing. The turbulent advective diffusion equation in three dimensions for a continuous point source (Fischer 1979) is as follows:

$$C = \frac{\dot{M}}{4\pi x\sqrt{\epsilon_v \epsilon_t}} \exp^{-}\left[\frac{U}{4x}\left[\frac{y^2}{\epsilon_t} + \frac{z^2}{\epsilon_v}\right]\right] \qquad \text{(Equation 5.3)}$$

where:

C = Concentration at Coordinate (x, y, z)
$\dot{M}$ = Mass Flux (Concentration x Effluent Discharge)
x = Downstream Distance from Outfall
ϵ_v = Vertical Turbulent Diffusion Coefficient
ϵ_t = Lateral Turbulent Diffusion Coefficient
U = Average River Current Speed
y = Lateral Distance from Outfall
z = Vertical Distance from Outfall

Lateral and vertical turbulent diffusion coefficients are a measure of river-induced mixing. A larger diffusion coefficient indicates a higher degree of effluent mixing. The lateral turbulent diffusion coefficient is calculated as follows (Fischer et al. 1979):

$$\epsilon_t = 0.6du* \qquad \text{(Equation 5.4)}$$

where:

d = Average River Depth
g = Acceleration due to Gravity (32.2 ft/sec^2)
S = River Slope
$u*$ = *Shear Velocity* = $\sqrt{gdS}$

The vertical turbulent diffusion coefficient is roughly 1/10 the lateral mixing coefficient and is calculated as follows:

$$\epsilon_v = 0.067du* \qquad \text{(Equation 5.5)}$$

where:

d and $u*$ are as previously described.

The three-dimensional form of the advective diffusion equation is used when the discharge cannot be considered completely vertically mixed; however, if the plume does become completely vertically mixed, the two-dimensional form of Equation 5.5 may be used:

$$C = \frac{\dot{M}}{d\sqrt{4\pi\epsilon_t xU}} \exp^{-}\left[\frac{Uy^2}{4x\epsilon_t}\right] \qquad \text{(Equation 5.6)}$$

Complete vertical mixing can be predicted using the following equation (Fischer, 1979).

$$L = \frac{C'UW^2}{\epsilon_v} \qquad \text{(Equation 5.7)}$$

where:

C′ = 0.1 (mid-depth discharge)
U = Average River Current Speed
W = River Width
ϵ_v = Vertical Turbulent Diffusion Coefficient

Distance to complete vertical mixing in the Rogue River using Equation 5.7 is 30 feet at regulatory low river flow. Actual distance to complete vertical mixing will be somewhat less than this prediction due to near-field mixing, which is not taken into account. Complete vertical mixing occurs prior to the current authorized mixing zone boundary, 100 feet downstream; therefore, Equation 5.6 can be used for far-field mixing predictions.

When the plume interacts with the river banks, surface, or bottom, image sources must be used to compensate for plume spread beyond these boundaries. One image source was used to account for near-bank interaction, 25 feet distant.

Another useful calculation is plume width at a fixed downstream distance:

$$b = 4\sigma = 4\sqrt{\frac{2\epsilon_t x}{U}} \qquad \text{(Equation 5.8)}$$

where:

b	=	Plume Width
σ	=	Standard Deviation of Guassian Plume Concentration Profile
ϵ_t	=	Lateral Turbulent Diffusion Coefficient
x	=	Downstream Distance
U	=	Average River Current Speed

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5.3.5.4 Far-Field Model Calibration

The far-field model was calibrated using the results of the *City of Grants Pass Mixing Zone Study* (Brown and Caldwell 1992). In this mixing zone study, a fluorescent dye tracing study was used to measure lateral and vertical mixing of the Grants Pass discharge downstream of the outfall. The Grants Pass outfall discharges near the north bank of the Rogue River at river mile 100.9.

Each river has particular mixing characteristics, that depend on channel characteristics such as roughness, current speed, tortuosity, and depth. Plume width measurements at 250 and 1,000 feet downstream of the Grants Pass discharge allowed calculation, using Equation 5.8, of the lateral turbulent diffusion coefficient (ϵ_t) for the Rogue River near Grants Pass.

The Grants Pass study results were used to determine u^* (shear velocity), and Equation 5.4 was used to extrapolate ϵ_t for application near the Redwood WWTP. The lateral turbulent diffusion coefficient for the Rogue River near the Redwood discharge—under regulatory, low-flow conditions–is estimated as 0.22 ft^2/sec. The vertical turbulent diffusion coefficient is estimated as 0.024 ft^2/sec.

5.3.5.5 Model Input

Near- and far-field model input parameters are summarized in Table 5-9. Two statistical measures of effluent flows were used in the modeling analysis. Acute mixing ratios are calculated at peak day effluent flows to correspond with the short duration exposures leading to acute toxicity. Acute criteria are evaluated at the ZID. Chronic mixing ratios are calculated at peak month flows to correspond with the longer duration exposures associated with chronic toxicity. Chronic criteria are evaluated at the mixing zone boundary. Effluent flow projections are discussed in more detail in Section 5.1.

5.3.5.6 Model Results

Table 5-10 summarizes near- and far-field model results. The dilution ratio is the inverse of percent effluent. Results are given for both the current authorized mixing zone boundaries as well as at distances which may be considered for mixing zone expansion. Complete model results appear in Appendix F.

Table 5-9
Near- and Far-Field Model Input Parameters

Parameter	Source	Near-Field Model (PLUMES)	Far-Field Model (Advective Dispersion Equation)	Comment
PLANT				
Existing Flow	Oct. 1992-Sept. 1993 Daily Monitoring Reports (DMRs)	0.54 mgd (summer peak day)	0.48 mgd (summer peak month)	
Future Flow	Projected (see Section 5.1)	1.1 mgd (summer peak day)	1.0 mgd (summer peak month)	
Effluent Temperature	Estimate	21°C	N/A	Mixing not sensitive to this parameter
Effluent Salinity	Estimate	0	N/A	Mixing not sensitive to this parameter
OUTFALL				
Port Diameter	As-built Drawings (see Appendix F)	18"	18"	
Vertical Angle	As-built Drawings (see Appendix F)	0°	0°	
Horizontal Angle	As-built Drawings (see Appendix F)	Co-Flowing	Co-Flowing	Actual angle 45° with respect to currents
Contraction Coefficient	Estimate	0.95	N/A	
Port Depth	November 30, 1993 Survey (Gary Wicks Engineering)	1.85 ft	Mid Level	To center of port
Port Elevation	November 30, 1993 Survey (Gary Wicks Engineering)	1.3 ft	Mid Level	To center of port
RECEIVING WATERS				
River Flow	USGS, 1990	1,200 cfs	1,200 cfs	Regulatory low flow
Average Current	Equation 5.1	1.25 ft/s	1.25 ft/s	Regulatory low flow
River Depth	November 30, 1993 Survey (Gary Wicks Engineering)	2.8 ft (at outfall)	2.4 ft (average)	Regulatory low flow

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Table 5-10
Mixing Model Results

Mixing Criteria	Distance (ft)	Percent Effluent		Dilution Ratio		Plume Width (ft)
		Existing[1] (%)	Future[2] (%)	Existing	Future	
Acute	10	24	50	4.2	2.0	7.5
	20	15	36	6.5	2.8	10.6
	30	12	29	8.5	3.4	13.0
Chronic	100	1.7	3.5	59.8	28.7	23.7
	200	1.2	2.6	80.2	38.5	33.6
	300	1.1	2.3	90.4	43.4	41.1

(1) At existing summer plant flows (May-November).
(2) At future summer plant flows (May-November).

5.3.6 Effluent Quality

5.3.6.1 Chlorine

Chlorine residual statistical variability was determined from Discharge Monitoring Reports (DMRs) for the period October 1992 to September 1993. The following results were obtained:

Number of Samples: 365

Low:	0.0 mg/L
High:	1.1 mg/L
Mean:	0.44 mg/L

5th Percentile:	0.10 mg/L
50th Percentile:	0.40 mg/L
95th Percentile:	0.80 mg/L

5.3.6.2 Fecal Coliforms

Fecal coliform samples are collected once each week. DMRs for the period January 1992 to September 1993 show that permit limits are consistently met:

Number of Samples: 156

Low:	0/100 mL
High:	109/100 mL
Weekly Average:	6/100 mL
Monthly Average:	7/100 mL

These results would suggest that the chlorine dose could be reduced and still meet permit limits.

5.3.6.3 Biochemical Oxygen Demand (BOD)

Summer and winter BOD concentrations for the past six-year period are shown in Appendix C. Maximum summer month BOD was 25 mg/ℓ (October 1992).

5.3.6.4 Total Suspended Solids (TSS)

Summer and winter TSS concentrations for the past six-year period are shown in Appendix C. Maximum summer month TSS was 20 mg/ℓ (June 1991).

5.3.6.5 Turbidity and Suspended Solids

The Redwood WWTP does not sample for turbidity; however, results from the Pullman WWTP in Pullman, Washington, can be used to represent typical secondary effluent values. The Pullman WWTP uses a process similar to the Redwood WWTP; however, turbidity values for the Redwood WWTP may be less than for Pullman, which has a higher summer suspended solids effluent limitation of 30 mg/L monthly and 45 mg/L weekly.

Turbidity of the Pullman effluent was measured three times daily from January 1993 to March 1993. Results of sampling are given below:

Average ± Standard Deviation:	3.62 ± 3.04
Maximum:	18.6
Minimum:	0.5
95th Percentile:	9.7

5.3.6.6 Temperature

High effluent temperatures are critical for receiving water impacts. At high river temperatures, dissolved oxygen (DO) concentrations are lowered, which can adversely affect aquatic life. Effluent temperature is not measured at the Redwood WWTP; however, records from the Grants Pass WRP show effluent temperatures there may reach as high as 76°F. Similar summer effluent temperatures are expected at the Redwood WWTP and will likely periodically exceed 70°F (Brown and Caldwell 1992). Effluent temperature monitoring at the Redwood WWTP should be initiated.

5.3.6.7 Ammonia, Metals, Cyanide, Phenols

A grab effluent sample was analyzed for ammonia, metals, cyanide, and phenols in December 1993 by Nielson Laboratories, Medford, Oregon. Toxicants including pesticides, PCBs, and volatile organic chemicals (VOCs) were not tested in the effluent, because they were not likely to pose a concern for the primarily municipal influent.

EPA guidance from the *Technical Support Document for Water Quality Based Toxics Control* (USEPA 1991) requires use of a reasonable potential multiplier to account for effluent variability

when less than 20 effluent samples have been measured. The multiplier used for evaluating water quality standards compliance will be based on a 95th percentile probability basis and 95th percentile distribution. A coefficient of variation of 0.6 is assumed unless data are available for its calculation.

No ammonia was detected in the Redwood sample. Typical secondary effluent ammonia concentrations are highly variable and depend in part on effluent temperature, pH, hydraulic detention time in the plant, and wastewater composition and strength. Each of these parameters can vary seasonally.

To better represent ammonia variability in the Redwood effluent, testing results for the Grants Pass WRP will be used in lieu of the single Redwood sample. Results of limited ammonia testing in June 1991 are presented below (Brown and Caldwell 1992):

Number of Samples:	5
Low:	9.5 mg/L
High:	12.3 mg/L
Average:	11.1 mg/L

5.3.6.8 pH

Redwood's NPDES permit stipulates that effluent pH be in the range 6.0 to 9.0. Effluent pH is measured three times weekly and ranges from 6.7 to 7.8 (January 1992-September 1993).

5.3.7 Water Quality Analysis

In this section, the ability of the existing and future Redwood discharge to comply with water quality standards is evaluated. Tables 5-11 and 5-12 summarize the water quality analysis and compliance determination for each parameter analyzed. Information provided in Table 5-12 includes:

- Ambient concentrations from STORET data (see Table 5-3).
- Effluent concentrations from DMRs, laboratory effluent scan (see Appendices B and D) or are estimated based on typical secondary treatment plant performance (i.e., Pullman WWTP and Grants Pass WRP).
- Laboratory detection limits, when not detected.
- Number of samples analyzed.
- Reasonable potential multipliers for toxicants with less than 20 representative samples (USEPA 1991).
- Freshwater acute and chronic water quality standards from Table 20, OAR-340-41 and from numeric criteria in OAR 340-41-365.
- Mixing ratios required to meet water quality standards.

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Table 5-11
Water Quality Analysis Summary

Parameter	Units	Critical Ambient Concentration	Effluent			Water Quality Standard OAR 340-41		Mixing Ratio Required	
			Critical Concentration	Number of Samples	RPM	Acute	Chronic	Acute (ZID)	Chronic (Mixing Zone)
CONVENTIONAL									
Dissolved Oxygen	mg/L	10.2	4.0	NA	1	—	95% saturation	—	13.4
Temperature	°F	58°	68°	NA	1	—	<0.25 decrease	—	40
pH	SU	6.3-8.2	6.7-7.8	252	1	—	6.5-8.5	—	1
Total Suspended Solids	mg/L	NA	(4) 30	365	1	—	500	—	1
Turbidity	NTU	1.0	9.7	NA	1	—	<10% increase	—	87
Fecal Coliforms	MPN/100 mL	940	109	156	1	—	200	—	1
TOXICANTS									
Chlorine	mg/L	0	(3) 0.80	365	1	0.019	0.011	42.1	72.7
Ammonia	mg/L as N	0.14	(5) 12.3	1	1	(1) 3.3	(1) 0.45	3.8	39.2
Cyanide	µg/L	NA	<20	1	6.2	22	5.2	5.6	23.8
Phenols	µg/L	NA	<50	1	6.2	10,200	2,560	1	1
Arsenic	µg/L	<5	<1	1	6.2	360	190	1	1
Cadmium	µg/L	<1	0.2	1	6.2	(2) 1.05	(2) 0.45	4.8	(6) 1
Chromium (III)	µg/L	<2	<50	1	6.2	(2) 665	(2) 79.3	1	4.0
Copper	µg/L	3	24	1	6.2	(2) 5.9	(2) 4.4	50.3	10.4
Lead	µg/L	<10	<1	1	6.2	(2) 18.4	(2) 0.72	1	(6) 8.6
Mercury	µg/L	<0.5	<0.2	1	6.2	2.4	0.012	1	(6) 103
Nickel	µg/L	NA	<40	1	6.2	(2) 527	(2) 58.5	1	4.2
Selenium	µg/L	<5	<2	1	6.2	20.0	5.0	1	(6) 2.5
Silver	µg/L	<1	<10	1	6.2	(2) 0.54	0.12	115	517
Zinc	µg/L	40	85	1	6.2	(2) 43.4	(2) 39.3	143	(7) —

(1) Criteria based on *Ambient Aquatic Life Water Quality Criteria for Ammonia* (USEPA 1989) using highest observed pH and temperature at Grants Pass (pH=8.2, Temp.=22.0°C).

(2) Criteria hardness dependent (95th percentile value of 31 mg/L as $CaCO_3$ critical).

(3) 95% percentile value from daily measurement.

(4) Permit limitation.

(5) Based on maximum of 5 measurements at Grants Pass Water Pollution Control Facility (6/13/91 - 6/28/91)

(6) Background assumed zero.

(7) Background is standard.

NA - Not Available

RPM - Reasonable Potential Multiplier

Table 5-12
Compliance Summary

Parameter	Scenario[1]							
	Current Flow				Future Flow			
	100′ Radius		300′ Radius		100′ Radius		300′ Radius	
	Acute	Chronic	Acute	Chronic	Acute	Chronic	Acute	Chronic
CONVENTIONAL								
Dissolved Oxygen	—	Yes	—	Yes	—	Yes	—	Yes
Temperature	—	Yes	—	Yes	—	No	—	Yes
pH	—	Yes	—	Yes	—	Yes	—	Yes
TSS	—	Yes	—	Yes	—	Yes	—	Yes
Turbidity	—	No	—	Yes	—	No	—	No
Fecal Coliforms	—	Yes	—	Yes	—	Yes	—	Yes
TOXICANTS[2]								
Chlorine	No	No	No	Yes	No	No	No	No
Ammonia	Yes	Yes	Yes	Yes	No	No	No	Yes
Cyanide	LDN	Yes	Yes	Yes	LDN	Yes	LDN	Yes
Phenols	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Arsenic	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Cadmium	LDN	Yes	Yes	Yes	LDN	Yes	LDN	Yes
Chromium (III)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Copper	No	No	No	No	No	No	No	No
Lead	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Mercury	Yes	LDN	Yes	LDN	Yes	LDN	Yes	LDN
Nickel	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Selenium	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Silver	LDN	LDN	LDN	LDN	LDN	LDN	LDN	LDN
Zinc	No	No	No	No	No	No	No	No

(1) All scenarios assume current plant effluent quality.
(2) Compliance based on detection limits when not detected.
LDN – Lower effluent detection limits and more effluent samples needed to assess compliance.

Table 5-11 includes a listing of each parameter and determination of compliance with water quality standards at each combination of existing and future predicted summer plant flows and current and expanded mixing zone boundaries (100 feet to 300 feet). Compliance is discussed for each parameter in the following sections.

5.3.7.1 Dissolved Oxygen

Salmonid species are present in the Rogue River and require that 95 percent DO saturation be maintained. Figure 5-4 shows mean values and 95th percentile confidence limits of DO concentrations and DO saturation for the Rogue River at all STORET ambient monitoring stations. Values are also shown separately by month for the Grants Pass station (Brown and Caldwell 1992).

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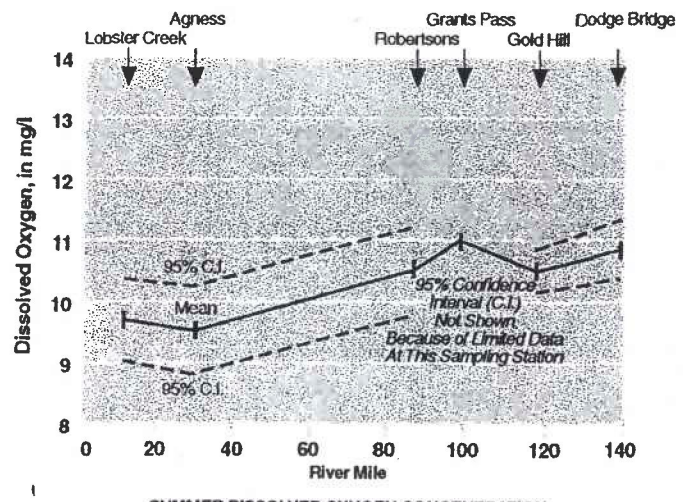
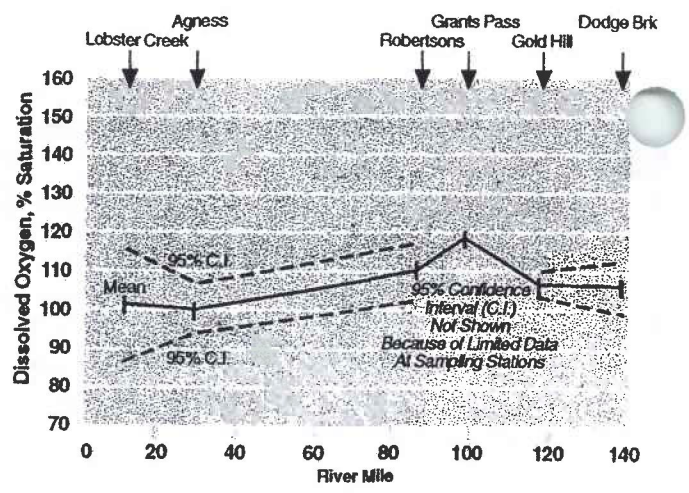
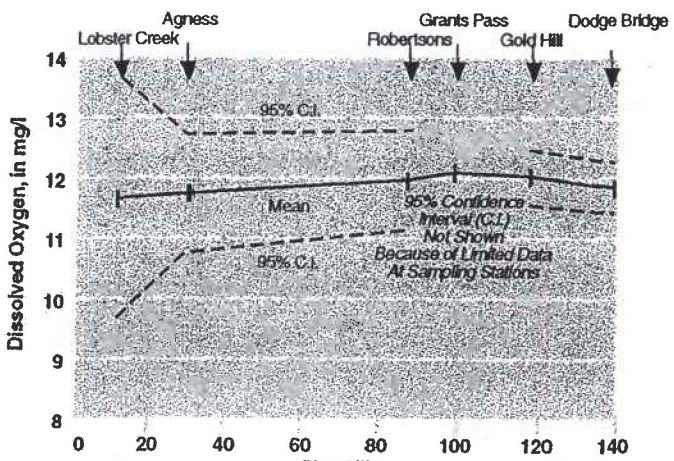


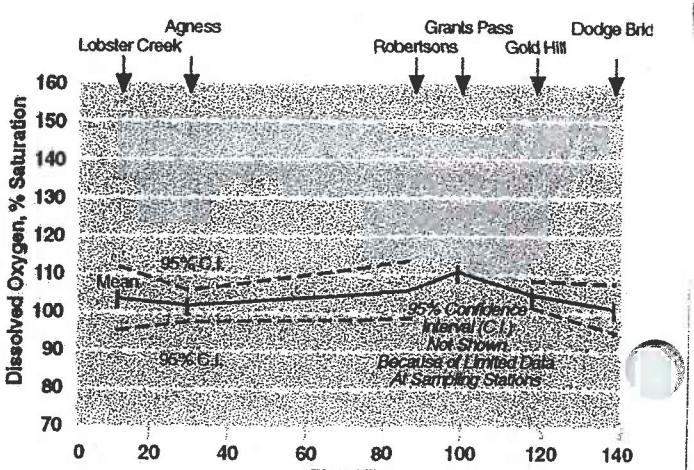
SUMMER DISSOLVED OXYGEN CONCENTRATION



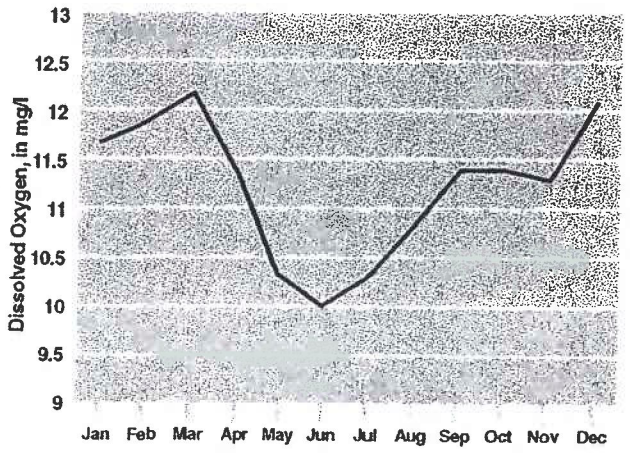
SUMMER DISSOLVED OXYGEN SATURATION



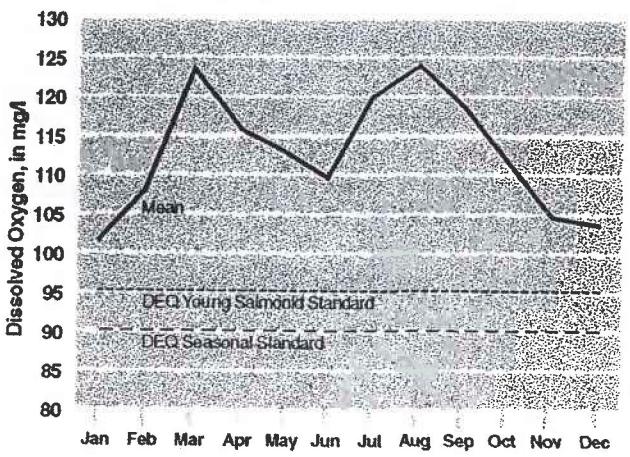
WINTER DISSOLVED OXYGEN CONCENTRATION



WINTER DISSOLVED OXYGEN SATURATION



MINIMUM DISSOLVED OXYGEN CONCENTRATIONS AT GRANTS PASS



DISSOLVED OXYGEN SATURATION AT GRANTS PASS

Source: Reproduced from Water Restoration Plant Facilities Plan Draft, City of Grants Pass (Brown and Caldwell, 1992)

Figure 5-4
Rogue River
Dissolved Oxygen

Average DO saturation values exceed 95 percent in both summer and winter for the entire river length. At Grants Pass and Robertson Bridge Stations, annual mean saturation values are in excess of 106 percent. The lower 95th percentile confidence limit of both summer and winter DO saturation at Grants Pass is over 100 percent. The 95th percentile lower confidence limit of DO concentration at Grants Pass is 10.2 mg/L.

A decreasing downstream trend in DO concentrations and DO saturation levels in the Rogue River is apparent from the plots. The decreasing downstream DO trend is a cumulative effect of all point and non-point sources, which appears to weigh against natural re-aeration rates.

Total oxygen demand is composed of both carbonaceous biochemical oxygen demand (CBOD) and nitrogenous biochemical oxygen demand (NBOD). CBOD and NBOD are exerted by bacteria and algae breaking down the effluent's waste materials for cellular production and metabolism. CBOD and NBOD in the effluent can be estimated using empirical relations found in the *Revised Section 301(h) Technical Support Document* (EPA 1982). NBOD is based on total kjheldal nitrogen (TKN), which is not measured at the Redwood WWTP. In lieu of TKN data, Redwood NBOD loadings are based on a conservative effluent ammonia value of 25 mg/L. The ratio of TKN to ammonia nitrogen (found in the effluent of the LOTT Secondary Treatment Plant in Olympia, Washington), when applied to the Redwood WWTP ammonia estimate, is used to obtain TKN concentrations in the Redwood effluent.

Because initial dilution occurs rapidly (approximately 30 to 60 seconds), BOD exertion (a slow process) is negligible during this period. However, an immediate dissolved oxygen demand (IDOD) may occur due to rapidly oxidized reduced substances (i.e., sulfides to sulphates). The DO concentration following initial dilution can be predicted from Equation VI-7 of the *Revised Section 301(h) of the Technical Support Document* (EPA 1982). This equation with conservative input parameters is presented as Equation 5.9.

$$DO_f = DO_a + \frac{(DO_e - IDOD - DO_a)}{S_a} \qquad \text{(Equation 5.9)}$$

where:

Parameter	Definition	Value	Source
DO_a	Critical ambient dissolved oxygen concentration	10.2 mg/L	95th percentile lowest observed value at Grants Pass (STORET, see Table 5-3)
IDOD	Immediate dissolved oxygen demand	1.0 mg/L	Table VI-7 of 301(h) Technical Support Document (TSD). Assumes advanced primary treatment.
S_a	Initial dilution	28.7	Modeling results for future effluent flows at current mixing zone (100 feet)
DO_e	Effluent DO	4.0 mg/L	Low value estimate for secondary effluent.

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With this conservative combination of effluent DO, ambient DO, and IDOD, the critical 10.2 mg/L ambient DO concentration is reduced after initial dilution to:

9.95 mg/L

This represents a 2.5 percent decrease in DO at the 100-foot authorized mixing zone boundary. With DO saturation at Grants Pass above 100 percent, a 2.5 percent decrease will allow DO saturation above 97 percent to be maintained, thereby meeting the 95 percent standard.

The DO sag at distances farther downstream can be predicted using Equation VI-17 of the 301(h) TSD. This equation, with conservative input variables, is presented as Equation 5.10.

$$DO(t) = DO_a + \frac{DO_f - DO_a}{D_s} - \left[\frac{L_{fc}}{D_s}(1-e^{-k_c t})\right] - \left[\frac{L_{fn}}{D_s}(1-e^{-k_n t})\right] \qquad \text{(Equation 5.10)}$$

where:

Parameter	Definition	Value	Source
DO_f	Dissolved oxygen concentration at completion of initial dilution (100 feet).	9.95 mg/L	Results of Equation 5.9.
D_s	Dilution obtained subsequent to initial dilution.	24.6	Full mixing of 1.1 mgd effluent flow with 1,200 cfs river flow.
L_{fc}	Ultimate CBOD concentration above ambient at completion of initial dilution (100 feet).	1.53 mg/L	$CBOD_{max}$ = 1.46 x BOD_5[30 mg/L (summer NPDES permit limit)]
L_{fn}	NBOD concentration above ambient at completion of initial dilution (100 feet).	4.50 mg/L	$NBOD_{max}$ = 4.57x1.13xNH_3 [25 mg/L]
K_c	Decay rate for CBOD @ 22°C.	0.60/day	Conservative Value from QUAL2E (EPA 1987)
K_n	Decay rate for NBOD @ 22°C.	0.40/day	Conservative Value from QUAL2E (EPA 1987)
t	Travel time, in river.	3 days	From discharge to mouth of Rogue River assuming 2 ft/sec current speed.

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As with Equation 5.9, the least favorable combination of input variables is used. The analysis gives the following DO decreases:

River Mile 50:	0.13 mg/L
Gold Beach (river mouth):	0.19 mg/L

This simplistic analysis neglects re-aeration. With typical re-aeration rates of 3 mg/L to 5 mg/L per day (EPA 1987), re-aeration will negate the DO impacts from the Redwood discharge. Decreases in DO due to BOD exertion are so minimal at future predicted effluent flows that a measurable DO decrease outside of the mixing zone could never be attributed to the Redwood discharge.

5.3.7.2 Temperature

State water quality standards do not allow a measurable increase in receiving water temperature outside of the mixing zone when ambient water temperature is above 58°F. Mean water temperatures at Grants Pass are above 58°F for most of the summer (May-October), as shown in Figure 5-5.

To analyze water quality impacts, it is assumed the maximum difference between effluent and receiving water temperature is 10°F. This is a conservative estimate based on DMRs from similar secondary treatment plants in which effluent temperature is measured (Parametrix, Inc., 1993). For example, when the receiving water temperature is 58°F, the effluent temperature would be 68°F.

The definition of "measurable increase" supported by ODEQ is an increase of 0.25°F at the mixing zone boundary (Brown and Caldwell 1992). With this allowance, a mixing ratio of 40 is required at critical low river flows to assure compliance. This mixing ratio is achieved at current summer effluent flows at the current authorized mixing zone boundary; however, at increased future summer effluent flows, the mixing ratio is not achieved at mixing zones from 100 to 200 feet from the outfall. If the mixing zone size were increased to a distance of 300 feet from the outfall, the temperature standard would be met.

Although any increase in temperature is technically a violation, temperature impacts due to the discharge are expected to be minimal. Natural temperature fluctuations in the Rogue River are expected to mask any apparent increases.

5.3.7.3 pH

Redwood WWTP effluent pH ranges from 6.7 to 7.8 (January 1992 to September 1993), meeting the standard with no dilution. The buffering capacity of the effluent is anticipated to increase river pH when ambient pH is below neutral. This is a benefit when river pH levels are below the standard, as has occurred on occasion (see Table 5-8).

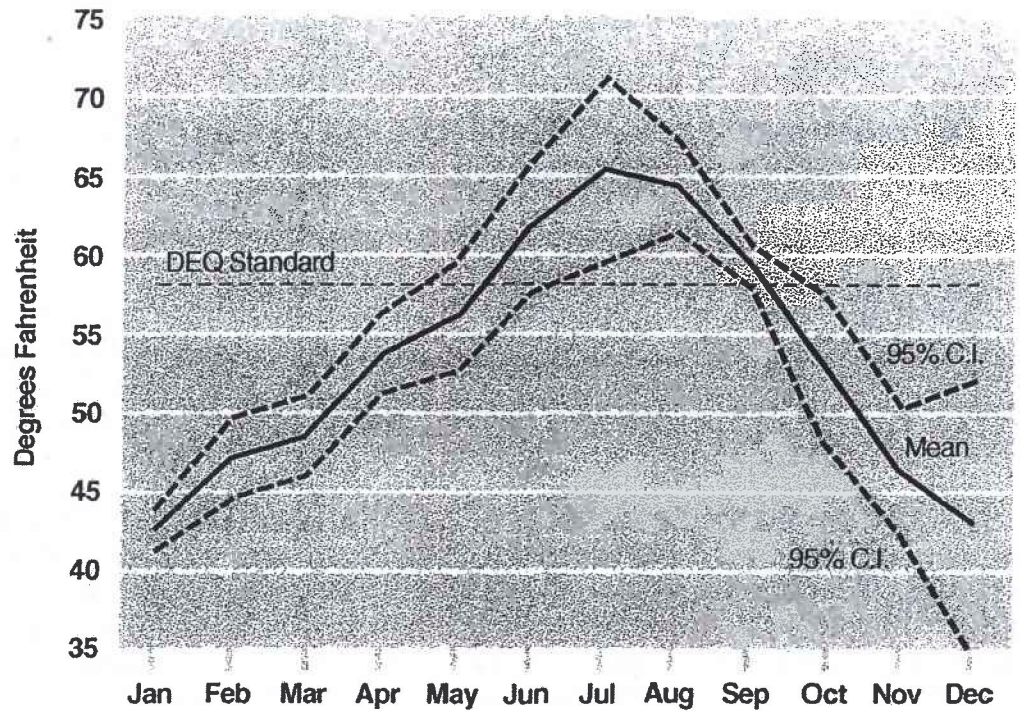

Figure 5-5
Rogue River Temperature
(at Grants Pass)

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5.3.7.4 Total Suspended Solids

The TSS standard of 500 mg/L will be met by virtue of the 30 mg/ℓ summer effluent permit limit.

5.3.7.5 Turbidity

Low river turbidities attest to the high clarity of the Rogue River. Critical 95th percentile ambient turbidity is less than 1.0 NTU. Using the 95th percentile effluent value of 9.7 NTU from the Pullman WWTP (see Section 5.3.6.5), a mixing ratio of 87 would be required to meet the turbidity standard, which allows a 10 percent increase. This mixing ratio will not be met under current or future discharge scenarios during critical summer river flows. At a median river turbidity of 3.0 NTU, the mixing ratio required is lowered to 22.3. This mixing ratio is met at the current mixing zone under existing and future summer discharge conditions.

5.3.7.6 Fecal Coliforms

Median fecal coliform count of the Rogue River near Grants Pass is 91/100 mL. In the past, exceedances above the river coliform standard of 200/100 mL have occurred. Mean effluent concentrations are less than median river concentrations; therefore the Redwood discharge, on average, reduces river coliform concentrations. Even at the highest observed effluent concentration of 109/100 mL, the fecal coliform standard is met. Current NPDES permit limits are adequately protective of water quality.

5.3.7.7 Chlorine

At a 95th percentile highest chlorine residual of 0.80 mg/L and current plant flow, acute and chronic water quality standards are exceeded. The chronic standard could be met at a 200-foot or 300-foot mixing zone. At future predicted effluent flows, the acute and chronic standards could not be met, regardless of mixing zone size.

5.3.7.8 Ammonia

Acute and chronic ammonia water quality standards are currently met with the assumption that effluent ammonia concentrations are similar to the Grants Pass WRP (Brown and Caldwell 1992). Acute and chronic ammonia standards may not be met at future discharge. Prior to assessing compliance at future plant flows, seasonal effluent ammonia measurements are required. Testing frequency is discussed in Section 5.3.8.

5.3.7.9 Cyanide, Phenols, Metals

With the limited effluent and river data available, copper and zinc are of notable concern. Before an adequate compliance assessment can be made, more effluent testing is required. Testing frequencies and other monitoring requirements are discussed in Section 5.3.8.

5.3.8 Recommendations

Based on the receiving water quality criteria analysis, the following treatment and monitoring recommendations can be asserted:

5.3.8.1 Treatment Recommendations

Lower chlorine doses can be applied yet wastewater can still achieve adequate disinfection. At lower effluent residuals, chronic chlorine standards can be met under existing and future effluent flows. However, it is highly unlikely that more efficient chlorination or outfall modifications would improve compliance within the ZID; therefore, dechlorination is recommended.

Ammonia removal should be included in this facilities plan; however, final judgment on whether ammonia removal is necessary should be reserved until seasonal ammonia sampling results are available (see Section 5.3.8.2).

5.3.8.2 Monitoring Recommendations

Water quality analysis and compliance determination depends, to a large degree, on the number of effluent and receiving water samples collected. With larger numbers of samples, statistical uncertainties will be reduced, lowering reasonable potential multipliers and confidence limits. Reducing test detection limits will likewise reduce uncertainty. Monitoring recommendations for parameters showing a potential to exceed water quality standards are presented in Table 5-13. When sample collection is completed, further treatment recommendations can be made, such as the need for outfall modifications, pretreatment, or ammonia removal.

Table 5-13
Monitoring Recommendations

Parameter	Frequency/Duration	Location		Sample Type
		Effluent	River[1]	
Dissolved Oxygen	2/month for one year	X		Grab
Temperature	1/week for one year	X	X	Grab
Turbidity	1/week for one year	X	X	24-hr Composite
Ammonia	1/week for one year	X		24-hr Composite
Bioassay	Quarterly for one year			
Acute		X		
Chronic		X		

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Table 5-13
Monitoring Recommendations

Parameter	Frequency/Duration	Location		Sample Type
		Effluent	River[1]	
Toxicants	Quarterly, using 3			
Cyanide	consecutive days between	X	X	24-hr Composite
Cadmium	Monday and Friday,	X	X	24-hr Composite
Copper	inclusive.	X	X	24-hr Composite
Lead		X	X	24-hr Composite
Mercury		X	X	24-hr Composite
Selenium		X	X	24-hr Composite
Silver		X	X	24-hr Composite
Zinc		X	X	24-hr Composite

(1) Upstream of outfall.

5.4 FUTURE TREATMENT REQUIREMENTS

This section summarizes the requirements, discussed in Sections 5.1 through 5.3, which addressed future wasteload and flow, regulatory treatment criteria, and water quality criteria. In addition, the analysis of the existing treatment plant in Section 4 has shown a need for plant improvements to increase treatment efficiency and to meet critical needs if this plant continues to be used to treat wastewater from the District. The following is a list of requirements for future treatment:

- If the District's wastewater flow is to be treated by either the Redwood WWTP or the City of Grants Pass WRP, either of these facilities should be designed for a maximum monthly dry weather flow (MMDWF) of 1.50 mgd, a maximum monthly wet weather flow (MMWWF) of 2.36 mgd, and a peak instantaneous flow of 4.3 mgd in the year 2020 from the District.

- Further, if the Redwood WWTP is to be used to treat this flow, it should be designed for effluent limitations at the plant for one of two possible situations:

  - Effluent limitations as established by OAR 340-41-375 or as shown in Table 5-4.

  - Effluent limitations as established by OAR 340-41-026 or as shown in Table 5-5.

  An evaluation of the cost and consequences of each of these effluent limitations should be conducted.

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Also, if the Redwood WWTP is to be used to treat flow from the District, the plant should be upgraded to include the following:

- Significantly reduce or remove chlorine residual in effluent at all times.
- Provide for partial ammonia removal in the treatment process as part of any plant modifications.
- Provide automatic composite sampling equipment to assist making data consistent and reliable.
- Provide variable speed controls on the influent pumps to minimize the impacts of this pump station on the treatment abilities of the plant.
- Add blower capacity to maintain adequate oxygen levels in the aeration basin during peak loadings.

To meet these requirements, a number of alternatives have been developed and analyzed. These alternatives are presented in Section 6.

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5.3.5.4 Far-Field Model Calibration

The far-field model was calibrated using the results of the *City of Grants Pass Mixing Zone Study* (Brown and Caldwell 1992). In this mixing zone study, a fluorescent dye tracing study was used to measure lateral and vertical mixing of the Grants Pass discharge downstream of the outfall. The Grants Pass outfall discharges near the north bank of the Rogue River at river mile 100.9.

Each river has particular mixing characteristics, that depend on channel characteristics such as roughness, current speed, tortuosity, and depth. Plume width measurements at 250 and 1,000 feet downstream of the Grants Pass discharge allowed calculation, using Equation 5.8, of the lateral turbulent diffusion coefficient (ϵ_t) for the Rogue River near Grants Pass.

The Grants Pass study results were used to determine u^* (shear velocity), and Equation 5.4 was used to extrapolate ϵ_t for application near the Redwood WWTP. The lateral turbulent diffusion coefficient for the Rogue River near the Redwood discharge—under regulatory, low-flow conditions—is estimated as 0.22 ft^2/sec. The vertical turbulent diffusion coefficient is estimated as 0.024 ft^2/sec.

5.3.5.5 Model Input

Near- and far-field model input parameters are summarized in Table 5-9. Two statistical measures of effluent flows were used in the modeling analysis. Acute mixing ratios are calculated at peak day effluent flows to correspond with the short duration exposures leading to acute toxicity. Acute criteria are evaluated at the ZID. Chronic mixing ratios are calculated at peak month flows to correspond with the longer duration exposures associated with chronic toxicity. Chronic criteria are evaluated at the mixing zone boundary. Effluent flow projections are discussed in more detail in Section 5.1.

5.3.5.6 Model Results

Table 5-10 summarizes near- and far-field model results. The dilution ratio is the inverse of percent effluent. Results are given for both the current authorized mixing zone boundaries as well as at distances which may be considered for mixing zone expansion. Complete model results appear in Appendix F.

Table 5-9
Near- and Far-Field Model Input Parameters

Parameter	Source	Near-Field Model (PLUMES)	Far-Field Model (Advective Dispersion Equation)	Comment
PLANT				
Existing Flow	Oct. 1992-Sept. 1993 Daily Monitoring Reports (DMRs)	0.54 mgd (summer peak day)	0.48 mgd (summer peak month)	
Future Flow	Projected (see Section 5.1)	1.1 mgd (summer peak day)	1.0 mgd (summer peak month)	
Effluent Temperature	Estimate	21°C	N/A	Mixing not sensitive to this parameter
Effluent Salinity	Estimate	0	N/A	Mixing not sensitive to this parameter
OUTFALL				
Port Diameter	As-built Drawings (see Appendix F)	18"	18"	
Vertical Angle	As-built Drawings (see Appendix F)	0°	0°	
Horizontal Angle	As-built Drawings (see Appendix F)	Co-Flowing	Co-Flowing	Actual angle 45° with respect to currents
Contraction Coefficient	Estimate	0.95	N/A	
Port Depth	November 30, 1993 Survey (Gary Wicks Engineering)	1.85 ft	Mid Level	To center of port
Port Elevation	November 30, 1993 Survey (Gary Wicks Engineering)	1.3 ft	Mid Level	To center of port
RECEIVING WATERS				
River Flow	USGS, 1990	1,200 cfs	1,200 cfs	Regulatory low flow
Average Current	Equation 5.1	1.25 ft/s	1.25 ft/s	Regulatory low flow
River Depth	November 30, 1993 Survey (Gary Wicks Engineering)	2.8 ft (at outfall)	2.4 ft (average)	Regulatory low flow

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Table 5-10
Mixing Model Results

Mixing Criteria	Distance (ft)	Percent Effluent		Dilution Ratio		Plume Width (ft)
		Existing[1] (%)	Future[2] (%)	Existing	Future	
Acute	10	24	50	4.2	2.0	7.5
	20	15	36	6.5	2.8	10.6
	30	12	29	8.5	3.4	13.0
Chronic	100	1.7	3.5	59.8	28.7	23.7
	200	1.2	2.6	80.2	38.5	33.6
	300	1.1	2.3	90.4	43.4	41.1

(1) At existing summer plant flows (May-November).
(2) At future summer plant flows (May-November).

5.3.6 Effluent Quality

5.3.6.1 Chlorine

Chlorine residual statistical variability was determined from Discharge Monitoring Reports (DMRs) for the period October 1992 to September 1993. The following results were obtained:

Number of Samples: 365

Low:	0.0 mg/L
High:	1.1 mg/L
Mean:	0.44 mg/L

5th Percentile:	0.10 mg/L
50th Percentile:	0.40 mg/L
95th Percentile:	0.80 mg/L

5.3.6.2 Fecal Coliforms

Fecal coliform samples are collected once each week. DMRs for the period January 1992 to September 1993 show that permit limits are consistently met:

Number of Samples: 156

Low:	0/100 mL
High:	109/100 mL
Weekly Average:	6/100 mL
Monthly Average:	7/100 mL

These results would suggest that the chlorine dose could be reduced and still meet permit limits.

5.3.6.3 Biochemical Oxygen Demand (BOD)

Summer and winter BOD concentrations for the past six-year period are shown in Appendix C. Maximum summer month BOD was 25 mg/ℓ (October 1992).

5.3.6.4 Total Suspended Solids (TSS)

Summer and winter TSS concentrations for the past six-year period are shown in Appendix C. Maximum summer month TSS was 20 mg/ℓ (June 1991).

5.3.6.5 Turbidity and Suspended Solids

The Redwood WWTP does not sample for turbidity; however, results from the Pullman WWTP in Pullman, Washington, can be used to represent typical secondary effluent values. The Pullman WWTP uses a process similar to the Redwood WWTP; however, turbidity values for the Redwood WWTP may be less than for Pullman, which has a higher summer suspended solids effluent limitation of 30 mg/L monthly and 45 mg/L weekly.

Turbidity of the Pullman effluent was measured three times daily from January 1993 to March 1993. Results of sampling are given below:

Average ± Standard Deviation:	3.62 ± 3.04
Maximum:	18.6
Minimum:	0.5
95th Percentile:	9.7

5.3.6.6 Temperature

High effluent temperatures are critical for receiving water impacts. At high river temperatures, dissolved oxygen (DO) concentrations are lowered, which can adversely affect aquatic life. Effluent temperature is not measured at the Redwood WWTP; however, records from the Grants Pass WRP show effluent temperatures there may reach as high as 76°F. Similar summer effluent temperatures are expected at the Redwood WWTP and will likely periodically exceed 70°F (Brown and Caldwell 1992). Effluent temperature monitoring at the Redwood WWTP should be initiated.

5.3.6.7 Ammonia, Metals, Cyanide, Phenols

A grab effluent sample was analyzed for ammonia, metals, cyanide, and phenols in December 1993 by Nielson Laboratories, Medford, Oregon. Toxicants including pesticides, PCBs, and volatile organic chemicals (VOCs) were not tested in the effluent, because they were not likely to pose a concern for the primarily municipal influent.

EPA guidance from the *Technical Support Document for Water Quality Based Toxics Control* (USEPA 1991) requires use of a reasonable potential multiplier to account for effluent variability

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when less than 20 effluent samples have been measured. The multiplier used for evaluating water quality standards compliance will be based on a 95th percentile probability basis and 95th percentile distribution. A coefficient of variation of 0.6 is assumed unless data are available for its calculation.

No ammonia was detected in the Redwood sample. Typical secondary effluent ammonia concentrations are highly variable and depend in part on effluent temperature, pH, hydraulic detention time in the plant, and wastewater composition and strength. Each of these parameters can vary seasonally.

To better represent ammonia variability in the Redwood effluent, testing results for the Grants Pass WRP will be used in lieu of the single Redwood sample. Results of limited ammonia testing in June 1991 are presented below (Brown and Caldwell 1992):

Number of Samples:	5
Low:	9.5 mg/L
High:	12.3 mg/L
Average:	11.1 mg/L

5.3.6.8 pH

Redwood's NPDES permit stipulates that effluent pH be in the range 6.0 to 9.0. Effluent pH is measured three times weekly and ranges from 6.7 to 7.8 (January 1992-September 1993).

5.3.7 Water Quality Analysis

In this section, the ability of the existing and future Redwood discharge to comply with water quality standards is evaluated. Tables 5-11 and 5-12 summarize the water quality analysis and compliance determination for each parameter analyzed. Information provided in Table 5-12 includes:

- Ambient concentrations from STORET data (see Table 5-3).
- Effluent concentrations from DMRs, laboratory effluent scan (see Appendices B and D) or are estimated based on typical secondary treatment plant performance (i.e., Pullman WWTP and Grants Pass WRP).
- Laboratory detection limits, when not detected.
- Number of samples analyzed.
- Reasonable potential multipliers for toxicants with less than 20 representative samples (USEPA 1991).
- Freshwater acute and chronic water quality standards from Table 20, OAR-340-41 and from numeric criteria in OAR 340-41-365.
- Mixing ratios required to meet water quality standards.

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Table 5-11
Water Quality Analysis Summary

Parameter	Units	Critical Ambient Concentration	Effluent			Water Quality Standard OAR 340-41		Mixing Ratio Required	
			Critical Concentration	Number of Samples	RPM	Acute	Chronic	Acute (ZID)	Chronic (Mixing Zone)
CONVENTIONAL									
Dissolved Oxygen	mg/L	10.2	4.0	NA	1	—	95% saturation	—	13.4
Temperature	°F	58°	68°	NA	1	—	<0.25 decrease	—	40
pH	SU	6.3-8.2	6.7-7.8	252	1	—	6.5-8.5	—	1
Total Suspended Solids	mg/L	NA	(4) 30	365	1	—	500	—	1
Turbidity	NTU	1.0	9.7	NA	1	—	<10% increase	—	87
Fecal Coliforms	MPN/100 mL	940	109	156	1	—	200	—	1
TOXICANTS									
Chlorine	mg/L	0	(3) 0.80	365	1	0.019	0.011	42.1	72.7
Ammonia	mg/L as N	0.14	(5) 12.3	1	1	(1) 3.3	(1) 0.45	3.8	39.2
Cyanide	µg/L	NA	<20	1	6.2	22	5.2	5.6	23.8
Phenols	µg/L	NA	<50	1	6.2	10,200	2,560	1	1
Arsenic	µg/L	<5	<1	1	6.2	360	190	1	1
Cadmium	µg/L	<1	0.2	1	6.2	(2) 1.05	(2) 0.45	4.8	(6) 1
Chromium (III)	µg/L	<2	<50	1	6.2	(2) 665	(2) 79.3	1	4.0
Copper	µg/L	3	24	1	6.2	(2) 5.9	(2) 4.4	50.3	10.4
Lead	µg/L	<10	<1	1	6.2	(2) 18.4	(2) 0.72	1	(6) 8.6
Mercury	µg/L	<0.5	<0.2	1	6.2	2.4	0.012	1	(6) 103
Nickel	µg/L	NA	<40	1	6.2	(2) 527	(2) 58.5	1	4.2
Selenium	µg/L	<5	<2	1	6.2	20.0	5.0	1	(6) 2.5
Silver	µg/L	<1	<10	1	6.2	(2) 0.54	0.12	115	517
Zinc	µg/L	40	85	1	6.2	(2) 43.4	(2) 39.3	143	(7) —

(1) Criteria based on *Ambient Aquatic Life Water Quality Criteria for Ammonia* (USEPA 1989) using highest observed pH and temperature at Grants Pass (pH=8.2, Temp.=22.0°C).

(2) Criteria hardness dependent (95th percentile value of 31 mg/L as $CaCO_3$ critical).

(3) 95% percentile value from daily measurement.

(4) Permit limitation.

(5) Based on maximum of 5 measurements at Grants Pass Water Pollution Control Facility (6/13/91 - 6/28/91)

(6) Background assumed zero.

(7) Background is standard.

NA - Not Available

RPM - Reasonable Potential Multiplier

Table 5-12
Compliance Summary

Parameter	Scenario[1]							
	Current Flow				Future Flow			
	100′ Radius		300′ Radius		100′ Radius		300′ Radius	
	Acute	Chronic	Acute	Chronic	Acute	Chronic	Acute	Chronic
CONVENTIONAL								
Dissolved Oxygen	—	Yes	—	Yes	—	Yes	—	Yes
Temperature	—	Yes	—	Yes	—	No	—	Yes
pH	—	Yes	—	Yes	—	Yes	—	Yes
TSS	—	Yes	—	Yes	—	Yes	—	Yes
Turbidity	—	No	—	Yes	—	No	—	No
Fecal Coliforms	—	Yes	—	Yes	—	Yes	—	Yes
TOXICANTS[2]								
Chlorine	No	No	No	Yes	No	No	No	No
Ammonia	Yes	Yes	Yes	Yes	No	No	No	Yes
Cyanide	LDN	Yes	Yes	Yes	LDN	Yes	LDN	Yes
Phenols	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Arsenic	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Cadmium	LDN	Yes	Yes	Yes	LDN	Yes	LDN	Yes
Chromium (III)	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Copper	No	No	No	No	No	No	No	No
Lead	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Mercury	Yes	LDN	Yes	LDN	Yes	LDN	Yes	LDN
Nickel	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Selenium	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Silver	LDN	LDN	LDN	LDN	LDN	LDN	LDN	LDN
Zinc	No	No	No	No	No	No	No	No

(1) All scenarios assume current plant effluent quality.
(2) Compliance based on detection limits when not detected.
LDN – Lower effluent detection limits and more effluent samples needed to assess compliance.

Table 5-11 includes a listing of each parameter and determination of compliance with water quality standards at each combination of existing and future predicted summer plant flows and current and expanded mixing zone boundaries (100 feet to 300 feet). Compliance is discussed for each parameter in the following sections.

5.3.7.1 Dissolved Oxygen

Salmonid species are present in the Rogue River and require that 95 percent DO saturation be maintained. Figure 5-4 shows mean values and 95th percentile confidence limits of DO concentrations and DO saturation for the Rogue River at all STORET ambient monitoring stations. Values are also shown separately by month for the Grants Pass station (Brown and Caldwell 1992).

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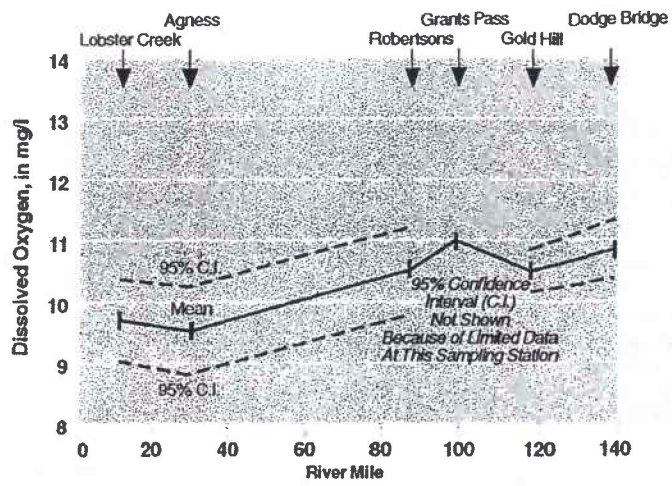
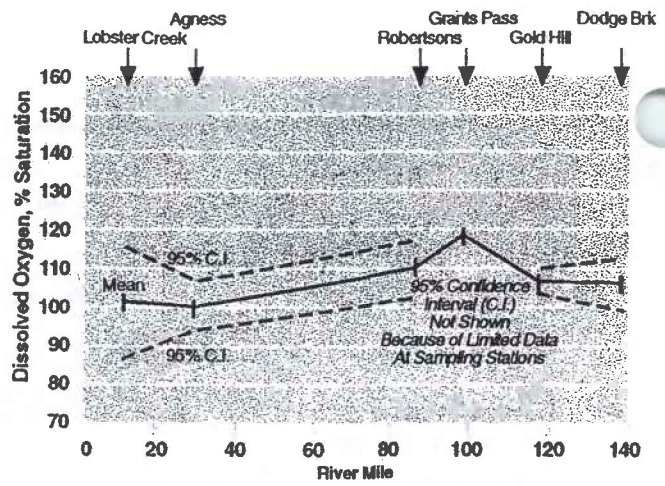
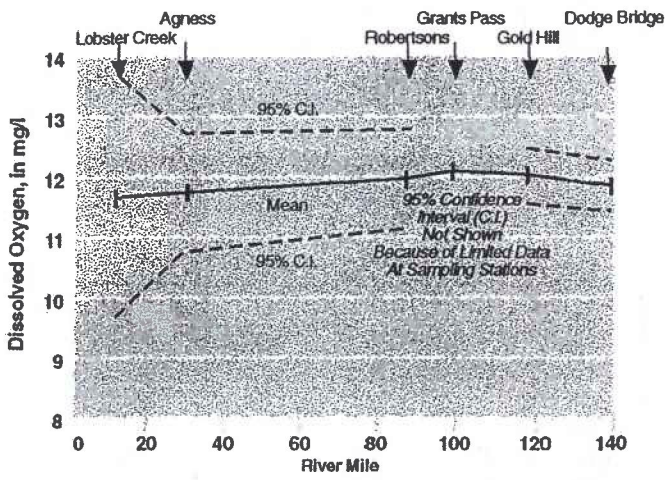


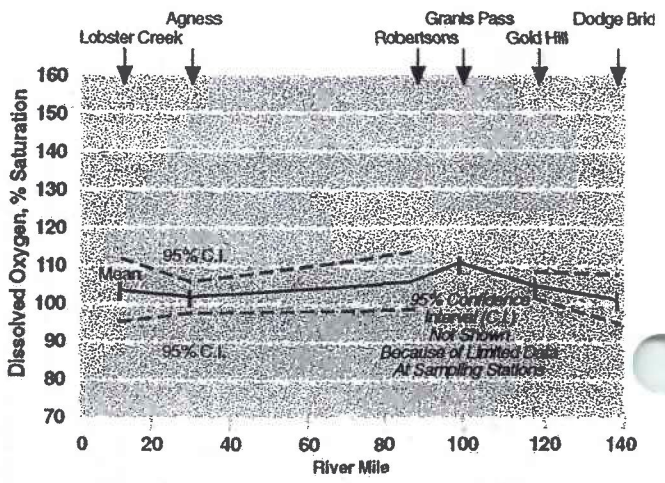
SUMMER DISSOLVED OXYGEN CONCENTRATION



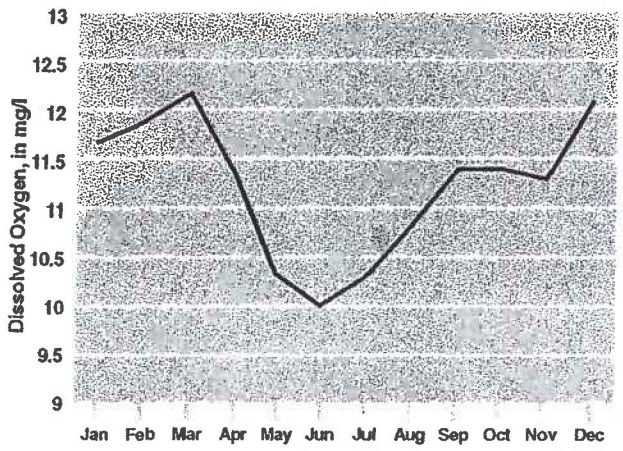
SUMMER DISSOLVED OXYGEN SATURATION



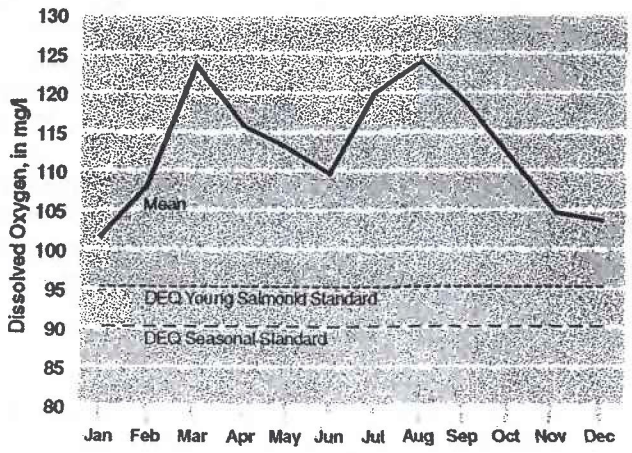
WINTER DISSOLVED OXYGEN CONCENTRATION



WINTER DISSOLVED OXYGEN SATURATION



MINIMUM DISSOLVED OXYGEN CONCENTRATIONS AT GRANTS PASS



DISSOLVED OXYGEN SATURATION AT GRANTS PASS

Redwood Wastewater Treatment Plant
#27-2192-05 02/99

Source: Reproduced from Water Restoration Plant Facilities Plan Draft, City of Grants Pass (Brown and Caldwell, 1992)

Figure 5-4
Rogue River
Dissolved Oxygen

Average DO saturation values exceed 95 percent in both summer and winter for the entire river length. At Grants Pass and Robertson Bridge Stations, annual mean saturation values are in excess of 106 percent. The lower 95th percentile confidence limit of both summer and winter DO saturation at Grants Pass is over 100 percent. The 95th percentile lower confidence limit of DO concentration at Grants Pass is 10.2 mg/L.

A decreasing downstream trend in DO concentrations and DO saturation levels in the Rogue River is apparent from the plots. The decreasing downstream DO trend is a cumulative effect of all point and non-point sources, which appears to weigh against natural re-aeration rates.

Total oxygen demand is composed of both carbonaceous biochemical oxygen demand (CBOD) and nitrogenous biochemical oxygen demand (NBOD). CBOD and NBOD are exerted by bacteria and algae breaking down the effluent's waste materials for cellular production and metabolism. CBOD and NBOD in the effluent can be estimated using empirical relations found in the *Revised Section 301(h) Technical Support Document* (EPA 1982). NBOD is based on total kjheldal nitrogen (TKN), which is not measured at the Redwood WWTP. In lieu of TKN data, Redwood NBOD loadings are based on a conservative effluent ammonia value of 25 mg/L. The ratio of TKN to ammonia nitrogen (found in the effluent of the LOTT Secondary Treatment Plant in Olympia, Washington), when applied to the Redwood WWTP ammonia estimate, is used to obtain TKN concentrations in the Redwood effluent.

Because initial dilution occurs rapidly (approximately 30 to 60 seconds), BOD exertion (a slow process) is negligible during this period. However, an immediate dissolved oxygen demand (IDOD) may occur due to rapidly oxidized reduced substances (i.e., sulfides to sulphates). The DO concentration following initial dilution can be predicted from Equation VI-7 of the *Revised Section 301(h) of the Technical Support Document* (EPA 1982). This equation with conservative input parameters is presented as Equation 5.9.

$$DO_f = DO_a + \frac{(DO_e - IDOD - DO_a)}{S_a} \qquad \text{(Equation 5.9)}$$

where:

Parameter	Definition	Value	Source
DO_a	Critical ambient dissolved oxygen concentration	10.2 mg/L	95th percentile lowest observed value at Grants Pass (STORET, see Table 5-3)
IDOD	Immediate dissolved oxygen demand	1.0 mg/L	Table VI-7 of 301(h) Technical Support Document (TSD). Assumes advanced primary treatment.
S_a	Initial dilution	28.7	Modeling results for future effluent flows at current mixing zone (100 feet)
DO_e	Effluent DO	4.0 mg/L	Low value estimate for secondary effluent.

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With this conservative combination of effluent DO, ambient DO, and IDOD, the critical 10.2 mg/L ambient DO concentration is reduced after initial dilution to:

9.95 mg/L

This represents a 2.5 percent decrease in DO at the 100-foot authorized mixing zone boundary. With DO saturation at Grants Pass above 100 percent, a 2.5 percent decrease will allow DO saturation above 97 percent to be maintained, thereby meeting the 95 percent standard.

The DO sag at distances farther downstream can be predicted using Equation VI-17 of the 301(h) TSD. This equation, with conservative input variables, is presented as Equation 5.10.

$$DO(t) = DO_a + \frac{DO_f - DO_a}{D_s} - \left[\frac{L_{fc}}{D_s}(1-e^{-k_c t})\right] - \left[\frac{L_{fn}}{D_s}(1-e^{-k_n t})\right] \qquad \text{(Equation 5.10)}$$

where:

Parameter	Definition	Value	Source
DO_f	Dissolved oxygen concentration at completion of initial dilution (100 feet).	9.95 mg/L	Results of Equation 5.9.
D_s	Dilution obtained subsequent to initial dilution.	24.6	Full mixing of 1.1 mgd effluent flow with 1,200 cfs river flow.
L_{fc}	Ultimate CBOD concentration above ambient at completion of initial dilution (100 feet).	1.53 mg/L	$CBOD_{max}$ = 1.46 x BOD_5[30 mg/L (summer NPDES permit limit)]
L_{fn}	NBOD concentration above ambient at completion of initial dilution (100 feet).	4.50 mg/L	$NBOD_{max}$ = 4.57x1.13xNH_3 [25 mg/L]
K_c	Decay rate for CBOD @ 22°C.	0.60/day	Conservative Value from QUAL2E (EPA 1987)
K_n	Decay rate for NBOD @ 22°C.	0.40/day	Conservative Value from QUAL2E (EPA 1987)
t	Travel time, in river.	3 days	From discharge to mouth of Rogue River assuming 2 ft/sec current speed.

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As with Equation 5.9, the least favorable combination of input variables is used. The analysis gives the following DO decreases:

River Mile 50:	0.13 mg/L
Gold Beach (river mouth):	0.19 mg/L

This simplistic analysis neglects re-aeration. With typical re-aeration rates of 3 mg/L to 5 mg/L per day (EPA 1987), re-aeration will negate the DO impacts from the Redwood discharge. Decreases in DO due to BOD exertion are so minimal at future predicted effluent flows that a measurable DO decrease outside of the mixing zone could never be attributed to the Redwood discharge.

5.3.7.2 Temperature

State water quality standards do not allow a measurable increase in receiving water temperature outside of the mixing zone when ambient water temperature is above 58°F. Mean water temperatures at Grants Pass are above 58°F for most of the summer (May-October), as shown in Figure 5-5.

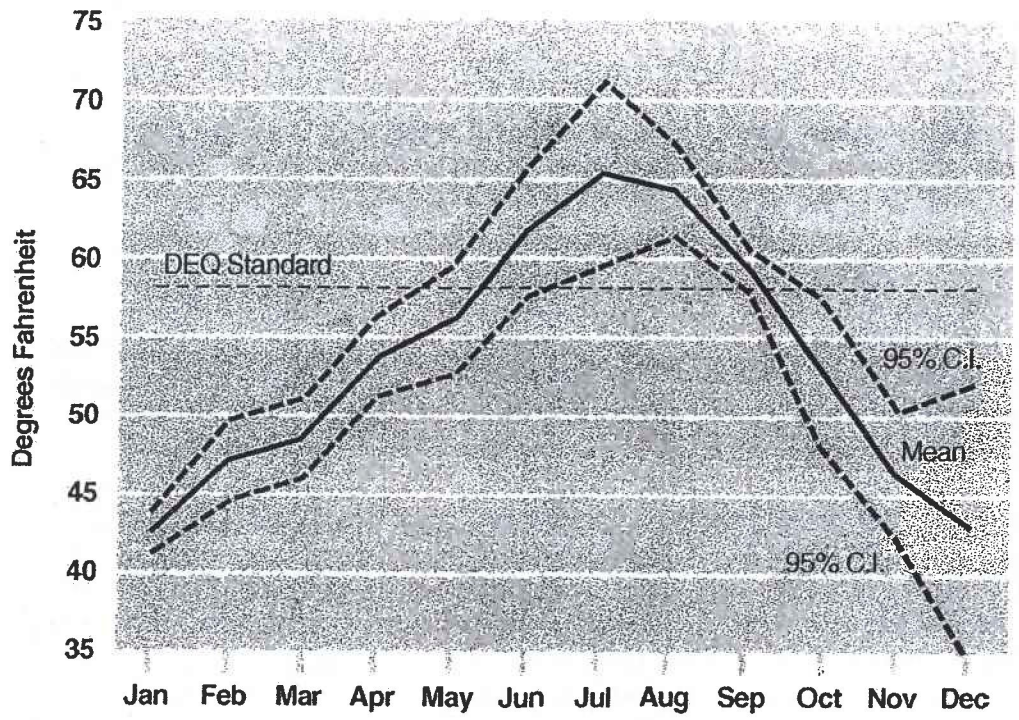
To analyze water quality impacts, it is assumed the maximum difference between effluent and receiving water temperature is 10°F. This is a conservative estimate based on DMRs from similar secondary treatment plants in which effluent temperature is measured (Parametrix, Inc., 1993). For example, when the receiving water temperature is 58°F, the effluent temperature would be 68°F.

The definition of "measurable increase" supported by ODEQ is an increase of 0.25°F at the mixing zone boundary (Brown and Caldwell 1992). With this allowance, a mixing ratio of 40 is required at critical low river flows to assure compliance. This mixing ratio is achieved at current summer effluent flows at the current authorized mixing zone boundary; however, at increased future summer effluent flows, the mixing ratio is not achieved at mixing zones from 100 to 200 feet from the outfall. If the mixing zone size were increased to a distance of 300 feet from the outfall, the temperature standard would be met.

Although any increase in temperature is technically a violation, temperature impacts due to the discharge are expected to be minimal. Natural temperature fluctuations in the Rogue River are expected to mask any apparent increases.

5.3.7.3 pH

Redwood WWTP effluent pH ranges from 6.7 to 7.8 (January 1992 to September 1993), meeting the standard with no dilution. The buffering capacity of the effluent is anticipated to increase river pH when ambient pH is below neutral. This is a benefit when river pH levels are below the standard, as has occurred on occasion (see Table 5-8).



Redwood Wastewater Treatment Plant
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Figure 5-5
Rogue River Temperature
(at Grants Pass)

5.3.7.4 Total Suspended Solids

The TSS standard of 500 mg/L will be met by virtue of the 30 mg/ℓ summer effluent permit limit.

5.3.7.5 Turbidity

Low river turbidities attest to the high clarity of the Rogue River. Critical 95th percentile ambient turbidity is less than 1.0 NTU. Using the 95th percentile effluent value of 9.7 NTU from the Pullman WWTP (see Section 5.3.6.5), a mixing ratio of 87 would be required to meet the turbidity standard, which allows a 10 percent increase. This mixing ratio will not be met under current or future discharge scenarios during critical summer river flows. At a median river turbidity of 3.0 NTU, the mixing ratio required is lowered to 22.3. This mixing ratio is met at the current mixing zone under existing and future summer discharge conditions.

5.3.7.6 Fecal Coliforms

Median fecal coliform count of the Rogue River near Grants Pass is 91/100 mL. In the past, exceedances above the river coliform standard of 200/100 mL have occurred. Mean effluent concentrations are less than median river concentrations; therefore the Redwood discharge, on average, reduces river coliform concentrations. Even at the highest observed effluent concentration of 109/100 mL, the fecal coliform standard is met. Current NPDES permit limits are adequately protective of water quality.

5.3.7.7 Chlorine

At a 95th percentile highest chlorine residual of 0.80 mg/L and current plant flow, acute and chronic water quality standards are exceeded. The chronic standard could be met at a 200-foot or 300-foot mixing zone. At future predicted effluent flows, the acute and chronic standards could not be met, regardless of mixing zone size.

5.3.7.8 Ammonia

Acute and chronic ammonia water quality standards are currently met with the assumption that effluent ammonia concentrations are similar to the Grants Pass WRP (Brown and Caldwell 1992). Acute and chronic ammonia standards may not be met at future discharge. Prior to assessing compliance at future plant flows, seasonal effluent ammonia measurements are required. Testing frequency is discussed in Section 5.3.8.

5.3.7.9 Cyanide, Phenols, Metals

With the limited effluent and river data available, copper and zinc are of notable concern. Before an adequate compliance assessment can be made, more effluent testing is required. Testing frequencies and other monitoring requirements are discussed in Section 5.3.8.

5.3.8 Recommendations

Based on the receiving water quality criteria analysis, the following treatment and monitoring recommendations can be asserted:

5.3.8.1 Treatment Recommendations

Lower chlorine doses can be applied yet wastewater can still achieve adequate disinfection. At lower effluent residuals, chronic chlorine standards can be met under existing and future effluent flows. However, it is highly unlikely that more efficient chlorination or outfall modifications would improve compliance within the ZID; therefore, dechlorination is recommended.

Ammonia removal should be included in this facilities plan; however, final judgment on whether ammonia removal is necessary should be reserved until seasonal ammonia sampling results are available (see Section 5.3.8.2).

5.3.8.2 Monitoring Recommendations

Water quality analysis and compliance determination depends, to a large degree, on the number of effluent and receiving water samples collected. With larger numbers of samples, statistical uncertainties will be reduced, lowering reasonable potential multipliers and confidence limits. Reducing test detection limits will likewise reduce uncertainty. Monitoring recommendations for parameters showing a potential to exceed water quality standards are presented in Table 5-13. When sample collection is completed, further treatment recommendations can be made, such as the need for outfall modifications, pretreatment, or ammonia removal.

Table 5-13
Monitoring Recommendations

Parameter	Frequency/Duration	Location		Sample Type
		Effluent	River[1]	
Dissolved Oxygen	2/month for one year	X		Grab
Temperature	1/week for one year	X	X	Grab
Turbidity	1/week for one year	X	X	24-hr Composite
Ammonia	1/week for one year	X		24-hr Composite
Bioassay	Quarterly for one year			
Acute		X		
Chronic		X		

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Table 5-13
Monitoring Recommendations

Parameter	Frequency/Duration	Location		Sample Type
		Effluent	River[1]	
Toxicants	Quarterly, using 3			
Cyanide	consecutive days between	X	X	24-hr Composite
Cadmium	Monday and Friday,	X	X	24-hr Composite
Copper	inclusive.	X	X	24-hr Composite
Lead		X	X	24-hr Composite
Mercury		X	X	24-hr Composite
Selenium		X	X	24-hr Composite
Silver		X	X	24-hr Composite
Zinc		X	X	24-hr Composite

(1) Upstream of outfall.

5.4 FUTURE TREATMENT REQUIREMENTS

This section summarizes the requirements, discussed in Sections 5.1 through 5.3, which addressed future wasteload and flow, regulatory treatment criteria, and water quality criteria. In addition, the analysis of the existing treatment plant in Section 4 has shown a need for plant improvements to increase treatment efficiency and to meet critical needs if this plant continues to be used to treat wastewater from the District. The following is a list of requirements for future treatment:

- If the District's wastewater flow is to be treated by either the Redwood WWTP or the City of Grants Pass WRP, either of these facilities should be designed for a maximum monthly dry weather flow (MMDWF) of 1.50 mgd, a maximum monthly wet weather flow (MMWWF) of 2.36 mgd, and a peak instantaneous flow of 4.3 mgd in the year 2020 from the District.

- Further, if the Redwood WWTP is to be used to treat this flow, it should be designed for effluent limitations at the plant for one of two possible situations:

  - Effluent limitations as established by OAR 340-41-375 or as shown in Table 5-4.

  - Effluent limitations as established by OAR 340-41-026 or as shown in Table 5-5.

  An evaluation of the cost and consequences of each of these effluent limitations should be conducted.

Also, if the Redwood WWTP is to be used to treat flow from the District, the plant should be upgraded to include the following:

- Significantly reduce or remove chlorine residual in effluent at all times.
- Provide for partial ammonia removal in the treatment process as part of any plant modifications.
- Provide automatic composite sampling equipment to assist making data consistent and reliable.
- Provide variable speed controls on the influent pumps to minimize the impacts of this pump station on the treatment abilities of the plant.
- Add blower capacity to maintain adequate oxygen levels in the aeration basin during peak loadings.

To meet these requirements, a number of alternatives have been developed and analyzed. These alternatives are presented in Section 6.

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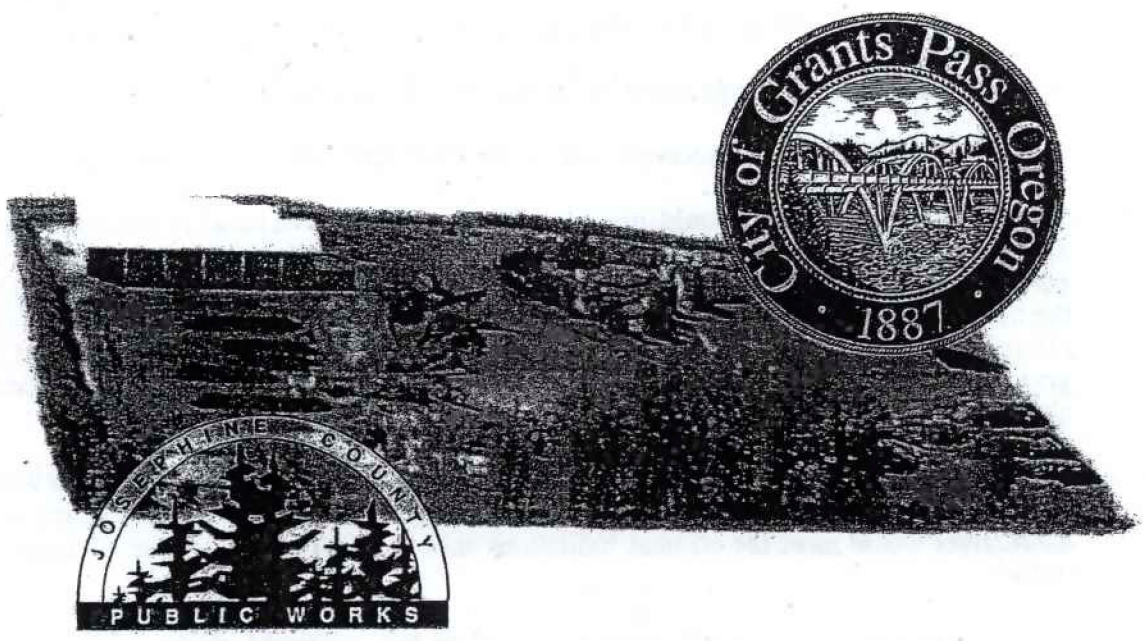
Treatment Alternatives

Redwood Sanitary Sewer Service District

Josephine County, Oregon



Parametrix, Inc.

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6. TREATMENT ALTERNATIVES

6.1 ALTERNATIVES INVESTIGATED

To evaluate the best method for meeting the District's wastewater treatment needs to the year 2020, twelve alternatives were analyzed. These alternatives were separated into three groups, the first and second of which use the Redwood WWTP for treatment and are considered treatment alternatives. The third group uses the City of Grants Pass WRP for treatment and are considered conveyance alternatives.

- Alternatives 1, 2, and 3 would provide secondary treatment at the existing Redwood WWTP with some ammonia removal. These alternatives would meet the effluent limitations shown on Table 5-4. These alternatives include:

  - ► Construct a new contact-stabilization activated sludge system.
  - ► Construct a new Sequencing Batch Reactor (SBR) treatment system.
  - ► Modify the existing process to a trickling filter/activated sludge treatment system.

  Each of these alternatives would provide additional organic removal as well as partial ammonia removal at the plant but would require ODEQ approval of an increase in permitted mass discharges. The alternatives were investigated because they appear to be the most cost-effective and reasonable ways to expand the plant and they would meet the effluent limitations established by OAR 340-41-375; however, ODEQ would need to grant a special effluent discharge load limit to the Rogue river for any of these alternatives to be implemented.

- Alternatives 4 and 5 would provide tertiary treatment at the existing Redwood WWTP and would not require a special discharge load limit to the Rogue River. These two alternatives would meet the effluent limitations shown in Table 5-5. These alternatives include:

  - ► Construct a new anoxic selector complete mix activated sludge system with effluent filtration.
  - ► Construct a new sequencing batch reactor (SBR) treatment system with effluent filtration.

  Both of these alternatives would also provide additional organic removal as well as significant ammonia removal at the plant. They would not require ODEQ approval of permitted mass discharge increases and they would meet the effluent limitations established by OAR 340-41-026.

- Alternatives 6 through 12 provide pumping and transmission facilities to convey all of the wastewater to the Grants Pass WRP for treatment.

  This group of alternatives evaluates conveying wastewater to the Grants Pass WRP instead of upgrading the Redwood WWTP. The Grants Pass WRP has adequate treatment capacity available. Each of the six conveyance alternatives considers a different sewer force-main route between the existing Redwood WWTP and the Grants Pass WRP. A more detailed description of these conveyance alternatives is presented later in Subsection 6.5.

6.2 ALTERNATIVES 1, 2, AND 3

Preliminary sizing criteria, for each of the three alternatives investigated that meet the effluent limitations established by OAR 340-41-375 (shown in Table 5-4), are presented in Table 6-1. A short description of each of these three alternatives follows.

Table 6-1
Preliminary Sizing Criteria, New Unit Treatment Processes
Treatment Plant Expansion Alternatives Meeting OAR 340-41-375

Unit Process	Size Criteria
Alternative 1 – Contact Stabilization with Anoxic Selectors	
Anoxic Selector Basins	
No. of Trains	2
No. of Cells	3/Train
Total Volume	10,200 ft^3
Contact Basins	
No. of Trains	2
Total Volume	26,400 ft^3
Depth	15 ft
Stabilization Basins	
No. of Trains	2
Total Volume	26,400 ft^3
Depth	15 ft
Secondary Clarifier[1]	
Diameter	60 ft
Depth	13 ft
Surface Area (effective)	2,714 ft^2
Anaerobic Digesters	
Volume (total of 2)	17,000 ft^3
Diameter	25' dia. – 20'

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Table 6-1
Preliminary Sizing Criteria, New Unit Treatment Processes
Treatment Plant Expansion Alternatives Meeting OAR 340-41-375

Unit Process	Size Criteria
Alternative 2 – SBR (Sequencing Batch Reactor)	
Sequencing Batch Reactor	
Number	2
Dimensions	75′ x 65′ x 17′ SWD
Cycles per Day	5 – summer, 6 – winter
Anaerobic Digesters	
Volume (total of 2)	17,000 ft^3
Dimensions	25′ dia. – 20′ SWD
Alternative 3 – TF/AS (Trickling Filter/Activated Sludge)	
Primary Sedimentation Basins	
Volume	21,000 ft^3
Depth	11 ft
Biofilters	
Number	2
Media Depth	15 ft
Diameter	40 ft
Secondary Clarifier(1)	
Diameter	60 ft
Surface Area (effective)	2,714 ft^2
Anaerobic Digesters	
Volume (total of 2)	17,000 ft^3
Diameter	25′ dia. – 20′ SWD

(1) In Alternatives 1 and 3, the existing secondary clarifier also remains in service. A polymer addition system would be provided to improve clarifier performance if the effluent suspended solids limits were not being met.

A new contact stabilization activated sludge process would closely resemble the existing conventional activated sludge treatment process at the plant. In this process the activated sludge returned from the secondary clarifiers is aerated in a separate basin (the stabilization tank) before being mixed with the raw sewage. The combined raw sewage/RAS flow is then aerated using standard DO control in the contact tanks. Unaerated selector zones ahead of the contact tanks would be used to control filamentous bacteria by recycling nitrate-rich mixed liquor into the raw sewage. This alternative would have the least impact to operation and maintenance since the staff is accustomed to the processes and control. Waste-activated sludge from the clarifiers would be sent to the existing aerobic digester. Also, anaerobic digestion facilities would be constructed and are described in Section 6.4. A schematic diagram of this treatment process alternative is presented in Figure 6-1.

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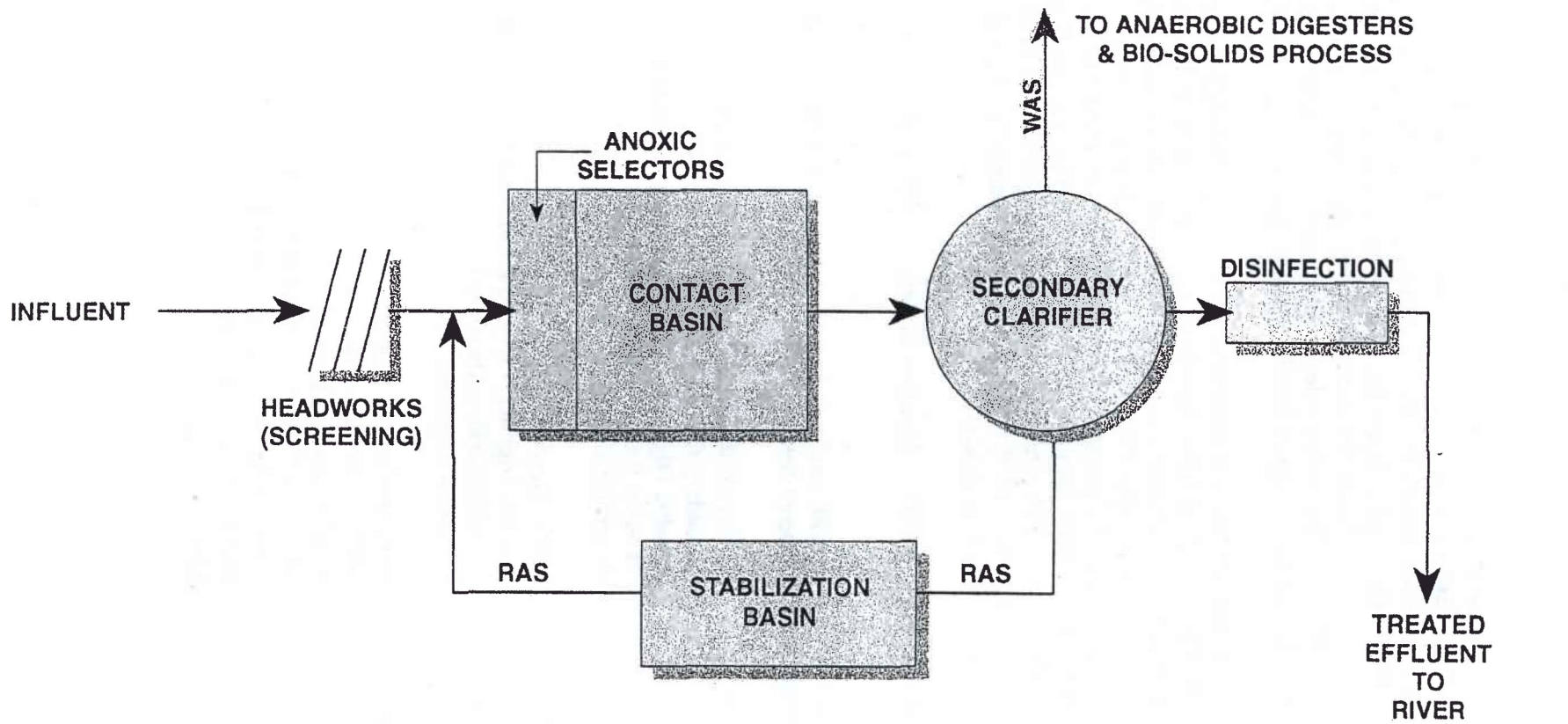

RAS – Return Activated Sludge
WAS – Waste Activated Sludge

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NOTE: EXISTING CLARIFIER NOT SHOWN

Figure 6-1
Treatment Alternative 1
Construct Stabilization With Anoxic Selectors

The second alternative considered is a sequencing batch reactor (SBR). Several manufacturers offer these as package units where all of the mechanical equipment required is supplied and installed in concrete tanks and structures built by the Owner. An SBR completes aeration and decanting of the mixed liquor in a single basin in 4- to 6-hour cycles. Two reactors are provided so that while one is filling the other is undergoing aeration and decanting of the clarified secondary effluent. The anoxic selector is created by sequencing aeration and idle cycles in the basin. A schematic diagram of this treatment process is presented in Figure 6-2.

The third alternative considered was the construction of trickling filters upstream of the existing activated sludge process. Trickling filters have been successfully used as an add-on process to provide additional organic and ammonia removal beyond that which can be achieved in the conventional activated-sludge process. Placing trickling filters before the existing aeration basin would require the installation of a primary clarifier between the headworks and the trickling filters. The existing aeration basin would remain in service, and the aerobic digester would be converted to biosolids storage. This alternative would also require the use of filtration for the secondary effluent to assure that a 10/10 effluent could consistently be attained. A schematic diagram of this treatment process is shown in Figure 6-3.

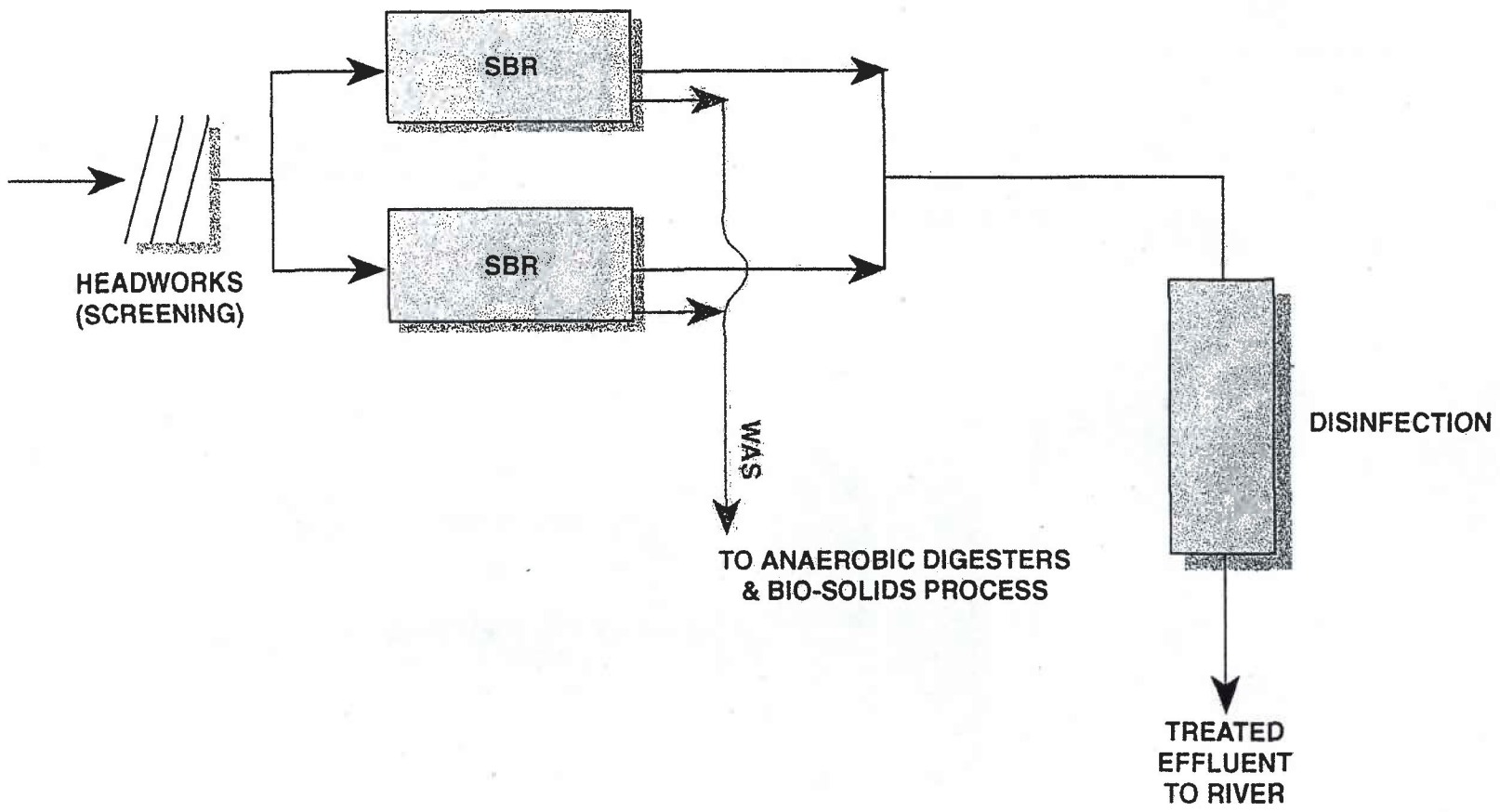
In both Alternatives 2 and 3, anaerobic digestion facilities, which are described in Section 6.4, would be added.

The unit processes associated with each of these alternatives and the use of each existing structure at the plant are summarized below:

Alternative 1
Contact Stabilization Basin with Anoxic Zones – New
Secondary Clarifiers – New and Existing
Aerobic Digester – Converted to Biosolids Storage
Aeration Basin – Abandoned
Anaerobic Digesters – New

Alternative 2
SBR – New
Aerobic Digester – Converted to Biosolids Storage
Aeration Basin – Abandoned
Anaerobic Digesters – New

Alternative 3
Primary Clarifier – New
Trickling Filters – New
Aeration Basin – Existing
Secondary Clarifiers – New and Existing
Aerobic Digester – Converted to Biosolids Storage
Secondary Effluent Filters – New
Anaerobic Digesters – New



WAS - WASTE ACTIVATED SLUDGE

NOTE: EXISTING CLARIFIER NOT SHOWN

Figure 6-2
Treatment Alternative 2
Sequencing Batch Reactor (SBR)

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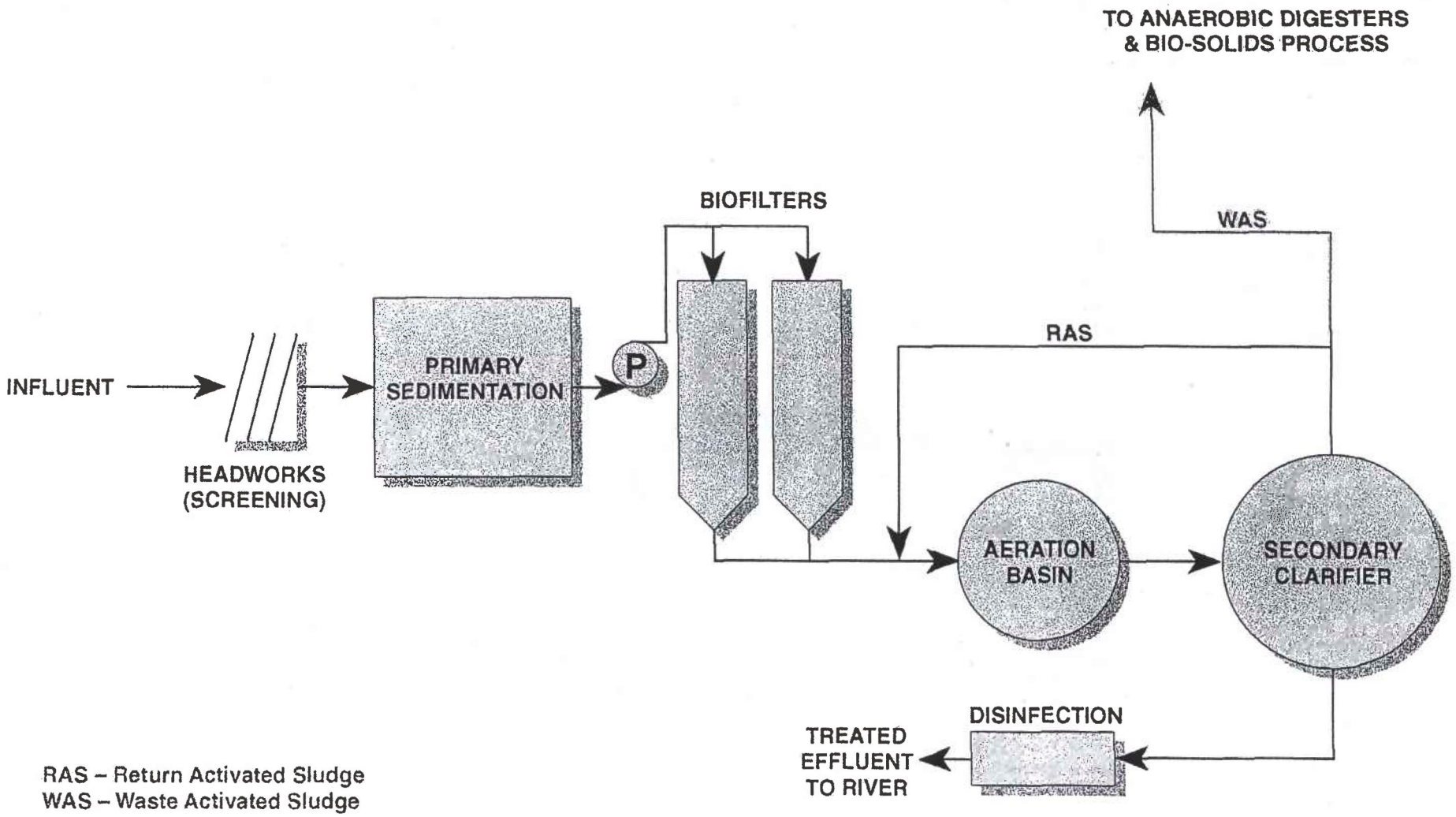

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NOTE: EXISTING CLARIFIER NOT SHOWN

Figure 6-3
Treatment Alternative 3
Add Biofilter Process

In addition to the three plant expansion alternatives above, several plant improvements are needed, as suggested in Section 5.4. Other plant improvements are also needed to meet future treatment requirements regardless of which of the three expansion alternatives is selected. These improvements include:

- Upgrade the influent pump station to handle future wastewater flows and provide variable-speed pumps. This improvement was identified in Section 5.4.

- Modify the existing headworks to remove the comminutor and install additional screening. This will effectively remove rags and plastics and provide the existing screen as a backup unit.

- Provide adequate disinfection for future wastewater flows.

- Investigate the need for an emergency effluent pump station to provide for effluent discharge to the river during high river elevations.

To reduce the effect of the influent pump operation on downstream treatment processes, it is recommended that variable-speed pumps be installed. Originally, there were mechanical devices to regulate the flow, but this system no longer functions. Installation of variable-speed pumps would allow treatment units at the plant to operate more efficiently.

It is also recommended that a new screen be installed. Presently, the comminutor is used at the headworks, but this merely grinds large solids into a smaller, more manageable size material. Unwanted debris, hair, and plastics are still in the wastewater stream. It would be better for equipment and the composting operation if these materials could be removed. A screen installed in the present headworks would perform this function. The headworks structure would have to be modified, however, to accommodate a new screen. This would include raising the outside walls to provide for adequate hydraulic capacity.

To provide for future disinfection, a new ultraviolet disinfection system would be required.

An emergency effluent pump station may also be a required plant improvement. If required, effluent pumps would be sized to match influent pumping capacity.

Recent Rogue River flood elevation mapping has apparently raised the projected 100-year flood elevations in the river at the plant site from those previously identified. At this new higher projected elevation, wastewater flows can no longer gravity discharge to the river as suggested in the original 1977-78 design. Instead, an effluent pump station would be necessary to ensure plant discharge capabilities during extreme river floods.

During design of any plant improvement, the need for an effluent pump station would be carefully evaluated. An analysis of the need for an effluent pump station would be based on the

hydraulics of the expanded process which would show the effects of an extreme flood condition at the plant.

6.3 ALTERNATIVES 4 AND 5

Preliminary sizing criteria for each of the two alternatives investigated which meet the effluent limitations established by OAR 340-41-026 (shown in Table 5-5) are presented in Table 6-2.

Table 6-2
Preliminary Sizing Criteria, New Unit Treatment Processes
Treatment Plant Expansion Alternatives Meeting OAR 340-41-026

Unit Process	Size Criteria
Alternative 4 – Complete Mix/Anoxic Selector with Effluent Filtration	
Anoxic Selector Basins	
No. of Trains	2
No. of Cells	3
Total Volume	10,300 ft^3
Anoxic Basins	
No. of Trains	2
No. of Cells	1
Total Volume	29,800 ft^3
Aeration Basins	
No. of Trains	2
No. of Cells	2
Total Volume	121,000 ft^3
Secondary Clarifier[1]	
Diameter	60 ft
Depth	13 ft
Surface Area (effective)	2,714 ft^2
NOTE: Polymer addition system would be provided to improve clarifier performance if 10 mg/ℓ suspended solids limits were not being met.	
Effluent Filtration System	
Average Daily Loading	4 gpm/ft^2
Maximum Daily Loading	6 gpm/ft^2
Total Surface Area	360 ft^2, minimum

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Table 6-2
Preliminary Sizing Criteria, New Unit Treatment Processes
Treatment Plant Expansion Alternatives Meeting OAR 340-41-026

Unit Process	Size Criteria
Alternative 5 – SBR with Effluent Filtration	
Sequencing Batch Reactor	
Number	2
Dimensions	75′ x 65′ x 17′ SWD
Cycles per Day	5 – summer, 6 – winter
Anaerobic Digesters	
Volume (total of 2)	17,000 ft^3
Dimensions	25′ dia. - 20′ SWD
Effluent Filtration System	
Average Daily Loading	4 gpm/ft^2
Maximum Daily Loading	6 gpm/ft^2
Total Surface Area	360 ft^2, minimum

(1) In Alternative 4, existing secondary clarifier remains in service.

A new anoxic selector complete mix activated-sludge process would closely resemble the existing conventional activated-sludge treatment process at the plant but would be much larger. In this process, the return activated sludge from the secondary clarifiers would be returned to the anoxic selector basins to be combined with raw sewage. Also, mixed liquor would be recycled to the anoxic selector to provide for substantial ammonia removal. To meet effluent limitations of OAR 340-41-206, effluent filtration would be required. A schematic diagram of this treatment process is presented in Figure 6-4.

The second alternative being considered to meet the effluent limitations of OAR 340-41-026 is an SBR with effluent filtration. This alternative is essentially identical to Alternative 3 presented in Section 6.1.1, except that it also includes effluent filtration. This alternative also requires a flow equalization basin prior to effluent filtration due to flow fluctuations from the SBR process. A schematic diagram of this treatment process is presented in Figure 6-5.

For both Alternatives 4 and 5, anaerobic digestion would be added as described in Section 6.4.

The unit processes associated with each of these alternatives and the use of each existing structure at the plant are summarized below:

Alternative 4
Anoxic Selector/Anoxic/Aeration Basin – New
Secondary Clarifiers – New and Existing
Filtration – New
Aerobic Digester – Converted to Biosolids Storage
Aeration Basin – Abandoned
Anaerobic Digesters – New

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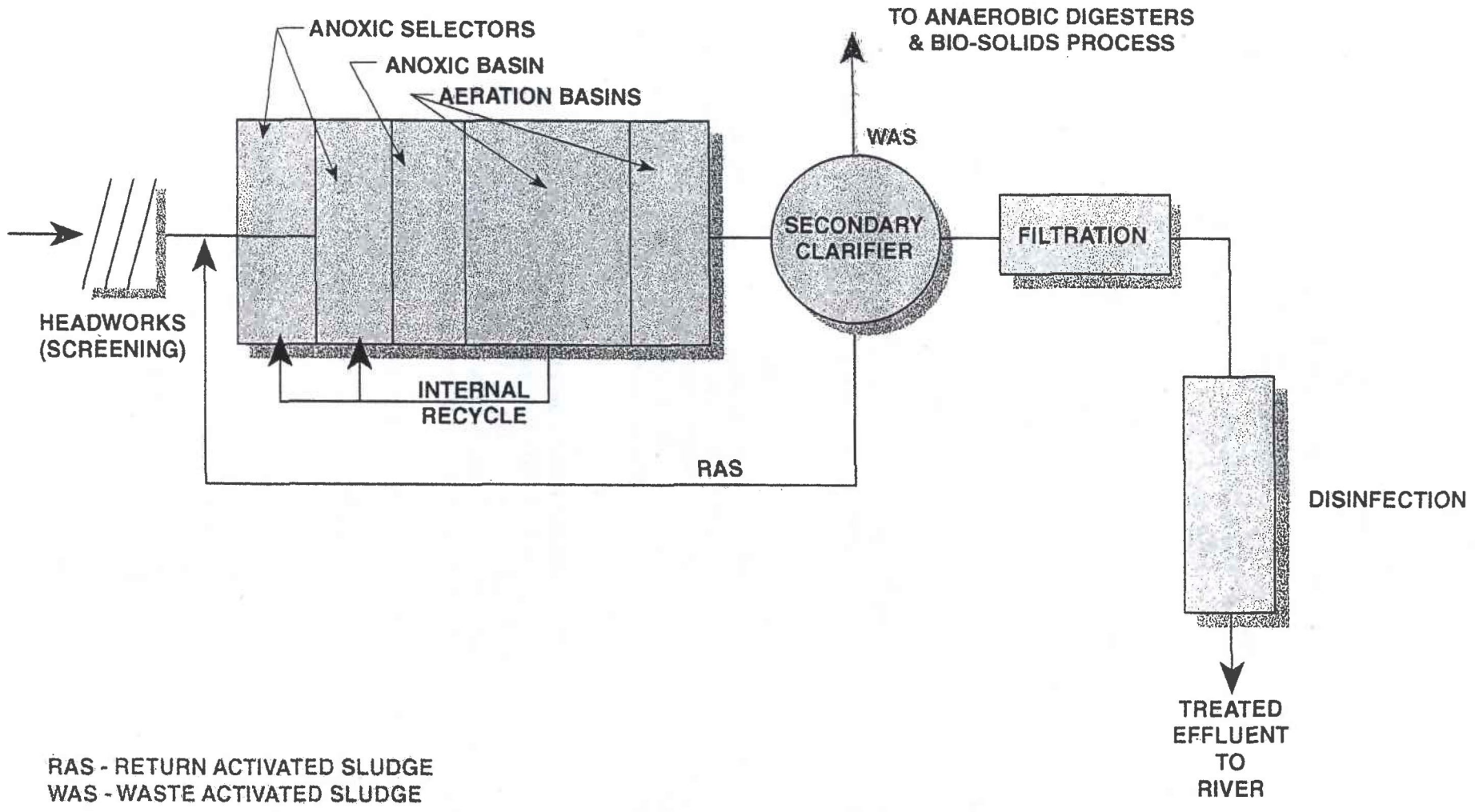

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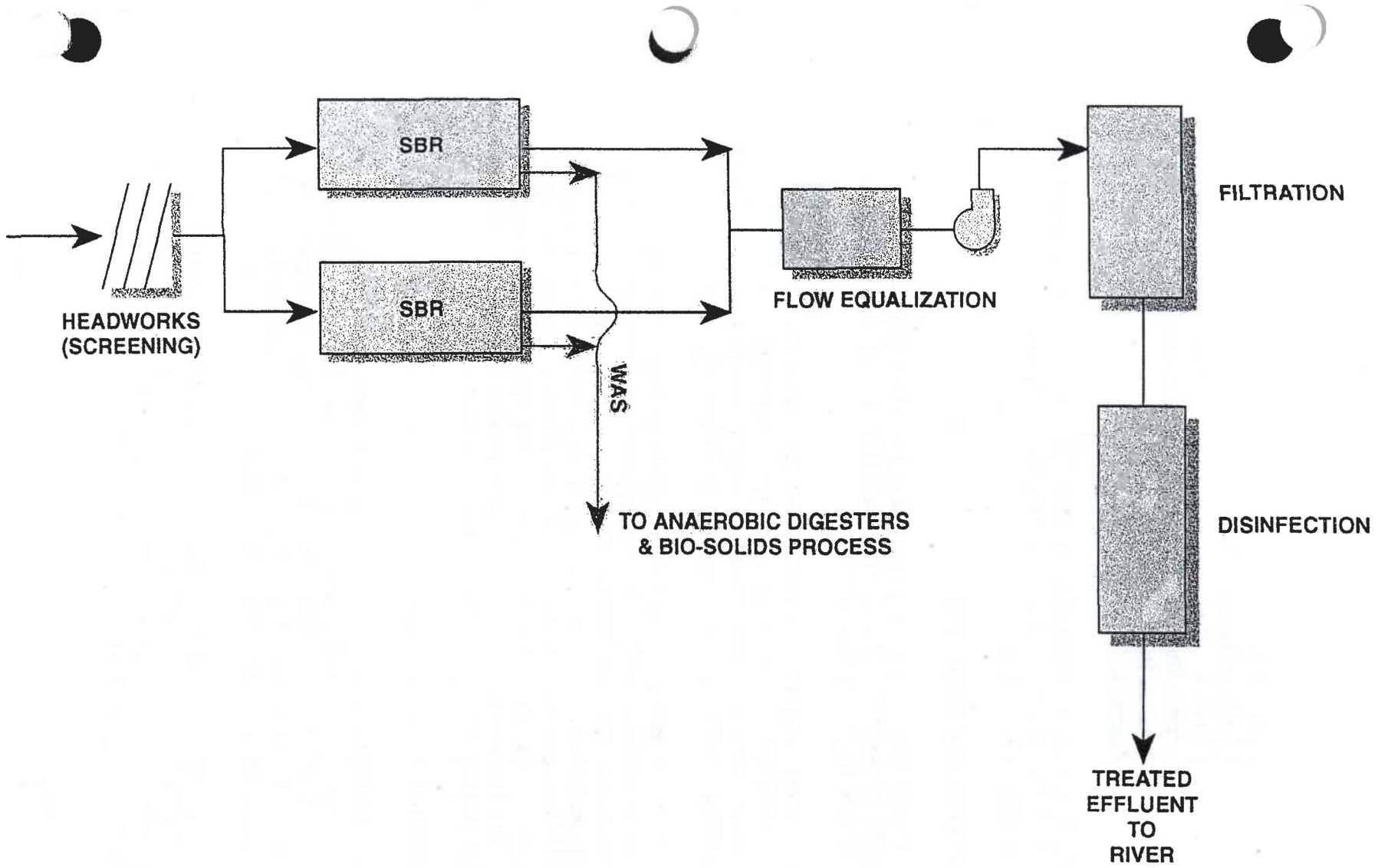
NOTE: EXISTING CLARIFIER NOT SHOWN

Figure 6-4
Treatment Alternative 4
Complete Mix/Anoxic Selectors
With Effluent Filtration



WAS - WASTE ACTIVATED SLUDGE

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NOTE: EXISTING CLARIFIER NOT SHOWN

Figure 6-5
Treatment Alternative 5
Sequencing Batch Reactor
(SBR) With Effluent Filtration

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Alternative 5	SBR – New Flow Equalization – New Filtration – New Aerobic Digester – Convert to Biosolids Storage Aeration Basin – Abandoned Anaerobic Digester – New	

Similar to the three plant alternatives discussed in Section 6.1.1, several plant improvements are also needed as part of Alternatives 4 and 5, as suggested in Section 4. All of these improvements are provided in either Alternative 4 or 5.

6.4 NEW BIOSOLIDS TREATMENT PLAN

Regardless which of the Alternatives 1 through 5 are evaluated, new biosolids treatment facilities will be necessary at the Redwood WWTP. This is because the existing biosolids composting operation has been ordered to close in October 1999.

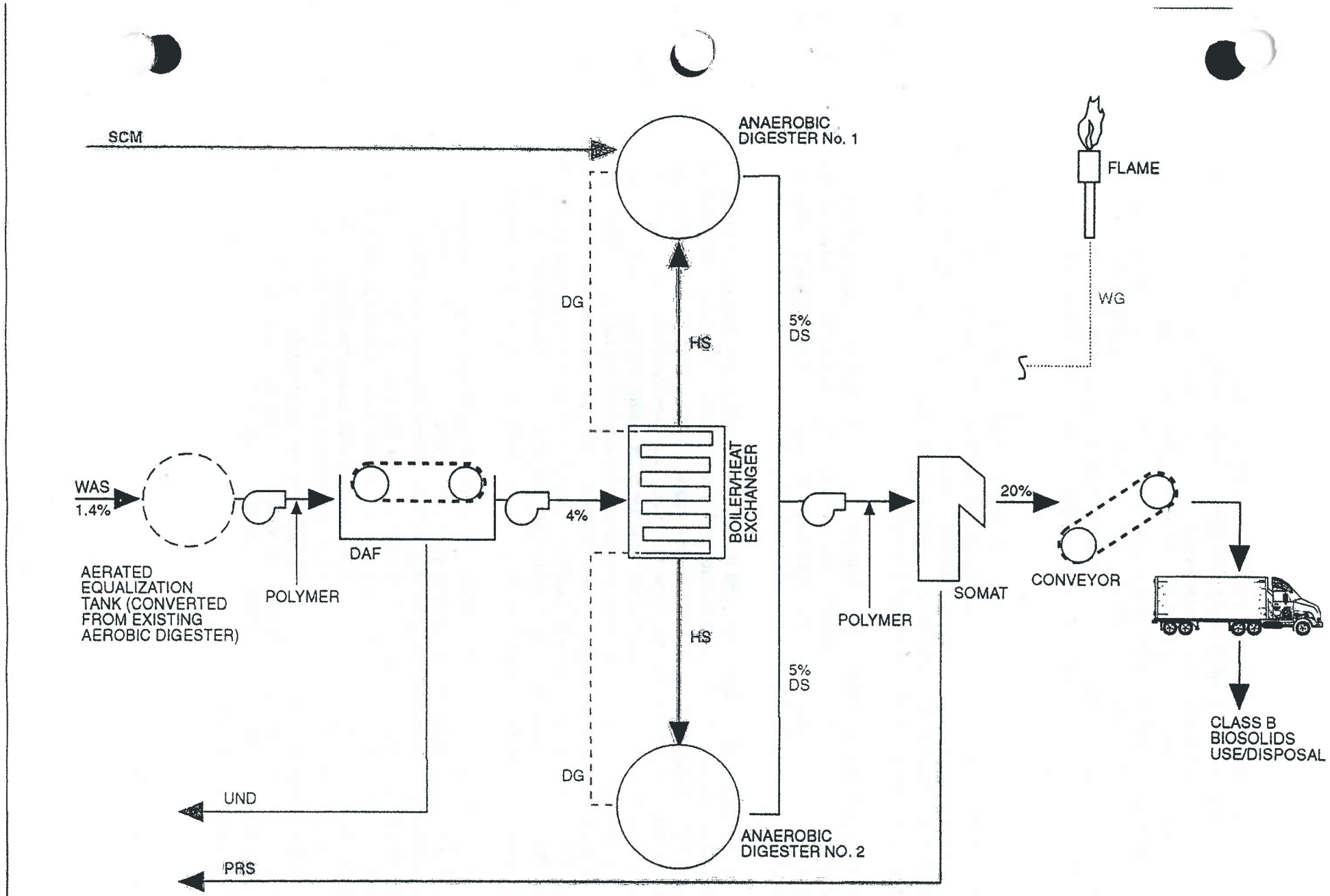
The District's biosolids composting facility, which has been in operation since 1988, aerobically digests sludge and blends it with yard waste, saw dust, lumber waste and livestock bedding to produce a material sold as a soil amendment. The composting operation is designed as a process to further reduce pathogens (PFRP), sometimes referred to as Class A biosolids treatment.

Concerns about odor and dust generation from the composting operation are the main reasons why composting must be closed in 1999. Wastewater treatment alternatives 1 through 5 therefore need to include a new biosolids treatment and disposal process. The existing aerobic digesters do not have adequate future solids loading capacity nor can they meet PFRP standards. The Grants Pass WRP uses anaerobic digesters to treat biosolids before they are land applied. For evaluation purposes, it was assumed that the Redwood WWTP would also require new anaerobic digesters. The biosolids would then be land applied, which is similar to the biosolids handling and disposal plan currently used at the Grants Pass Facility.

6.4.1 Anaerobic Digestion Process Description

Anaerobic digestion is a biological process that uses bacteria, in an oxygen-free environment, to convert volatile solids into carbon dioxide, methane, and ammonia. The best suited anaerobic digester for the Redwood WWTP would be a mesophilic single-stage, high-rate digester. High-rate anaerobic digestion combines active mixing and carefully controlled, elevated temperature to increase sludge stabilization rate. Figure 6-6 shows a process flow diagram for this system.

Temperature variations are more critical to the anaerobic digestion process than the aerobic digestion process, and careful temperature control is required to avoid process upsets. The Redwood WWTP anaerobic digesters would be operated in the mesophilic temperature range of 30 to 38°C. Gas production, a by-product of anaerobic digestion, is optimized in this range.



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SCM - Scum
WAS - Waste Activated Sludge
SUP - Supernatant
CEN - Centrate
PRS - Pressate
UND - Underflow
HS - Heated Sludge
DS - Digested Sludge
DG - Digested Gas
WG - Waste Gas

Figure 6-6
Anaerobic Digestion Flow Schematic

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A typical municipal anaerobic digester handling primary and waste-activated sludge should produce approximately 15 to 18 cu-ft gas/lb of volatile suspended solids (VSS) destroyed. The amount of gas produced is a function of temperature, sludge retention time, and volatile solids loading. Methane and carbon dioxide are the two main constituents of digester gas, nitrogen, hydrogen, and hydrogen sulfide are found in trace amounts. Methane concentrations range from 60 to 70 percent by volume. Typical digester gas has a heat content between 500 to 700 BTU/cu-ft and may be used as a fuel supply for heating incoming sludge or for space heating of WWTP buildings. For periods when digester gas production is low, natural gas may be used as a supplemental fuel source.

Two types of covers are available for anaerobic digesters: fixed and floating. Floating covers may either directly float on the sludge, or rest on side skirts and float on the gas (gas holder type). Floating covers are generally more expensive than fixed types, but provide better safety against explosions. The District may wish to consider the benefits of floating covers at a later time; however, a fixed cover is used in this analysis.

Several heating methods have been used with anaerobic digesters, including submerged burners, steam injection, internal heat exchangers, and external heat exchangers. External heat exchangers are the most popular because of their flexibility and ease of maintenance. This type of heat exchanger is recommended for the Redwood WWTP.

Three general mixing methods have been used in anaerobic digesters: mechanical, pumped, and gas recirculation. Mixing the digester contents minimizes thermal stratification, disperses the substrate for better contact with the active biomass, reduces scum and grit buildup, dilutes any inhibitory substances or adverse pH and temperature feed characteristics, increases the effective volume of the reactor, allows reaction gases to separate more easily, and keeps more inorganic material in suspension. A gas recirculation system was chosen for this evaluation. Mechanical mixers and pump impellers are subject to wear resulting from grit and debris abrasion. The Redwood WWTP has no grit removal facilities, so grit is expected to accumulate in the digesters. The digester should be designed with highly sloped floors, several withdrawal ports, and easy access though wall openings near the ground to facilitate cleaning and grit removal.

The first step of this process would be aerated equalization. Equalization prior to thickening would facilitate even feeding of the thickening device. Next, sludge is thickened prior to anaerobic digestion, which reduces sludge heating and tank volume requirements. Pre-thickening the sludge also helps maintain constant feed conditions to the reactor. With dissolved air floatation thickening, some grit can be removed prior to digestion. Following thickening to approximately 4 percent solids, the sludge is transferred into the anaerobic digesters. Digester gas is fed into a boiler. Waste gas in excess of that needed for the boiler and gas mixing systems would be flared or alternately stored during periods when gas production is low. A heat exchanger transfers heat from the boiler to feed sludge. Following anaerobic digestion, the sludge is dewatered in the most cost-effective device, a SOMAT press, and subsequently conveyed to a trailer for hauling.

Chemical feed systems may be necessary if changes in alkalinity, pH, sulfides, or heavy metals cause process upsets. The ability to feed certain chemicals such as sodium bicarbonate, ferrous chloride, ferrous sulfate, lime, and alum should be considered early in the design process. Chemical feed equipment would include standby chemical metering pumps, several points of application, and associated piping.

To facilitate process control, all water, sludge, and gas flows should be metered. Sight access ports should be provided on the digester tank as well as 36-inch-diameter manholes in the roof and on the lower portions of the digester tanks to facilitate cleaning.

The major advantages of anaerobic digestion include the following:

- Excess energy over that required by the process is produced. Methane can be used to heat and mix the reactor. Excess methane gas can be used to space heat.
- The quantity of total solids is reduced over the aerobic digestion process. About 30 to 40 percent of the total incoming solids may be destroyed.
- The product is a stabilized sludge, that may be free from strong or foul odors.

The major disadvantages of anaerobic digestion are as follows:

- The digester is easily upset by unusual or erratic conditions and therefore requires increased operator attention; it is also slow to recover.
- Anaerobic digestion requires more equipment than other processes (i.e., boiler, heat exchanger, waste gas flare), and this greatly increases system complexity.
- Cleaning operations are difficult because the reactors are closed vessels.
- Grit can accumulate in the reactors, and grit removal facilities are not currently part of the Redwood WWTP design.
- A natural gas main would need to be constructed from the Redwood Elementary School to the plant to supplement digester gas when digester gas production is low. This gas main may cost the District as much as $50,000 per mile (Ball, E., October 17, 1996).
- There is a possibility of explosion as a result of inadequate operation and maintenance, leaks, or operator carelessness.
- Gas line condensation or clogging can cause major maintenance problems.

6.4.2 Preliminary Design Criteria

Factors that govern the design of anaerobic digestion systems include feed characteristics, temperature, solids loading, detention time, mixing method, heating requirements, and energy recovery. Design parameters were evaluated, and suitable preliminary design criteria and equipment selected for an anaerobic digestion system at the Redwood WWTP. Preliminary design criteria are listed in Table 6-3.

Table 6-3
Anaerobic Digestion Preliminary Design Criteria

Volatile Suspended Solids Reduction (minimum):	40 – 60%
Total Solids Reduction:	33 – 50%
Detention Time:	18 days
Design Temperature:	35°C
Feed Concentration:	4.0%
Feed Rate (maximum month):	8,540 gpd
Solids Loading (maximum month):	0.13 lbs VSS/cu-ft/day
Anaerobic Digesters:	
Number:	2
Material:	Concrete
Type:	Aboveground
Volume (each):	128,000 gal
Diameter:	25 feet
Height:	20 feet
Cover:	Fixed
Floor Slope:	1:6
Aerated Equalization Tank (existing):	
VSS Reduction:	<9%
Digester Mixing:	
Type:	Gas recirculation
Gas Requirement:	55 – 75 scfm/compressor
Power Consumption:	2½ HP
Foam Cutters:	Included
Digester Heating:	
Boiler type:	Recirculating water
Gas Production:	18,000 cu-ft/day
Methane Content:	66.6%
Minimum Incoming Sludge Temperature:	15°C
Heat Value of Gas:	+ 7.0 million BTU/day
Requirement for Sludge Heating:	- 2.6 million BTU/day
Transfer Losses (tank, floors, walls)	- 2.1 million BTU/day
Net Heat Surplus (deficit):	2.3 million BTU/day

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6.4.3 Site Plan

Figure 6-7 shows how the facilities might be placed on the site. Ample space exists for all proposed facilities. The biosolids handling building footprint is based on space requirements for a dissolved air floatation thickener and SOMAT press for dewatering device. An integral digester operations and control building is located between the two digesters. This building would contain the boiler and heat exchanger, process pumps, polymer feed equipment, piping, valves, flow meters, and appurtenances. A biosolids pump station adjacent to the equalization tank pumps sludge to the thickening device. Gas storage and odor control facilities are not included in this evaluation. Facilities are grouped together to minimize site piping.

6.4.4 Capital and Operations and Maintenance Costs

The estimated cost to construct anaerobic digesters is $3,370,000 ($2,590,000 without contingency). A detailed breakdown estimate of probable costs is presented in Appendix G. Included in this estimate are two aboveground concrete digester tanks with fixed covers, vessel access platforms and ladders, integral operations building, thickened sludge transfer pumps, conversion of existing aerobic digester to an aerated sludge equalization tank, equalization transfer pumps, yard piping, dedicated trailer for dewatered sludge storage, interconnecting piping, valves, and appurtenances, reactor design, installation, start-up and training services. Sludge thickening and dewatering facilities are estimated separately. Additional facilities that may be needed upon further review, such as polymer feed equipment and waste gas scrubbing equipment, are not included in this estimate. It is assumed no natural gas will be used in the process.

Operations and maintenance costs shown in Appendix G include pumping of thickened sludge, equalized sludge, and digested sludge; conveying dewatered sludge; and compressing digester gas for mixing requirements. Sludge thickening and dewatering labor requirements, as well as polymer costs, are included. Heating energy for the anaerobic digestion process is supplied primarily by digester gas recovery.

6.4.5 Preliminary Screening of Alternatives 1 Through 5

To reduce the complexity of evaluating twelve different alternatives, a preliminary screening of Alternatives 1 through 5 was conducted based on cost. Capital cost estimates for the each of these five alternatives, which include the cost of new biosolids treatment, are shown in Table 6-4. Detailed cost estimates are included in Appendix G. Operation and maintenance cost estimates for each of these alternatives are summarized in Table 6-5. Also shown in this table is the operation and maintenance cost of the existing Redwood WWTP for 1999.

Based on cost, Alternative 1 - Contact Stabilization and Alternative 4 – Complete Mix/Anoxic Selector/Filter were selected for further comparison to Alternatives 6 through 12.

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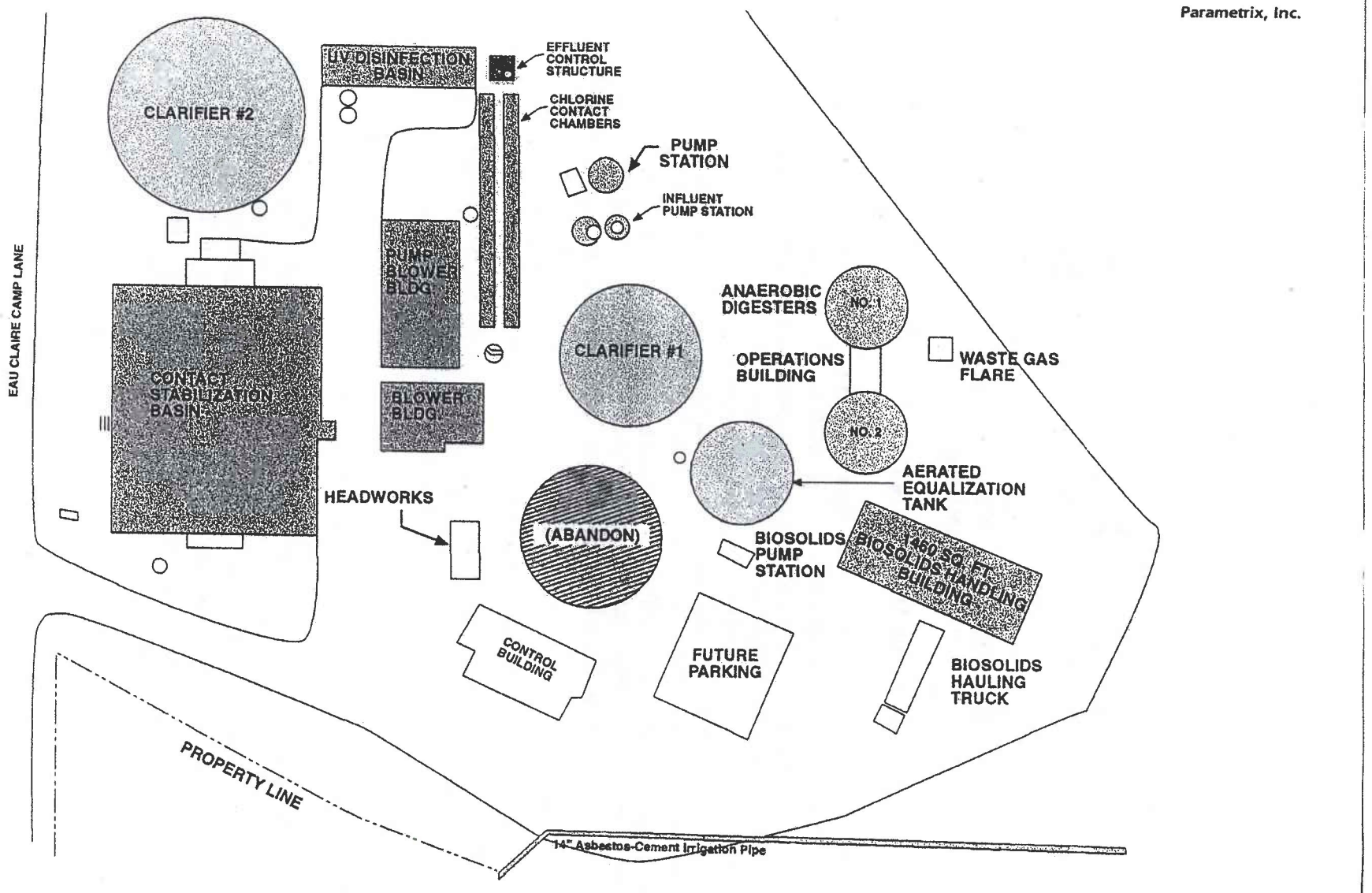
Parametrix, Inc.



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Scale

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Figure 6-7
Treatment Option
Site Plan

Table 6-4
Treatment Alternatives 1 through 5
Estimate of Probable Costs (costs in $1,000s)

Wastewater Process	Alt. 1 Contact Stabilization	Alt. 2 SBR	Alt. 3 Trickling Filter	Alt. 4 Complex Mix Anox Select Filter	Alt. 5 SBR Filter
Biofilters	--	--	$719	--	--
Aeration Basin	$1,239	--	--	$1,742	--
Secondary Clarifier	$513	--	$513	$513	--
SBR Basin	--	$1,717	--	--	$1,585
Blower Building	$129	--	--	$129	$123
Primary Sedimentation	--	--	$586	--	--
Package Filtration	--	--	$409	$409	$409
Effluent EQ	--	$95	--	--	$95
Influent Pump Stn Mods	$113	$148	$115	$113	$148
Headworks Mods	$121	$161	$56	$121	$161
UV Disinfection	$315	$315	$315	$315	$315
Site/Civil	$72	$72	$72	$72	$72
Yard Pipe	$72	$72	$72	$108	$108
Electrical	$433	$433	$503	$559	$475
Subtotal	**$3,008**	**$3,014**	**$3,356**	**$4,082**	**$3,491**
Biosolids Process	$2,590	$2,590	$2,590	$2,590	$2,590
Contingency, 30%	$1,679	$1,681	$1,784	$2,002	$1,824
Subtotal Construction	**$7,278**	**$7,285**	**$7,730**	**$8,673**	**$7,906**
Engineering and Admin, 22%	$1,601	$1,603	$1,701	$1,908	$1,739
Total Project Cost[1]	**$8,880**	**$8,890**	**$9,430**	**$10,580**	**$9,650**

(1) Rounded to three significant figures.

Table 6-5
Treatment Alternatives 1 through 5
Annual O&M Costs

	Existing Plant	Alt. 1 Contact Stabilization	Alt. 2 SBR	Alt. 3 Biofilter	Alt. 4 Comp Mix Effluent Filter	Alt. 5 SBR with Effluent Filter
Salary and Wages	$55,500	$73,800	$73,800	$73,800	$84,300	$84,300
Employee Benefits	$18,949	$25,250	$25,250	$25,250	$28,850	$28,850
Supply and Material	$24,808	$27,808	$26,400	$29,800	$31,500	$31,000
Services	$55,835	$77,235	$76,800	$80,500	$82,500	$81,500
Interfund and Inter Gov.	$123,254	$136,700	$135,500	$144,000	$146,000	$145,000
Capital Replace/ Improvements	$39,870	$49,370	$49,370	$51,400	$55,400	$55,400
Subtotal	**$318,216**	**$390,163**	**$387,120**	**$404,750**	**$428,550**	**$426,050**
Anaerobic Digestion	$193,760	$193,760	$193,760	$193,760	$193,760	$193,760
Total O&M Cost[1]	**$512,000**	**$584,000**	**$581,000**	**$599,000**	**$622,000**	**$620,000**

(1) Rounded to three significant figures.

6.5 ALTERNATIVES 6 THROUGH 12

An alternative to upgrading the Redwood WWTP (Alternatives 1 through 5) would be to convey all wastewater to the Grants Pass WRP for treatment. This option is evaluated in Alternatives 6 through 12. Each of these alternatives utilize a slightly different route of conveying wastewater between these two points. In each case, a conveyance system consisting of either one or two pump stations and associated sewer force mains would be used to transfer all wastewater flow from the Redwood WWTP to the Grants Pass WRP, where it would be treated.

For any of these alternatives, centralized wastewater treatment for the entire Grants Pass area is provided at one treatment plant rather than at two plants. This reduces the operation and maintenance costs significantly. The following information is presented in this section for each of these alternatives.

- Design criteria
- Description of alternatives
- Capital costs estimates
- Preliminary screening of alternatives

6.5.1 Design Criteria

Based on the existing wastewater flow data presented in Table 4-1 and the wastewater flow projections presented in Table 5-1, Alternatives 6 through 12 design criteria are presented in Table 6-6.

Table 6-6
Treatment Alternatives 6 through 12
Design Flows

Item	1998 Actual	2020 Estimated
Average Flow, mgd		
Summer	0.49	0.88
Winter	0.63	1.22
Annual	0.56	1.05
Max. Day, mgd		
Summer	0.77	1.43
Winter	1.79	3.08
Min. Day, mgd	0.28	0.76

Pump Stations

To ensure that the facility has adequate capacity, a peaking factor was applied to maximum daily flow to obtain a peak hourly design flow of 4.2 mgd. Designing conveyance facilities that would handle such a large range of flow, 0.28 mgd to 4.2 mgd, is not a simple task. The pump station would also be required to have standby pumping capacity. State guidelines require that pump station capacity be met with the largest pump out of service.

Alternatives discussed later in this section include either one pump station or two pump stations. The larger pump station in the two-station alternative is very similar to that required for the single station alternatives. Design of this pump station would be based on the selected pipeline route but can be summarized for all alternatives as follows:

- Three pumps (one stand-by)
- 1,460 gpm, each pump (2.1 mgd each)
- Approximate total pumping head: 180 to 200 feet depending on route selection
- Variable speed drives

It is recommended that variable speed drives be used on the large pump station because of the high head associated with the peak flows. Variable speed drives will result in lower electrical costs and more continuous flow to the Grants Pass WRP.

To provide the high head required to convey the wastewater five to six miles to the Grants Pass WRP, it is necessary for the pump station to either consist of several sets of two pumps in series or operate single pumps at higher speed (1,800 rpm). The cost estimates in this report are based on 1,800 rpm pumps. This is because the cost to operate two pumps at a slower rpm in series would be more expensive than the cost to operate larger single pumps.

High-head pumping stations with long force mains often experience surge conditions. During design, the conveyance system should be analyzed to determine the surge conditions and provide appropriate protection as needed.

If there is only one pump station, it will be located at the same location and depth as the Redwood WWTP influent pump station (RI-0). The structure housing the pumps will be approximately 23 feet deep. The existing WWTP influent pump station dry well is too small and will be replaced with a new dry well large enough for the three pumps. An above-grade structure will house the chemical injection system, motor control center, and emergency generator.

It appears that the existing wet well could be used for two pumps. The two existing 12-inch suction pipes are adequately sized for the new pumps.

Alternatives involving two pumping stations locate a small pump station at the existing Redwood WWTP (RI-0) and a second larger pump station at manhole RI-25. Manhole RI-25 is near the north end of Darneille Lane, and near the west boundary of the designated urban growth area. Pump Station RI-0 would accept flow that is collected from the area west of RI-25 and would pump it to the larger station through a force main.

Construction of the small pump station at RI-0 would involve replacing the pumps and some of the electrical and mechanical equipment in the existing influent pumping station. The new pumps would each be sized for 0.5 mgd (350 gpm) at 40 feet of total pumping head, based on a 6-inch force main going to the larger pump station at RI-25. These pumps would likely need to be upgraded in year 2020.

Sewer Force Main

Because raw wastewater contains grit and other solids, force main flow velocity is a critical design factor. If the wastewater velocity in the force main is too fast, the grit and sand can prematurely erode the inside of the pipe. If the flow velocities are too slow, solids will settle

and accumulate in the pipe causing sedimentation and pipe blockage. The design criteria for any force main is as follows:

- Provide minimum pipe scour velocity for the following conditions:
  - Noncontinuous flow: 3 feet per second at start-up and ultimate peak flows
  - Continuous flow: 2.0 feet per second

- Keep pump head below 100 feet if possible

Based on the above criteria, preliminary design is based on the providing twin 12-inch force mains with Class 250 pressure rating.

The advantages to providing the twin force mains would be as follows:

- Allows for adequate scour velocity at start-up flows while minimizing pumping head at ultimate flows.

- Reduces odor and corrosion problems by reducing detention time in the pipeline and providing better scouring action.

- Allows for maximum pump turn down and optimum pump selection.

- Reduces initial peak flows to the Grants Pass WRP, by allowing for variable speed drives or a smaller lead pump.

Operating the conveyance system involves selecting the appropriate pipe (or pipes) to provide capacity and maintain proper scour velocities. Pipe selection will be done by valve manipulation at the pump station, either manually or automatically. In the first five years of operation, only one pipe will be used, except during occasional very high flows. Ultimately, the conveyance system valves will be set for continuous use of both pipes.

Odor and Corrosion Control

Generally, for long sewage force mains experiencing varying flows and detention times, it is necessary to provide odor and corrosion protection. Preliminary design for the conveyance system is based on providing the following methods of minimizing odor and corrosion:

- Two points of chemical injection: One at Pump Station RI-0, and one at a single point along the pipeline route (at Pump Station RI-25 for the two station alternatives).

- Activated carbon canisters at any automatic air release valves

The wastewater constituent responsible for most odor and corrosion problems is hydrogen sulfide (H_2S). Many of the other odor compounds in the wastewater are also reduced by targeting H_2S.

A variety of chemicals have been used to oxidize H_2S. The two most common for wastewater force mains are sodium hypochlorite and hydrogen peroxide. Sodium hypochlorite injection is less costly to operate; however, hydrogen peroxide provides the added benefit of providing residual oxygen to limit the reformation of H_2S. Another product available for odor control is Bioxide™. These three systems should be evaluated during detailed design.

Automatic air release valves will be provided at any high points in the pipeline. They are a potential source of odor, particularly during low summer flows. To minimize these odors, activated carbon canisters will be connected at the exhaust of each air release valve.

6.5.2 Description of Alternatives 6 Through 12

Seven potential pipeline routes were investigated for this study. The two most critical factors in selecting the route are the location and method of crossing the Rogue River and length of roadway to be disturbed.

River Crossing Alternatives

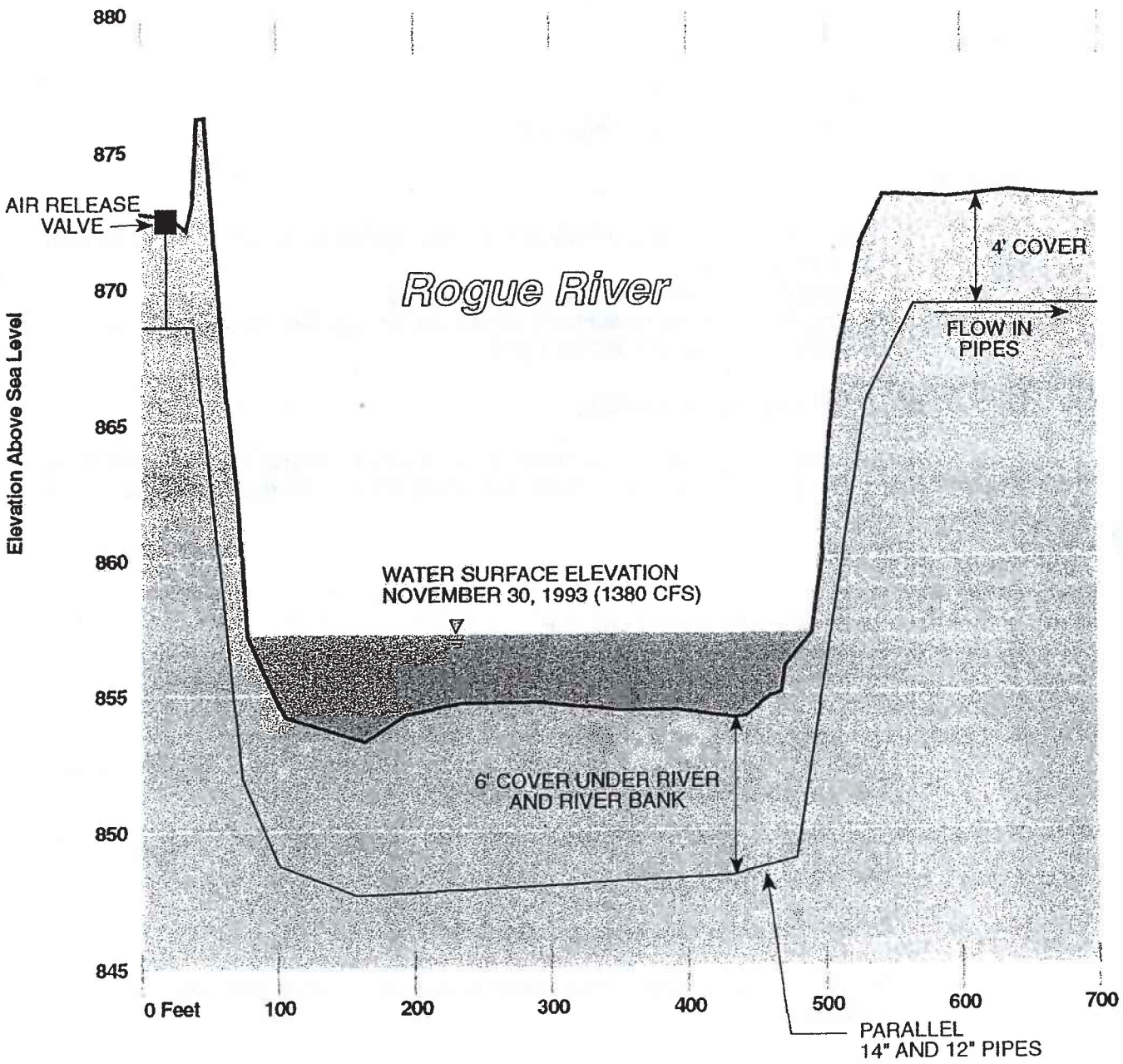
The following options for crossing the river were investigated:

- Open cut the river crossing
- Tunnel (boring) under the river
- Cross the river at the proposed pedestrian bridge near the Grants Pass WRP
- Cross the river at the existing bridge for SR 199

Tunnel crossing or open-cut alternatives would require investigation of geotechnical and bathymetric conditions. There are many potential locations for either of these options, but only two of the more feasible locations were evaluated.

Open Cut Crossing

A cross section of the river crossing near the Redwood WWTP (Figure 6-8) shows that the depth of the water is too shallow for laying the pipe on the river bottom and the pipe would need to be buried. In the figure, cover over the pipe is shown at 6 feet. Because of natural resource complications, the relatively long time needed to obtain a permit, and the potential for public resistance, this option was not further investigated.



Redwood Wastewater Treatment Plant
#27-2192-05 02/99

Figure 6-8
Rogue River Crossing
Cross Section

Tunnel (Boring) Under the River

Boring a pipeline under the river would be more feasible (from a permitting standpoint) than open cut crossing. The likely method would be directional drilling; the advantages and disadvantages of this alternative are presented here:

Advantages:

- No disruption to river
- Lower pump head than bridge crossings

Disadvantages:

- Requires geotechnical and bathymetric investigation to choose the best location for the river crossing
- Potentially the most expensive type of crossing
- If large boulders are encountered during the boring, the drill would have to be retracted and reset on a different path.

Use Proposed Pedestrian Bridge Crossing

A ribbon-type pedestrian bridge to be built about 500 feet west of Grants Pass WRP could be adapted to carry the pipeline. The advantages and disadvantages of this alternative are as follows:

Advantages:

- Least expensive type of crossing
- No disruption to river; few or no natural resource permits

Disadvantages:

- Must act quickly to coordinate with the City of Grants Pass to incorporate any necessary modifications to pedestrian bridge
- The bridge design has two high points. The force main at these locations will require air release valves.
- Provide odor control for the air release valves
- Possibility of a break in the force main at the river crossing releasing raw sewage to Rogue River.

006952

Use of SR 199 Bridge

This existing bridge carries motor traffic across the Rogue River west of the Redwood WWTP. The advantages and disadvantages of this alternative are as follows:

Advantages:

- Existing bridge
- No disruption to river
- Air release valves can be more easily located

Disadvantages:

- Longest route (additional 7,000 feet longer than alternative routes)
- High cost due to added length

Because of the high cost of this alternative, it was dropped from further consideration.

Based on the preceding river crossing evaluation, the two most feasible alternatives that were selected for further cost evaluation are 1) attaching force main(s) to the pedestrian bridge and 2) boring force main(s) under the river. An estimate of probable costs for both of these options is included in Appendix H.

Force Main Routes

Once the river crossing alternatives were narrowed down to the two most feasible options, seven alternative force-main routes were evaluated. These seven routes are shown on Figure 6-9 and designated Alternatives 6 through 12. A brief description of each alternative is as follows:

Alternative 6: Dual 12-inch force mains would be bored under the Rogue River just north of Redwood WWTP, then the force mains would follow the right-of-way of Lower River road and Webster Lane to the Grants Pass WRP Plant.

Alternative 7: A 6-inch force main would be placed in an existing sewer interceptor easement between Redwood WWTP and pump station RI-25. From there dual 12-inch mains would be bored under the Rogue River. From the Lathrop boat landing, the force mains would follow the right-of-way of Lower River Road and Webster Lane to the Grants Pass WRP.

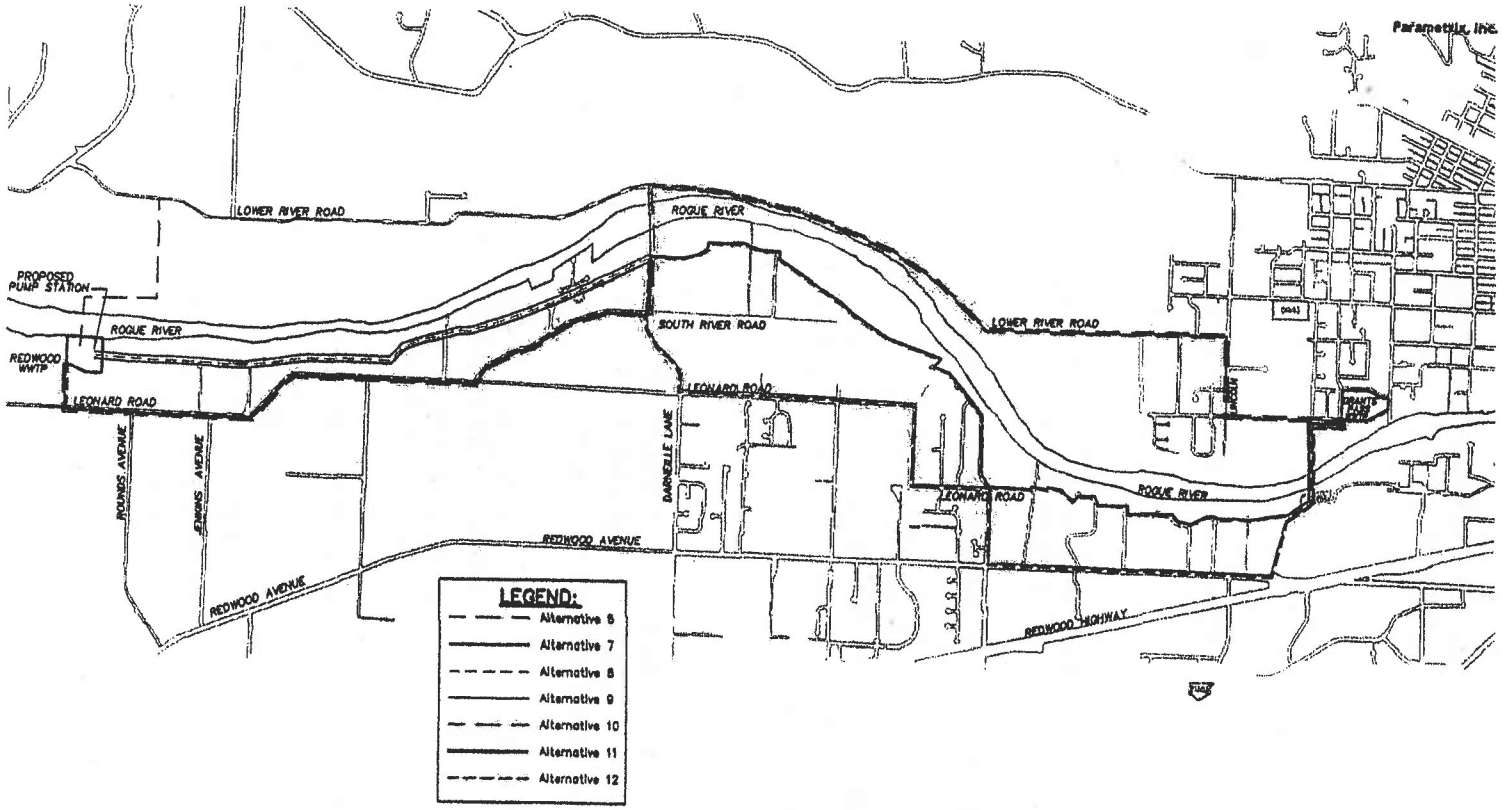


Figure 6-9
Treatment Alternatives
6 Through 12

Alternative 8: The force main would be placed in the existing sewer interceptor easement between Redwood WWTP and the proposed location of the pedestrian bridge near the Grants Pass WRP. A 6-inch main would be installed to pump station RI-25, then dual 12-inch mains from go from there to the Grants Pass WRP. The force mains would be attached to the pedestrian bridge.

Alternative 9: A 6-inch force main would be placed in the existing sewer interceptor easement between Redwood WWTP and pump station RI-25. From there dual 12-inch force mains would follow the right-of-way of Leonard Road and Redwood Avenue to the pedestrian bridge, then to the Grants Pass WRP.

Alternative 10: A 6-inch force main would be placed in the right-of-way of Leonard Road up to Pump Station RI-25. From there dual 12-inch mains would be bored under the Rouge River and follow the right-of-way of Lower River Road and Webster Lane to the Grants Pass WRP.

Alternative 11: A 6-inch force main would be placed in the right-of-way of Leonard Road up to Pump Station RI-25. From there dual 12-inch force mains would be placed in the existing sewer interceptor easement to the pedestrian bridge, and then to the Grants Pass WRP.

Alternative 12: A 6-inch force main would be placed in the right-of-way of Leonard Road up to Pump Station RI-25. From there dual 12-inch force mains would follow the right-of-way of Leonard Road and Redwood Avenue to the pedestrian bridge, then to the Grants Pass WRP.

The existing 20-foot-wide interceptor maintenance easement is wide enough to accept the new force main but not wide enough for the associated construction activity, deliveries, and staging. New temporary construction easements would need to be acquired.

CITY OF GRANTS PASS COMMUNITY DEVELOPMENT DEPARTMENT

PUBLIC WATER AND SEWER SERVICE
COMPREHENSIVE PLAN TEXT AMENDMENT
STAFF REPORT-URBAN AREA PLANNING COMMISSION

Procedure Type:	Type IV: Planning Commission Recommendation and City Council Decision
Project Number:	08-40500002
Project Type:	Comprehensive Plan Text Amendment
Applicant:	City of Grants Pass
Planner Assigned:	Jared Voice / Tom Schauer
Application Received:	April 18, 2008
Application Complete:	April 18, 2008
Date of Staff Report:	June 4, 2008
Date of Planning Commission Hearing:	June 11, 2008

I. PROPOSAL:

The purpose of the proposal is to update the water and sewer sections of Comprehensive Plan Element 10 (*Public Facilities and Services*), including the policies for each, based on recent water and sewer master plans.
See Exhibit 1 for Water Service and ***Exhibit 2 for Sewer Service,*** to replace existing sections within Comprehensive Plan Element 10.
See Exhibit 3 for revisions to Element 10 of Comprehensive Plan Policies Manual.

Additionally, the proposal would adopt the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:

Water
1. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001) ***Exhibit 4***
2. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002) ***Exhibit 5***
3. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MWH/Montgomery Watson Harza, April 2004) ***Exhibit 6***

Sewer
1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001) ***Exhibit 7***
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001) ***See note below***
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004) ***Exhibit 8***
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999) ***Exhibit 9***

NOTE: To conserve resources, copies of the facility plan appendices are not included in the packet. Please contact Jared Voice at the Community Development Department (474-6355) if you would like copies of the appendices made for you. Each full plan may also be viewed at the Public Works office, located on the first floor of the Municipal Building at 101 NW "A" Street.

II. AUTHORITY AND CRITERIA:

Section 13.5.3 of the Grants Pass and Urbanizing Area Comprehensive Plan provides that the City Council may initiate a text amendment.

Sections 13.5.5 and 13.8 of the Comprehensive Plan provide that joint review by the City Council and Board of County Commissioners shall be required for amendment and revision to Comprehensive Plan findings, goals, and policies.

The review shall be in accordance with the procedures of Section 13.8.3 of the Comprehensive Plan, which provides for a recommendation hearing by the Urban Area Planning Commission prior to a joint hearing of the City Council and Board of County Commissioners.

However, with adoption of the 1998 Intergovernmental Agreement, this provision requiring a joint hearing is modified with the result that City Council will make the decision, and the County will have automatic party status, as summarized below:

> Section III of the 1998 Intergovernmental Agreement (IGA) provides for transfer of authority for provision and management of planning services from the County to the City for the Urbanizing Area. It provides:
>
> > The City is hereby vested with the exclusive authority to exercise the County's legislative and quasi-judicial powers, rights, and duties within the Urbanizing Area. .
>
> Section V of the IGA contains provisions pertaining to notification and appeals for quasi-judicial and legislative decisions within the Urbanizing Area. For legislative decisions, the IGA provides:
>
> > The City agrees to provide written notice of all proposed legislative actions to the County at least 45 days prior to the public hearing at which the action is first considered. The County shall be deemed to have automatic party status regarding all such decisions for the purposes of standing for appeals.

Section 13.8.3 of the Comprehensive Plan provides that notice shall be as provided in Section 2.060 of the Development Code for a Type IV procedure. Section 13.8.3 further provides that the hearing shall be conducted in accordance with the Legislative Hearing Guidelines of Section 9 of the Development Code.

Therefore, the application will be processed through a "Type IV" procedure, with a recommendation from the Urban Area Planning Commission and a final decision by City Council. The County has automatic party status for appeals.

The text of the Comprehensive Plan may be recommended for amendment and amended provided the criteria in Section 13.5.4 of the Comprehensive Plan are met.

III. APPEAL PROCEDURE:

The City Council's final decision may be appealed to the State Land Use Board of Appeals (LUBA) as provided in state statutes. A notice of intent to appeal must be filed with LUBA within 21 days of the Council's written decision.

IV. BACKGROUND AND DISCUSSION:

The proposed new documents will replace the existing Water and Sewer sections of the Comprehensive Plan *Public Facilities* element, which were adopted in 1982 and determined service needs within the UGB through the year 2000.

Work on updating the Water and Sewer sections of the Comprehensive Plan Public Facilities element began in 2004, at the request of the City's former Utilities Department Director. The updated Comprehensive Plan element was intended to incorporate data from several recently-completed water and sewer master plans, and adopt the plans by reference.

In May of 2005, City Council adopted the Capital Improvement Programs (CIPs) recommended within the recently-completed water and sewer plans (see Resolution No's 4954 and 4955, attached as ***Exhibits 10 and 11***, corresponding City Council background sheets, attached as ***Exhibits 12 and 13***, and Technical Memorandum from Parametrix dated April 25, 2005, attached as ***Exhibit 14***); however, the plans were not adopted by reference in their entirety, nor were updated Comprehensive Plan *Water Service* and *Sewer Service* sections adopted. The CIPs shown in the technical memo (and water and sewer plans) and adopted by Council are identical to those shown in the currently-proposed water and sewer section updates. Therefore, adoption of the current proposal would not change any of the planned projects for public water and sewer facilities over the twenty-year planning period. Note that several of the items identified within the CIPs have since been completed. Remaining items will continue to be identified for completion within annual budget reports as funds become available.

The proposal incorporates the previously-listed water and sewer facility plans into the Comprehensive Plan, to serve current and future needs within the existing Urban Growth Boundary. Additional facility planning is expected to occur as part of the anticipated Urban Growth Boundary expansion. The current update does not account for areas outside the existing UGB, except for those within special sewer districts (i.e. Redwood Sanitary Sewer Service District, Harbeck Fruitdale Sewer Service District) or which are already committed or planned to receive City water services (i.e. Paradise Ranch, North Valley Industrial Park, etc.)

Throughout the water and sewer service policies, it is stated that the City and County are jointly obligated for managing water and sewer services within the Urbanizing Area (UA). However, under the 1998 Intergovernmental Agreement (IGA), the County agreed to transfer to the City "all of the County's authority to provide and manage... facility financing and development within the UA." Therefore, as long as the IGA is in effect, it is the City's responsibility to adopt and maintain public facility and service plans for the UA, including water and sewer service plans.

V. CONFORMANCE WITH APPLICABLE CRITERIA:

For amending the findings, goals, policies, and Land Use Map of the Comprehensive Plan, the City Council and Board of County Commissioners shall base their conclusions upon, and adopt findings in consideration of, all of the following criteria:

CRITERION (a): Consistency with other findings, goals and policies in the Comprehensive Plan.

> **Staff Response: Satisfied.** The proposal is consistent with other findings, goals and policies in the Comprehensive Plan.
>
> The existing *General Service*, *Water Service* and *Sewer Service* policies would be amended to eliminate language that is out of date, as the "Public Facilities and Services" element of the Comprehensive Plan *Policies Manual* was last amended in 1984. All amended policies are consistent with the goal of Element 10, which is: "To provide needed facilities and services for the Urban Growth Boundary area in a timely, orderly, efficient, economic and coordinated matter."
>
> Other goals and policies within the Comprehensive Plan that relate specifically to water and sewer facilities include the goal of *Element 4: Environmental Resource Quality* ("To maintain and improve the quality of the air, water and land resources of the area."), Policy 4.3 (d) ("The City and County shall affect water quality by increasing the hydraulic capacity of the City's wastewater treatment plant.") and the goal of *Element 8: Economy* ("To improve, expand, diversify and stabilize the economic base of the community.") The proposal is consistent with these goals and policies.

CRITERION (b): A change in circumstances, validated by and supported by the data base or proposed changes to the data base, which would necessitate a change in findings, goals and policies.

> **Staff Response: Satisfied.** The proposed amendment is necessary due to a change in circumstances that is supported by proposed changes to the database. The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated.
>
> The updated findings and policies are validated by and supported by the updated data base. As discussed in the response to the previous criterion, this policy is consistent with other findings, goals and policies found in the Comprehensive Plan.

CRITERION (c): Applicable planning goals and guidelines of the State of Oregon.

> **Staff Response: Satisfied.** The proposed amendment is consistent with applicable planning goals and guidelines of the State of Oregon. Applicable goals and guidelines include Goal 1 (Citizen Involvement), Goal 2 (Land Use Planning), Goal 6 (Air, Water and Land Resources Quality), Goal 11 (Public Facilities and Services), ORS 197.712(2)(e) (which requires a city to adopt a public facility plan for areas within a UGB containing a population of over 2,500) and OAR 660, Division 11 (Public Facilities Planning.)

Goal 1- Citizen Involvement (OAR 660-015-0000(1)): See response to Criterion (d) below.

Goal 2- Land Use Planning (OAR 660-015-0000(2)): Although Goal 2 does not specifically address public facility planning, the explanation of Part I (Planning) of Goal 2 states the following: "All land-use plans and implementation ordinances shall be adopted by the governing body after public hearing and shall be reviewed and, as needed, revised on a periodic cycle to take into account changing public polices and circumstances..." The existing water and sewer sections of the Comprehensive Plan's public facilities element, and associated policies, were adopted in 1982 and are outdated. Revisions to these sections and the associated policies are necessary to account for circumstances that have changed over the past 26 years. Further, subsection (C) of the "Guidelines" for Goal 2 states that: "Inventories and other forms of data are needed as the basis for the policies and other decisions set forth in the plan", including those for "man-made structures and utilities, their location and condition." Existing utilities inventories within the current Comprehensive Plan are not useful for any practical purpose and must be updated to be in full compliance with Goal 2.

Goal 6- Air, Water and Land Resources Quality (OAR 660-015-0000(6)): Goal 6 states that "all waste and process discharges from future development, when combined with such discharges from existing developments shall not threaten to violate, or violate applicable state or federal environmental quality statutes, rules and standards." To assure full compliance with Goal 6, an update of the City's sewer facility plans is necessary.

Goal 11- Public Facilities and Services (OAR 660-015-0000(11)) / ORS 197.712(2)(e) / OAR 660-011-0000: Goal 11 requires "a timely, orderly and efficient arrangement of public facilities and services to serve as a framework for urban and rural development." OAR 660, Division 11 interprets Goal 11 requirements and implements ORS 197.712(2)(e), which requires that a city develop and adopt a public facility plan for areas within a UGB containing a population greater than 2,500.

OAR 660-011-0005 defines public facilities plan as "a support document or documents to a comprehensive plan. The facility plan describes the water, sewer and transportation facilities which are to support the land uses designated in the appropriate acknowledged comprehensive plans within an urban growth boundary containing a population greater than 2,500..." The proposal contains three (3) separate supporting water documents and four (4) separate supporting sewer documents that would be adopted as supporting documents to the Comprehensive Plan. (Note that this proposal amends only the water and wastewater / sewer portion of the City's public facilities plan, and that other existing elements of the public facilities plan, including the Master Transportation Plan (adopted in 1997) and Master Storm Drain Plan (adopted in 1982) are not amended by this proposal.) The proposal also includes an update of the Comprehensive Plan itself, to reflect the supporting data contained in the separate water and sewer facility documents.

OAR 660-011-0010 through -0035 outlines the items that must be contained within a public facility plan. These include an inventory / assessment of existing facilities, a list of projected public facility projects (including rough cost estimates,

locations, and time estimates for each), applicable urban growth management agreements and funding mechanisms. The proposals for both water and sewer contain each of these required items.

The proposed amendment is also consistent with remaining sections of Goal 11 and OAR 660 Division 11, including OARs 660-011-0045 ("Adoption and Amendment Procedures for Public Facility Plans"), 660-011-0060 ("Sewer Service to Rural Lands") and 660-011-0065 ("Water Service to Rural Lands").

ORS 197.610: Notice of the proposed amendment was mailed to the Oregon Department of Land Conservation and Development on April 18, 2008, in accordance with ORS 197.610.

CRITERION (d): Citizen review and comment.

Staff Response: Satisfied. Notice of the proposed amendment has been posted in accordance with the procedure required for a Type IV-B procedure. In addition, the agenda and packet for the June 11th, 2008 Planning Commission meeting was posted on the City's website in advance of the hearing. No written comments have been received from citizens as of the date of the staff report.

CRITERION (e): Review and comment from affected governmental units and other agencies.

Staff Response: Satisfied. 45-day notice was provided to the Department of Land Conservation and Development (DLCD) in accordance with OAR 660 Division 18 and ORS 197.610. OAR 660-18-0035 provides that if DLCD is participating in the proceeding, they shall notify the local government 15 days prior to the first evidentiary hearing. DLCD has not provided notification to the City.

45-day notice was provided to Josephine County in accordance with the 1998 Intergovernmental Agreement for the Urbanizing Area. The County has replied and has no comments regarding the proposal. ***Exhibit 15***

Notice of the proposed amendment was also provided to affected agencies and governmental units, including the Oregon DHS Drinking Water Program, the Oregon Department of Environmental Quality (DEQ), Oregon Watermaster District 14, and Grants Pass Irrigation District. No additional comments regarding the proposal have been received.

CRITERION (f): A demonstration that any additional need for basic urban services (water, sewer, streets, storm drainage, parks, and fire and police protection) is adequately covered by adopted utility plans and service policies, or a proposal for the requisite changes to said utility plans and service policies as a part of the requested Comprehensive Plan amendment.

Staff Response: Satisfied. The proposal does not create the need for any additional basic urban services. The proposed utility plans contain facility improvements and upgrades necessary to serve the water and sewer needs for existing and future residents of the existing Urban Growth Boundary, as well as for a limited number of service areas outside the existing UGB, through at least the year 2020.

CRITERION (g): Additional information as required by the review body.

Staff Response: Satisfied Contingent on Review Body Direction. Additional information will be provided if requested.

CRITERION (h): In lieu of item (b) above, demonstration that the Plan as originally adopted was in error.

Staff Response: Not Applicable. Criterion (b) is applicable. The Plan was not adopted in error. The proposed amendment is being adopted in response to a change in circumstances. See Criterion (b) for discussion of the change in circumstances.

VI. RECOMMENDATION:

Recommend that City Council approve the proposal, which:

1) amends the sewer and water sections of Comprehensive Plan Element 10 by adopting new Comprehensive Plan Sections 10.20 (Water Services) and 10.30 (Sewer Services), attached as ***Exhibits 1 and 2***, and repealing existing Comprehensive Plan Sections 10.20 and 10.30, which were adopted by Ordinance 4471 in 1982, and

2) adopts revisions to the Element 10 (*Public Facilities and Services*) sewer and water policies, attached as ***Exhibit 3***, and

3) adopts the following documents by reference as part of the Public Facilities element of the Comprehensive Plan:
    i. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001, ***Exhibit 4***)
    ii. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002, ***Exhibit 5***)
    iii. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MWH/Montgomery Watson Harza, April 2004, ***Exhibit 6***)
    iv. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001, ***Exhibit 7***)
    v. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices- Final Report* (Parametrix, June 2001)
    vi. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004, ***Exhibit 8***)
    vii. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999, ***Exhibit 9***), and

4) repeals previously-adopted water and wastewater plans which are rendered obsolete by adoption of the new plans.

VII. PLANNING COMMISSION ACTION:

A. Positive Action: Recommend approval of the request:

1. as submitted.
2. as modified by the Planning Commission with the following revisions (list):

B. Negative Action: Recommend denial of the request for the following reasons (list):

C. Postponement: Continue item

1. indefinitely.
2. to a time certain.

NOTE: This is a legislative decision. State law does ***not*** require that a decision be made on the application within 120 days.

VIII. INDEX TO EXHIBITS:

1. Proposed Water Services Section 10.20
2. Proposed Sewer Services Section 10.30
3. Proposed Amended Public Facilities and Services Policies
4. *City of Grants Pass Water Distribution System Master Plan* (West Yost and Associates, January 2001)
5. *City of Grants Pass Water Management and Conservation Plan, Final Report* (West Yost and Associates, June 2002)
6. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report* (MWH / Montgomery Watson Harza, April 2004)
7. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001)
8. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
9. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999)
10. City Council Resolution No. 4954, Adopting Water CIP, 5/9/05
11. City Council Resolution No. 4955, Adopting Wastewater CIP, 5/9/05
12. City Council Background Sheet for Water CIP, 5/4/05
13. City Council Background Sheet for Wastewater CIP, 5/4/05
14. Parametrix Technical Memo Dated 4/25/2005
15. E-mail Message from Josephine County Planning Director, 6/3/08

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10.20 WATER SERVICES INDEX

10.20.1 PURPOSE AND INTENT

- 10.20.1.1 Purpose
- 10.20.1.2 Intent

10.20.2 WATER SOURCES

- 10.20.2.1 Ground Water
- 10.20.2.2 Surface Water

10.20.3 WATER RIGHTS

- 10.20.3.1 Rogue River
- 10.20.3.2 Long-Term Reliable Yield
- 10.20.3.3 Grants Pass Irrigation District
- 10.20.3.4 Savage Rapids Diversion Dam
- 10.20.3.5 Dam Removal

10.20.4 CITY OF GRANTS PASS WATER TREATMENT PLANT, DISTRIBUTION SYSTEM AND WATER DEMAND

- 10.20.4.1 System History
- 10.20.4.2 Service Pressures
- 10.20.4.3 Fire Protection
- 10.20.4.4 Booster Pumping Stations
- 10.20.4.5 Reservoirs
- 10.20.4.6 System Operation
- 10.20.4.7 Water Treatment Plant
- 10.20.4.8 Water Demand Analysis
- 10.20.4.9 Recent Water Use Statistics
- 10.20.4.10 Per Capita Water Demand
- 10.20.4.11 Unaccounted For Water
- 10.20.4.12 Unit Demand Factors by Land Use Pattern
- 10.20.4.13 Peak Hourly Demand
- 10.20.4.14 Historical Peaking Factors
- 10.20.4.15 Future Water Demand

10.20.5 CITY OF GRANTS PASS WATER SYSTEM CAPITAL IMPROVEMENTS PROGRAM (CIP)

10.20.6 PRIVATE WATER UTILITIES

10.20.7 URBAN SERVICE MASTER PLANS AND MANAGEMENT AGREEMENTS FOR WATER

EXHIBIT 1
to UAPC Staff Report 7/65

10.20.8 CAPITAL IMPROVEMENT PROJECT IMPLEMENTATION PLAN AND FUNDING MECHANISMS FOR WATER

- 10.20.8.1 Water Treatment Plant
- 10.20.8.2 Water Distribution System

10.20.9 WATER SERVICE FINDINGS

- 10.20.9.1 Water Source
- 10.20.9.2 Water Treatment
- 10.20.9.3 Water Storage and Distribution

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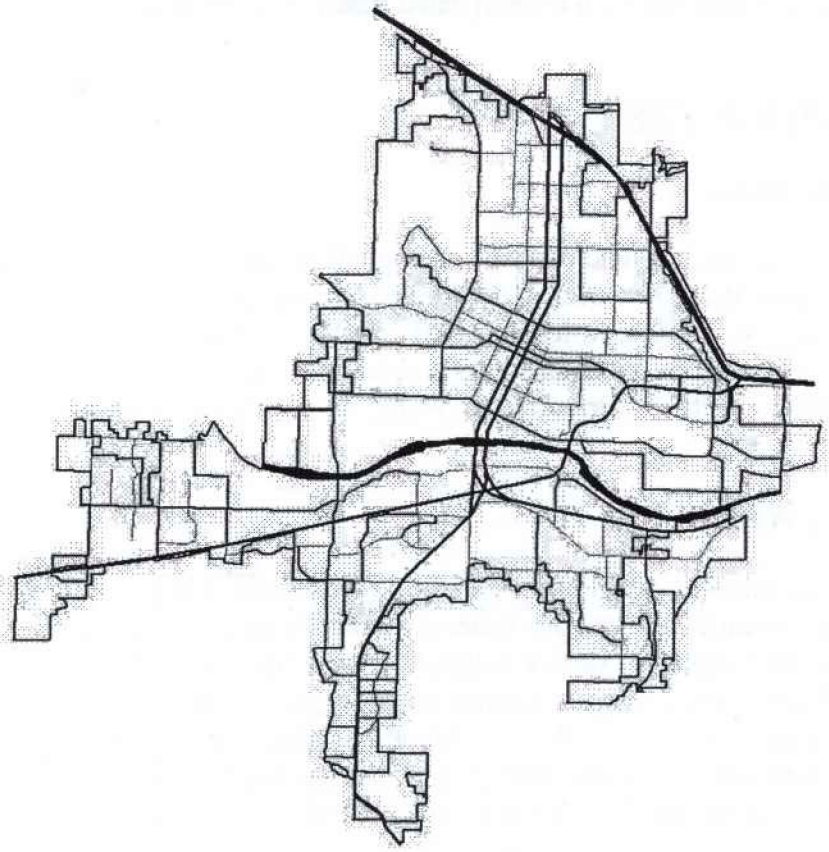
10.20.1 PURPOSE AND INTENT

10.20.1.1 Purpose.

The purpose of this section is to determine the domestic water demand requirements for the build-out of the Urban Growth Boundary (UGB), plus other areas served by municipal water, to assess the ability of the existing municipal water system to meet the projected requirements; to determine what capital improvements are necessary to serve the UGB at built-out, and to approximate costs; to suggest alternative methods for financing the required improvements; and to propose policies for the orderly provision of the required improvements.

Figure 10.20.1
Grants Pass Urban Growth Boundary



10.20.1.2 Intent.

The intent of this section is to enact the following public facilities water system master plans by ordinance as an update to the Public Facilities Element of the City of Grants Pass Comprehensive Plan:

1. *City of Grants Pass Water Management and Conservation Plan, Final Report,* West Yost and Associates, June 2002.
2. *City of Grants Pass Water Treatment Plant Facility Plan, Final Report,* MWH/Montgomery Watson Harza, April 2004.
3. *City of Grants Pass Water Distribution System Master Plan,* West Yost and Associates, January 2001.

NOTE: The Dyer Partnership, Inc. prepared a "Water Master Plan" for Merlin and North Valley in April 2001. The recommended alternative was connection to the City of Grants Pass system. The "Water Master Plan" was adopted by Josephine County as part of its' Merlin / North Valley Community Plan (see Article 101.015 of the Josephine County Rural Land Development Code.) The County has not taken steps to implement the plan, and the City of Grants Pass has not adopted the plan. The City will continue to provide water service to specific properties through individual service agreements; however there are no additional obligations to provide service to properties other than those which are currently served (as of 2008.) The North Valley Industrial Park and some residential uses adjacent to the Merlin Landfill are currently served by City water, and facilities have also been extended to serve the Paradise Ranch Development.

NOTE: Several of the tables within this section contain numbers that have been updated since the above-listed plans were completed. The plans are based on data which was available at the time they were adopted, and have not been updated based on more recent data.

10.20.2 WATER SOURCES

10.20.2.1 Ground Water.

Within the Grants Pass area, an "alluvial deposit" geologic formation is the only reliable source of ground water. In this formation, however, the expected maximum yield from wells of standard construction is 50 gallons per minute, which is insufficient for a municipal supply. Due to lack of adequate quantity, ground water in the Grants Pass area has no potential for municipal use beyond that presently developed. (City of Grants Pass Water System Study, May 1974, Brown and Caldwell)

10.20.2.2 Surface Water.

In the Grants Pass area, surface waters have been in the past, and will continue to be, the only reliable source for large quantities of potable water required for municipal purposes. (Ibid.) The Rogue River is the principal supply of surface water; however, water rights to the Rogue River are limited. The Rogue River drains a large watershed extending from the Cascade Mountains to the Pacific Ocean. Grants Pass is located at approximately River Mile 100 and there are approximately 2,460 square miles of watershed area upstream of the City. As a result of this extensive drainage area, the Rogue River is a plentiful and reliable source of drinking water for the community.

The U.S. Geological Survey (USGS) maintains a river gauging station at the Grants Pass water treatment plant that provides extensive historical data on the flow characteristics of the Rogue River. Because the Lost Creek Reservoir was constructed upstream of Grants Pass in 1977, USGS statistical data for the river are typically based on records from 1978 to the present. Based on USGS data for this station, Table 10.20.2 presents the average, maximum, and minimum monthly flow rates for the Rogue River. Since construction of Lost Creek Reservoir, the lowest daily average flow at Grants Pass was 744 cubic feet per second (cfs) on October 10, 1994, and the lowest seven-day average flow was 799 cfs during the week of September 22, 1994. In general, dry weather flows are maintained by the combination of snow melt from the Cascades in the early summer and the release of stored water from Lost Creek Reservoir in the late summer.[1]

1 Source: Grants Pass Water Management Plan, June 2002, West Yost and Associates, LLC., page 2-1.

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Table 10.20.2 Rogue River Average Monthly Flows at Grants Pass
USGS Data for the 28-Year Period Oct 1978 to Sept 2006

Month	Average Monthly Flow, cfs	Maximum Monthly Flow, cfs	Minimum Monthly Flow, cfs
January	5,094	16,600	1,348
February	4,500	10,960	1,250
March	4,020	8,119	1,099
April	3,950	6,843	1,211
May	3,750	6,428	1,857
June	2,790	4,572	1,549
July	2,120	3,485	1,059
August	2,080	3,080	1,620
September	1,780	2,642	1,333
October	1,450	2,282	1,008
November	2,530	7,669	1,160
December	4,910	17,620	1,557

Source: Grants Pass Water Management Plan, June 2002 (Updated 2/25/2008 by Jason Canady from USGS River Data.)

10.20.3 WATER RIGHTS

10.20.3.1 Rogue River.

The City has four separate permits for diverting water from the Rogue River for municipal use. The first is a "perfected right" of 12.5 cfs, dated 1888. The second and third are permits for 25 cfs each dated 1960 and 1965. The fourth permit, dated 1983, provides for an additional 25 cfs for a total of 87.5 cfs. The City's present water rights and water permits are shown in Table 10.20.3.

As water permits and rights are typically subject to cutbacks under conditions of low flow to serve parties with prior year rights, stored water can allow jurisdictions to augment flow otherwise cut back. Cutbacks during the 1977-78 drought years approached the 1965 level. Without Lost Creek Dam, cutbacks in 1981 would have reached back to the early 1900's, as released water accounted for 50% of stream flow that summer. However, since the City's point of diversion is downstream from the Savage Rapids Dam[2], and cutbacks are established by law as being those jurisdictions and individuals above the dam, there is some question as to whether the City could in fact be cut back, even under low water conditions.

2 Savage Rapids Dam is slated for removal to resolve anadromous fish passage issues. Dam removal will occur once the GPID secures an appropriation of funding from both the federal and state sources. GPID will then pump water from the Rogue River and deliver water to its customers, including those within the City of Grants Pass.

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TABLE 10.20.3
Grants Pass Water Rights

Permit Number	Priority Date	Permitted Use	Permitted Rate, cfs	Available Quantity, cfs	Source Availability Analysis		
					Reliability	Impact of ESA	Water Quality
D15839	1888	Municipal / Irrigation	12.5 cfs		High	Undefined	Good
S26901	1960	Municipal	25.0 cfs	735 cfs*	High	Undefined	Good
S45827	1965	Municipal	25.0 cfs		High	Undefined	Good
S47346	1983	Municipal	25.0 cfs		High	Undefined	Good

* Restriction that water can be diverted only when flow at the mouth of the Rogue exceeds 735 cfs.
Source: Grants Pass Water Management Plan, June 2002

Each Water Right and Permit has a specific geographical area within which the water may be used. The 1888 right stipulates "the city limits," which the City holds to be those city limits as they exist at any point in time. The 1960 permit shows an area approximating the City's 1979 Urban Growth Boundary (20 year expansion); the 1965 and 1983 permits show an area approximating a 50-year expansion.

10.20.3.2 Long-Term Reliable Yield.

Due to the nature of the City's surface water supply source, the long-term sustainability of drinking water supplies for Grants Pass is generally good. As noted earlier, the large size of the watershed drained by the Rogue River typically provides abundant water supplies throughout the year. Even during extreme dry weather periods when river flows are at their lowest, the reliable flow rate in the Rogue River is approximately 750 cfs or nearly fifty times larger than the highest drinking water demand ever experienced in Grants Pass. There are some special circumstances that may affect the long-term reliable yield for the Rogue River. For example, the listing of the Coho Salmon as an endangered species in the Rogue River may influence operational procedures at the Lost Creek Reservoir, which in turn may affect dry weather flow levels. Another issue is related to climate change and snow pack levels in the Cascade Range. Any reduction in average precipitation or the average snow pack will tend to reduce dry weather flow rates in the Rogue River. Since these factors are complex in nature, it is difficult to quantify their potential effect on the river's reliable yield at this time.[3]

10.20.3.3 Grants Pass Irrigation District.

The Grants Pass Irrigation District was organized by the local water users in January 1917. The area of the district was then about 6,000 acres. It originally was planned to irrigate by an extension of the Gravity Canal of the Gold Hill Irrigation District, which was further upstream on the Rogue River and was being organized at the same time. That plan was abandoned in 1920 and the present design was adopted to provide for a direct diversion system with permanent pumping units. The original works were constructed with private funds.

The Savage Rapids Diversion Dam was dedicated November 5, 1921, marking the beginning of the operating history of the district. Settlement and clearing of the undeveloped lands, which constituted a high proportion of the district's area, did not develop to the extent of the expectations upon which

3 Source: Grants Pass Water Management Plan, June 2002, West Yost and Associates, LLC., page 2-2.

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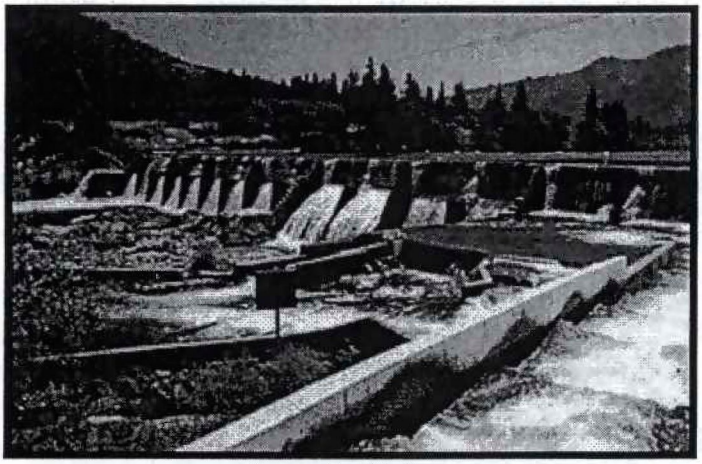
the district was founded and financed. As a result, just over one-half of the irrigable area was in production and therefore carried the entire tax burden.

The Savage Rapids Dam and the Northwest Unit pipeline were badly damaged by a flood in 1927. Emergency repairs were made at that time, but lack of sufficient funds prevented satisfactory completion of the work. The cost of maintenance on the pipeline had become almost prohibitive by 1949.

In 1949, the Bureau of Reclamation was requested to replace the old suspension pipeline and siphon with a new buried line under the Rogue River. Several years later Reclamation was asked to rehabilitate Savage Rapids Dam. After thorough investigations, both requests were undertaken and completed. In 1974, the Bureau of Reclamation and Bureau of Sport Fisheries and Wildlife investigated and prepared a report on anadromous fish passage improvements at Savage Rapids Dam.

On February 12, 1982, the Grants Pass Irrigation District perfected a right to 96.7 cfs with a priority date of 1916. In addition, the District "transports" 83 cfs for the Department of Fish and Game after use as irrigation water for stream enhancements, and was granted a "non-consumptive" right of 800 cfs of pass through water needed to drive the water turbines that lift irrigation water to the canals. The GPID point of diversion is at Savage Rapids Dam, see photo below.



Savage Rapids Dam
Source: U.S. Department of Interior, Bureau of Reclamation

10.20.3.4 Savage Rapids Diversion Dam.

The Savage Rapids Diversion Dam is on the Rogue River 5 miles east of Grants Pass. It is about 456 feet long and consists of a 16-bay spillway section and a hydraulic-driven pumping plant section at the right abutment. Maximum height of the spillway section is about 39 feet. The first seven bays at the right end of the dam are multiple arches with buttresses on 25-foot centers; the remaining nine bays have a concrete gravity section below the gates. Sixteen wooden-faced radial gates originally provided spillway control. Each of them constructed 23 feet wide and 10 feet high. During rehabilitation, the radial gates were replaced with metal stoplogs, and one double-gated river outlet with a capacity of 6,000 cubic feet per second was installed at the center of the dam. During the irrigation season, the stoplogs are used to raise the reservoir elevation 11 feet.

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The Savage Rapids Diversion Dam diverts water from the Rogue River into the South Main Canal to serve the lowlands on the south side of the river. The main pumping plant pumps water from the reservoir to the Tokay Canal to serve lands on the north side of the river, and to the South Highline Canal to irrigate lands above the gravity-type South Main Canal. There are also four lateral relift pumping plants along the canals.

A study by the Bureau of Reclamation (October, 1979) showed that the District served 400 acres zoned exclusive farm use utilized by commercial growers, while serving 7,000 acres of "urban-suburban" lands to irrigate lawns, gardens and pastures. Diversion of water was between 180 to 220 cfs. The Bureau of Reclamation Study indicated that urban-suburban development posed a major problem for canal maintenance and water distribution, being a prime contributor to the District's loss of 2,600 acres of formerly irrigated lands. One of the key factors contributing to maintenance problems is the silting up of those parts of the system from winter runoff, as many of the system canals and laterals also serve to carry storm drainage. The City's Storm Drainage Master Plan calls for a continuance and intensification of this practice, and will require improvements and maintenance coordination between the City and the District. As of 2007, no agreement exists between the City and GPID.

Some 15% of the District's 55 miles of major canals are lined or enclosed in pipe, the rest being unlined. The Bureau of Reclamation recommended either merger of the City and GPID into a water control district (Oregon Revised Statute 553) with both drainage and irrigation services, or some combination of improvements to the canal system to maintain or extend the provision of irrigation water through the system.

A 1974 Grants Pass Water System Study by Brown and Caldwell estimated that the combined water rights of the City and GPID would be sufficient to meet the needs of both the UGB and agricultural users outside the Boundary area.

While discussions with GPID and the Josephine County Water Advisory Board touched on this possibility, the present GPID Board policy is to continue to supply water through the irrigation delivery system, even though development should occur. The District supplies water to almost all of the urbanizing area, and in fact, much of the City, at this time. The District has elected to encourage the continued supply of irrigation water to urbanizing land as development proceeds within the UGB. The District feels this benefits the developer, as the cost of supplying water through a piped system does not often exceed the buy-out cost per acre; benefits the homeowner; and benefits the City, as it saves treatment costs for irrigation water, does not require an increase in the size of the distribution system piping to carry the additional water demand, and reduces the peak per capita usage of water in the summer, thus effectively expanding the capacity of the City's permits.

10.20.3.5 Dam Removal.

In 1995, the Bureau of Reclamation filed a Final Planning Report/Draft Environmental Statement to enhance the salmon and steelhead populations of the Rogue River by removing Savage Rapids Dam. Two pumping plants, one on the north bank and one on the south bank, would be constructed to lift water into Grants Pass Irrigation District's canal system. This plan and environmental law suits resulted in a court order to remove the dam by 2005. To date, the GPID has secured federal and state funding to accomplish dam removal by 2009, including the construction of a new pump station. The uncertainty of these mandated changes has the City and GPID uncertain about any future plans or agreements.

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10.20.4 CITY OF GRANTS PASS WATER TREATMENT PLANT, DISTRIBUTION SYSTEM AND WATER DEMAND

10.20.4.1 System History.

The Grants Pass Water Treatment Plant, located at 821 SE"M" Street, was originally built in 1931 with a single basin and three filters for a designed capacity of approximately 3.5 mgd. The plant has undergone several upgrades and expansions through the years to incrementally adjust to a growing population and more stringent treatment standards, including:

- 1950 – Capacity increased to 9 mgd through the addition of second basin and two additional filters.
- 1961 – Minor improvements to treatment process.
- 1983 – Capacity increased to 20 mgd through addition of third basin and three additional filters, construction of a new raw water intake and new chemical feed systems.
- 1995-2001 – Filter media and gravel support replaced due to suspected gravel/under drain upset caused by excessive air in the backwash line.
- 1997 – Filter-to-waste added for improved CT-removal credit.
- 1998 – SCADA upgrade; VFD included on BW pump.
- 1999-2000 – Improvements to the Equalization basin pumping station.
- 2001 – Liquid sodium hypochlorite system installed to replace gas system.
- 2001 – Riverbank stabilization adjacent to the intake structure, in cooperation with US Army Corps of Engineers.
- 2002 – New PLC-based SCADA system and new monitoring devices were installed in the plant to replace outdated analog transmitters and to allow for more accurate and complete process performance monitoring and automated process control. The PLC replaced obsolete analogue loop-controllers and chemical feed controllers.
- 2006 - Installation of one additional booster pump/ replacement of one aging inefficient booster pump. Reconstruction of the Intake Structure and complete rebuild of the filters and Surface Wash System.

The *City of Grants Pass Water Distribution System Master Plan*, 2001 (West Yost & Associates) inventories and evaluates the performance of the City's water distribution system for critical service standards. This analysis identified system improvements necessary to maintain adequate performance through build-out of the UGB. (**NOTE:** In its' "Future Water Demand" analysis, the plan also acknowledges that there are some properties contiguous to the UGB that are likely candidates for receiving water in the future, as well as some additional areas within the North Valley- see Chapter 3 of the *2001 Water Distribution System Master Plan.*) These identified improvements were developed to either eliminate existing deficiencies in system performance or expand service to satisfy community growth. The elements of the water distribution system that were evaluated include water treatment plant capacity, treated water storage capacity, booster pumping capacity, and pipeline network performance. Table 10.20.4, *Estimated Capital Costs for CIP Projects*, presents the specific costs of reservoir, pump station, and pipeline projects that are targeted for implementation through build-out of the UGB. The costs shown are the City's estimated share of the pipeline extensions and do not include the cost which will need to borne by developers. Timing of projects is based on the City's review of the expansion plans and reflects developer interest and submitted development plans. Some adjustment of timing and priorities should be expected.

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Table 10.20.4 Estimated Capital Costs for CIP Projects

Recommended Improvements	Capital Cost, $1,000
Period 2005-2010	
Pipelines	1,049
Pipeline Replacement	1,124
Total	2,173
Period 2010-2020	
Treated Water Storage	5,720
Pipelines	2,042
Pipeline Replacement	2,248
Total	10,010
Period Post 2020	
Treated Water Storage	1,560
Pump Stations	400
Pipelines	184
Total	2,144
Grand Total	14,327

Source: Grants Pass Water Distribution System Master Plan, January 2001

Per the *2001 Water Distribution System Master Plan*, the Grants Pass water supply system distributes water to developed properties covering an area of more than 3,500 acres. The majority of the properties currently connected to the water distribution system are within the present city limits, although the City does provide service to some areas outside the city limits, including the Redwood and Harbeck-Fruitdale areas within the UGB, as well as parts of North Valley. The overall system is composed of a water treatment plant, thirteen booster-pumping stations, eight reservoirs, three pressure-reducing valves, five altitude valves, and approximately 160 miles of pipelines ranging in size between 1" and 30"(most of which are made of cast iron or ductile iron and range in age up to approximately 80 years).

10.20.4.2 Service Pressures.

The urban growth boundary for the City of Grants Pass encompasses lands of wide ranging elevations. As a result, the water distribution system service area contains eight separate service pressure zones serving the UGB and North Valley (See Pressure Zone Map in the *City of Grants Pass Water Distribution System Master Plan*, West & Yost & Associates, 2001). Table 10.20.6 summarizes the service elevations and static range for each pressure zone. The lower end of the pressure range is based on reservoirs at 80 percent full and the upper end is based on full reservoirs. At this time, there are properties receiving City water service in each of the pressure zones except Zone 5.

In some areas, the pressure zone boundaries are modified slightly from these elevation ranges in order to accommodate special service pressure requirements. Pressure Zone 2A is a hybrid between Zones 1 and 2, as is the Rogue Community College's Zone 2B. The North Valley service area spans pressure Zones 1, 2, and 3, serving properties between the elevations of 995 and 1,165 feet. Due to the great range of elevations served in the North Valley, this pressure zone requires pressure reduction valves at service connections to maintain appropriate service pressures. Table 10.20.5 displays the water distribution system pipeline sizes and lengths.

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Table 10.20.5
Water Distribution System Pipeline Network

Pipe Size (inches)	Length (miles)
2	5.23
4	1.80
6	40.89
8	43.63
10	7.64
12	19.49
14	0.38
16	7.88
20	2.40
24	1.02
30	0.95
36	0.01
	131.32

Table 10.20.6 Pressure Zone Ranges

Zone	Elevation (feet)	Pressure (psi)
1	900 - 1,020	36 – 90
2	1,020 – 1,140	41 – 95
2A	960 – 1,035	61 – 94
2B	1,000 – 1,060	35 – 60[a]
3	1,140 – 1,280	36 – 100
4	1,280 – 1,420	42 – 104
5	1,420 – 1,560	41 – 104
NV	995 – 1,165	101 - 177

Source: Grants Pass Water Distribution System Master Plan, January 2001

10.20.4.3 Fire Protection.

The Grants Pass Department of Public Safety provides fire protection for properties within the City, and some properties with Service and Annexation Agreements within the Urban Growth Boundary. Since the water distribution system is an integral part of the City's fire protection system, the Department of Public Safety has adopted the Oregon Fire Code recommendations as the required fire flows for the various land use classifications within the City. These fire protection requirements are discussed in detail in Chapter 4, "*Water Distribution System Service Standards,*" *Grants Pass Water Distribution System Master Plan,* January 2001.

10.20.4.4 Booster Pumping Stations.

The water distribution system includes the water treatment plant pumps and thirteen booster pumping stations that transfer water to the higher pressure zones. These pump stations either fill the reservoirs that serve these higher pressure zones or pump to maintain a minimum pressure in those areas that are not served by reservoirs. On the following page, Table 10.20.7 details the technical information for each of the system's pumping stations.

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Table 10.20.7 Existing Booster Pumping Stations

Pump Station Location	Service Level	Feeds Reservoir	Number of Pumps	Horsepower and RPM	Rated Capacity	Rated Head (TDH)
Lawnridge	2	Yes (6)	4	1 – 25HP @ 1750 RPM 2 – 50 HP @ 1750 RPM 3 – 50 HP @ 1750 RPM 4 – 100 HP @ 1750 RPM	400 GPM 1000 GPM 1000 GPM 2000 GPM	120 TDH 120 TDH 120 TDH 148 TDH
Madrone	2	Yes (4)	3	1 – 60 HP @ 1750 RPM 2 – 30 HP @ 1750 RPM 3 – 40 HP @ 1750 RPM	2000 GPM 900 GPM 1200 GPM	170 TDH 170 TDH 170 TDH
Champion	3	Yes (8)	3	1 – 100 HP @ 1750 RPM 2 – 150 HP @ 1750 RPM 3 – 50 HP @ 1750 RPM	1600 GPM 2300 GPM 800 GPM	165 TDH 165 TDH 165 TDH
Hefley	3 & 4	Yes (13)	4	1 – 7.5 HP @ 3500 RPM 2 – 15 HP @ 3500 RPM 3 – 60 HP @ 3500 RPM 4 – 60 HP @ 3500 RPM	40 GPM 120 GPM 600 GPM 600 GPM	250 TDH 250 TDH 300 TDH 300 TDH
Starlite	3	No	5	1 – 5 HP @ 3461 RPM 2A – 7.5 HP @ 3525 RPM 2B – 7.5 HP @ 3525 RPM 3 – 60 HP @ 1760 RPM 4 – 30 HP @1760 RPM	30 GPM 84 GPM 84 GPM 1050 GPM 450 GPM	315 TDH 208 TDH 208 TDH 185 TDH 185 TDH
North Valley	5	Yes (15)	3	1 – 7.5 HP @ 3500 RPM 2 – 30 HP @ 3500 RPM 3 – 30 HP @ 3500 RPM	70 GPM 500 GPM 500 GPM	170 TDH 174 TDH 174 TDH
Harbeck	2	No	3	1 – 5 HP @ 3600 RPM 2 – 5 HP @ 3600 RPM 3 – 50 HP @ 3600 RPM	90 GPM 90 GPM 1200 GPM	112 TDH 112 TDH 125 TDH
Hilltop	2	No	3	1 – 5 HP @ 3600 RPM 2 – 7.5 HP @ 3600 RPM 3 – 40 HP @ 3600 RPM	100 GPM 150 GPM 750 GPM	120 TDH 120 TDH 120 TDH
New Hope	2	No	4	1 – 30 HP @ 3600 RPM 2 – 30 HP @ 3600 RPM 3 – 30 HP @ 3600 RPM 4 – 150 HP @ 1800 RPM 5A – 5 HP @ 3600 RPM 5B – 5 HP @ 3600 RPM	350 GPM 350 GPM 350 GPM 2000 GPM 53 GPM 53 GPM	212 TDH 212 TDH 212 TDH 200 TDH 227 TDH 227 TDH
Laurel Ridge	3	No	3	1 – 15 HP @ 3500 RPM 2 – 15 HP @ 3500 RPM 3 – 60 HP @ 3500 RPM	350 GPM 350 GPM 1100 GPM	127 TDH 127 TDH 160 TDH
Meadow Wood	2 & 3	No	6*	1 – 7.5 HP @ 3500 RPM 2A – 15 HP @ 3500 RPM 2B – 60 HP @ 3500 RPM 2C – 60 HP @ 3500 RPM 3 – 100 HP @ 3500 RPM 4 – 7.5 HP @ 3500 RPM	80 GPM 150 GPM 500 GPM 500 GPM 1500 GPM 63 GPM	280 TDH 155 TDH 275 TDH 275 TDH 150 TDH 341 TDH
Williams Crossing	3	No	2	1 – 5 HP @ 3500 RPM 2 – 5 HP @ 3500 RPM	70 GPM 70 GPM	145 TDH 145 TDH
Panoramic Loop	3	No	4	1 – 25 HP @ 3600 RPM 2 – 10 HP @ 3600 RPM 3 – 60 HP @ 3600 RPM 4 – 60 HP @ 1800 RPM	70 GPM 150 GPM 1000 GPM 1000 GPM	316 TDH 138 TDH 157 TDH 157 TDH
Treatment Plant	1	Yes	6	1 – 250 HP @ 1780 RPM 2 – 300 HP @ 1775 RPM 3 – 250 HP @ 1780 RPM 3A – 250 HP @ 1780 RPM 4 – 250 HP @ 1780 RPM 5 – 200 HP @ 1780 RPM	3500 GPM 3500 GPM 3500 GPM 3500 GPM 3500 GPM 2600 GPM	210 TDH 220 TDH 210 TDH 210 TDH 210 TDH 210 TDH

* Pumps 1, 2A and 3 service Zone 2 exclusively. Pump 4 services Zone 3 exclusively. Pumps 2B and 2C service both Zones 2 and 3.
Data source: Jason Canady- City of Grants Pass Water Treatment Plant Supervisor March 2008

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10.20.4.5 Reservoirs.

There are eight water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999. Design information for these reservoirs is detailed in Table 10.20.8.

Table 10.20.8 Existing Reservoirs

Reservoir Location	Reservoir Number	Pressure Zone Served	Year Built	Construction Materials	Capacity (mg)	Bottom Elevation (ft)	Overflow Elevation (ft)
500 Block Woodson Drive	3	1	1946	Concrete	3.5	1,089.5	1,108.5
1500 Block Ridge Road	4	2	1953	Concrete	0.75	1,216	1,240
1400 Block Sherman Lane	5	1	1983	Concrete	3.5	1,079.5	1,108.5
2200 Block Crown Street	6	2	1982	Concrete	3.5	1,211	1,240
Heiglen Loop Road	8	3	1983	Concrete	2.0	1,341	1,370
1420 Denton Trail	11	1	1999	Concrete	4.5	1,080.1	1,108.5
1700 Block Sunset Lane	13	4	1980	Concrete	0.08	1,510	1,520
3900 Block Highland Ave.	15	4	1985	Concrete	1.2	1,374	1,403

Source: Grants Pass Water Distribution System Master Plan, January 2001

10.20.4.6 System Operation.

The general procedures for operation of the Grants Pass water distribution system are discussed in the following:

- The water treatment plant operates as necessary to fill storage reservoirs in the distribution system on a daily basis. Therefore, the operating schedule adjusts to seasonal variations in water demand. During winter months, the plant generally operates seven days per week for an eight-hour period. Operational hours are extended during the high demand summer months, when the plant must operate up to twenty four hours a day in order to keep the storage reservoirs full.
- Those booster pumping stations that fill storage reservoirs are automatically controlled to maintain preset water levels. When sensors show that the water level of a reservoir has fallen below a preset threshold, the lead pump will activate and begin filling the reservoir to a high water level. If water demand on the reservoir is such that a single pump cannot maintain the water level, a lag pump (or pumps) will activate as necessary until the reservoir fills to a high water level.
- Those booster pumping stations that serve areas without storage reservoirs are automatically controlled to maintain a minimum discharge pressure at the pumping stations. When pressure sensors show that the discharge pressure has fallen below a preset threshold, the lead pump activates and pumps until the discharge pressure exceeds a high pressure level. If water demand in the pump station's service area is such that a single pump cannot maintain the pressure level, a lag pump (or pumps) will activate as necessary until the system pressure is restored.
- The reservoirs in the water distribution system are generally maintained between 80 and 100 percent full. This fluctuating volume represents the operating storage. The remaining storage is allocated to providing fire flow requirements and emergency reserves. In the case of Reservoir

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No. 15 in the North Valley, water levels are maintained at a much lower level due to limited demand in that portion of the distribution system. Altitude valves control the flow into and out of Reservoirs No. 3, No. 4, No. 5, No. 6, and No. 11. These valves are designed to close when the reservoir is full and open when the system pressure drops. The other reservoirs in the distribution system float on the system.

- There are three pressure reducing valve stations in the Grants Pass water distribution system. Two of the stations control the flow of water from Pressure Zone 2 to Pressure Zone 2A. Pressure Zone 2A extends to slightly lower elevations than Pressure Zone 2 and thus requires some pressure reduction. Each station contains a single pressure-reducing valve (one is a 10-inch valve and one is a 6-inch valve). The third pressure reducing valve station on NE Beacon Drive reduces Zone 4 water to Zone 3.
- The City upgraded the water distribution system to a Supervisory Control and Data Acquisition (SCADA) system in 1999. SCADA system monitors reservoir levels, pump operating status, and local pressures throughout the system. The central computer system for the man-machine interface is located at the water treatment plant.

10.20.4.7 Water Treatment Plant.

The City of Grants Pass Water Treatment Plant (WTP) has successfully met the City's drinking water needs for over 70 years. The Rogue River supply is typical of many Pacific Northwest surface waters with low mineral content, low pathogen concentrations, and normally low turbidity, but with seasonal increases in turbidity due to precipitation and runoff. The Rogue River quality and flow is also influenced by the operation of upstream reservoirs including Lost Creak Reservoir and Savage Rapids Dam (slated for removal). Peak withdrawals by the WTP to meet demands in the summer months coincide with minimum river flows and low turbidities.

The WTP's main purposes include removal of suspended particulates, removal and inactivation of pathogens, and production of non-corrosive, palatable water in accordance with Federal and State drinking water regulations. The plant has historically met all regulations and the few customer complaints are limited to occasional chlorinous tastes and odors. The plant appears well positioned to continue to meet current and future drinking water needs.

The plant's production has steadily increased over the last decade in response to increasing water demands within the City's service area. The City's service area has been expanding as areas previously served by small groundwater systems have been incorporated into the City's water system. Significant investments have been made to upgrade the distribution and storage systems over the past few years. Water production at the plant has increased by approximately 20% since 1995. In 2007, peak day water production was 11.87 mgd, and the average annual production was 5.85 mgd.

The plant has rated maximum capacity of 20 mgd with all raw water and finished water pumps in operation. The reliable plant capacity is approximately 15 mgd with one of the largest pumps out of service. The plant is operated in a start/stop mode each day, with the hours of production varying between 8 to 24 hours per day depending on demands and raw water quality. The plant occasionally operates at the peak production rate of 20 mgd (14,000 gpm) during the high demand season. Recently the plant has had to increase its operating staff to allow for 24-hour operations, to more reliably meet demand during peak summer season.

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The *Water Treatment Plant Facilities Plan*, April 2004 (WTPFP) provides guidance for improving this major element of the City's water system and recommends a capital improvement program (CIP) that will meet the City's water treatment needs for 20 to 25 years. Based on a water demand increase of 2.5 to 3 percent per year, it is expected that the plant will continue to be able to meet the City's water needs for at least the next 20 years, with some modifications and improvements. A major plant expansion is not envisioned until the middle to end of decade 2020. Although the existing plant site is extremely confined, the plant is capable of being expanded to approximately 30 mgd with major modifications. The plant expansion to provide 30 mgd of treatment would be required in 25 years (on-line by 2028) if demand growth is steady at 2.5 percent per year. The expansion would be required in 20 years (on-line by 2023) if demand growth is steady at 3 percent per year. The existing plant structures appear to have significant remaining useful life. However, the older plant structures are vulnerable to damage during a severe seismic event.

While the plant has been able to successfully meet the City's water demands and also produce good water quality, the *Water Treatment Plant Facility Plan* identified the following challenging issues which have regulatory compliance implications and which create production inefficiencies. Note that the WTPFP was adopted in 2004 and since then measures have been taken to address the following issues:

- The existing Rogue River intake does not comply with current Endangered Species Act (ESA) regulations to protect juvenile fish including salmonids, due to high approach velocities and screen deficiencies. **NOTE:** This problem has since been fixed. Completed in 2006, the intake structure screen was replaced and the intake inlet was increased in size to reduce intake velocity.[4]
- The backwash/sludge holding pond is completely full of solids and immediate action is required, including development of a long-term solids management plan, to ensure continued compliance with the City's NPDES permit for discharge to Skunk Creek. **NOTE:** A master plan for a long-term solution to this problem was being developed as of September 2007. Short-term solids handling procedures are in place until the plan is completed.[4]
- The filter media is in a degraded condition and the plant (specifically the filters and sedimentation basins) is operating inefficiently requiring frequent backwashing and excessive raw water pumping, resulting in higher operating costs. **NOTE:** The filter media has been upgraded and the resulting operational inefficiency is no longer a problem.[4]
- Proposed drinking water regulations, including the Disinfectants/Disinfection By-Products Rule and the Long-Term 2 Enhanced Surface Water Treatment Rule, have the potential to require significant plant modifications depending on the outcome of monitoring programs. **NOTE:** Increased monitoring programs will be underway in the near future. Depending on the outcome of these programs, plant modifications may be required to maintain drinking water compliance.[5]

These and other challenges require the City to implement near-term improvements to the plant. The plant also requires a longer-term capital improvement program to ensure reliability and redundancy of major equipment, including adding new equipment, replacement/repair of major equipment as it becomes less reliable, and to prepare for a major plant expansion.

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4 Per Dave Wright and Joey Wright, City of Grants Pass Public Works Department, September 2007

5 Per Jason Canady, City of Grants Pass Water Treatment Plant Supervisor, September 2007

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10.20.4.8 Water Demand Analysis.

Analysis of the City of Grants Pass historic water production and demand data allows for the identification of the unique water use patterns that characterize the City and provides a basis for estimating future water demand in the community. Additional analysis relates the various measures of water demand (maximum monthly demand, maximum daily demand, and peak hour demand) to the average annual demand through the use of peaking factors.

The projection of future water demand is based on unit demand factors developed by land use type and corresponding customer classifications. These future demand projections provide the basis for assessing the adequacy of the existing water production and distribution system and planning for future improvements.

10.20.4.9 Recent Water Use Statistics.

There are several measures of water use that are important to analyze during the development of the water master plan. Following is a description of the influential water demand factors that will guide planning decisions with respect to the Grants Pass water systems:

- Annual average demand – A measure of the amount of water that must be obtained from the available sources of supply on an annual basis.
- Monthly average demand – Review of monthly average water demand illustrates seasonal variations in demand due to such factors as climate, irrigation, industrial production, and domestic use patterns.
- Maximum day demand – The maximum daily water demand is used to size booster pumping stations that serve areas with storage reservoirs. This measure of demand is also used along with fire demands to size storage reservoirs.
- Peak hour demand – The peak hour water demand is used to size pipelines and booster pumping stations that serve pressure zones without reservoirs.

Three water usage rate variations are generally used in the design of water system facilities. These are the average annual demand, maximum day demand and peak hour demand. The annual average, monthly average, and maximum day water demand are calculated from analyzing WTP daily operational data. The analysis allows for the identification of annual average, monthly average, and maximum day water demand based on the period from 1995 to 1999. The average annual demand increased from 3.73 mgd to 4.50 mgd. The highest peak daily demand was 9.47 in August of 1998.6

10.20.4.10 Per Capita Water Demand.

Per capita water demand is a useful demand measure that is derived from historical data. However, it was not used in determining demand in the *2001 Water Distribution Plan* (which uses land use-based demand) or *2004 Water Treament Plant Facility Plan* (which is based on a water demand growth rate factor.) Table 10.20.9 presents the population for Grants Pass along with the average annual demand during the years 1995 to 1999, which allows for calculation of the average demand in gallons per capita per day (gpcd). Ranging from 190 to 215 gpcd, the average daily water demand for the years 1995 to 1999 is 202 gpcd. Note that this unit demand factor is based on water production and includes all uses: residential, commercial, industrial, public/institutional, and

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6 Data excerpted from *2001 Water Distribution System Master Plan*. Water use has since increased substantially. See Table 10.20.16 for more recent water use data.

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unaccounted. The variation in per capita demand for the different years reflects the regular variation in water use patterns caused by seasonal conditions and possible changes in end user demand characteristics.

For reference, an engineering report of the water distribution system, prepared by CH2M Hill in 1979, identified an average annual demand of 253 gpcd.

Table 10.20.9 Grants Pass Water Use for 1995 to 1999, gpcd*

Year	Population (City Limits)	Average Demand, mgd	Average Demand, gpcd
1995	19,660	3.73	190
1996	20,255	4.11	203
1997	20,535	3.97	193
1998	20,590	4.17	203
1999	20,935	4.50	215
2000	23,170	4.55	196
2001	23,670	4.86	205
2002	23,870	4.98	209
2003	24,470	4.81	197
2004	24,790	5.00	202
2005	26,085	4.75	182
2006	30,930	5.26	170
2007	31,740	5.84	184
13-Year Average	--	5.66	196

Source: For years 1995-1999, Grants Pass Water Distribution System Master Plan, January 2001;
For years 2000-2007, updated 4/2008 by Jason Canady and Jared Voice using population estimates from Portland State University's Population Research Center)

*Demands include all uses, including residential, commercial, industrial, and public/institutional.

NOTE: Not all City residents are connected to the municipal water system. There are also users outside of the City limits that are not included in the population data in this table. Therefore, although the demand for all users is shown, the per capita demand shown likely overstates actual per capita use.

10.20.4.11 Unaccounted for Water.

All water distribution systems experience losses of water during transmission from the treatment plant to the end user. These losses, known as unaccounted for water, result from many situations including un-metered customers, transmission system leaks, main breaks, faulty meters, fire fighting activities, system flushing, and other miscellaneous hydrant uses. Thus, the total volume of water metered for all end users is always somewhat less than the volume of water produced at the WTP.

Since the City meters water use for all customers, a comparison of water billing records and treatment plant production data provides a good estimate of the volume of unaccounted for water in the system. Table 10.20.10 shows the estimated volume of unaccounted for water in millions of gallons and also as a percentage of total production during the period of 1998 through 2000. Since a water loss rate of 10 to 15 percent is considered good, the calculated unaccounted for water rate indicates that the distribution system is in good condition. The City is also conducting several programs that will reduce the unaccounted for water rate. The programs include residential meter replacements, commercial meter upgrades, and improved monitoring of hydrant use.

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Table 10.20.10 Unaccounted for Water; 1998-2000

Year	Million Gallons	Percent of Total Water Production
1998	146	9.6%
1999	190	11.6%
2000	177	10.9%

Source: Grants Pass Water Management Plan, June 2002

10.20.4.12 Unit Demand Factors by Land Use Pattern.

Water demand factors related to land use patterns are used to analyze water demand in the community. Based on historical billing data provided by the City's Utilities Department for 1998 and 1999, Table 10.20.11 shows water demand within each land use pattern classification: commercial, single family residential, and multi-family residential showing the annual average and percent of total annual average demand for both years. The commercial classification includes general business, industrial, institutional and governmental-public land use categories. As indicated in the percentage summary of annual average demand by land use, the single-family residential classification accounts for nearly half of the water used in Grants Pass.

Table 10.20.11 Grants Pass Water Use by Customer Class for 1998 and 1999

	Demand			
	Commercial	Multi-Family Residential	Singe-Family Residential	Total
1998 Annual Average	1.36	0.61	1.80	3.77
Percent of total annual average demand	36.0%	16.2%	47.8%	100%
1999 Annual Average	1.40	0.67	1.91	3.98
Percent of total annual average demand	35.2%	16.8%	48.0	100%

Source: Grants Pass Water Management Plan, June 2002

To develop a unit demand factor for the three different land use patterns, the water demand data presented in Table 10.20.11, above, are combined with estimated areas for each of the land use classifications. Table 10.20.12, below, summarizes the acreages by land use classification and pressure zone for all areas receiving water service from the City of Grants Pass, including those within the Urban Growth Boundary and North Valley. This summary was derived from analysis of a distribution system for Pressure Zones 1 through 4. The acreage for Pressure Zone NV is based on an estimate of the properties connected to the system in the North Valley area. The quotient of water demand and acreage yields a unit demand factor for each land use classification in gallons per acre per day (gpad), as presented in Table 10.20.13 below. Table 10.20.11 above does not include unaccounted for water, the calculation of these unit demand factors includes an allowance for unaccounted for water.

Table 10.20.12 Land Use by Pressure Zone (Acres)

	Customer/Class	PZ/1	PZ/2	PZ/3	PZ/4	PZ/NV	Total	%
	Commercial	924	125	57	0	40	1,146	32%
	Residential	1,197	594	114	67	5	1,977	56%
	Multi-Family	358	42	35	0	0	435	12%
Total		2,479	761	206	67	45	3,558	100%
%		70%	21%	6%	2%	1%	100%	

*The Commercial customer class includes commercial, industrial, and public connections to the system.
Source: Grants Pass Water Distribution System Master Plan, January 2001

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Table 10.20.13 Unit Demand by Customer Class

Customer Class	1999 Average Demand (mgd)	Land Use Area (Acres)	Average Unit Demand (gal/acre day)
Commercial/Industrial/Public	1.56	1,146	1,400
Multi-Family Residential	0.75	435	1,700
Single-Family Residential	2.13	1,977	1,100

*The 1999 average demand is based on billing records plus 11.6 percent to reflect unaccounted for water.
Source: Grants Pass Water Management Plan, June 2002

10.20.4.13 Peak Hourly Demand.

The peak hour demand on the water distribution system typically occurs during the hottest, driest period of the year when customers are heavily irrigating landscaped yards and parks. For the City of Grants Pass service area, the peak hour demand usually happens in the month of August during the peak day demand. In order to evaluate this peak hour demand, hourly water level data was collected from each of the reservoirs in the distribution system during the summer of 1999. This data was analyzed in combination with the water production rate for the water treatment plant to identify the peak hour demand on each day for which data was available. Based on this analysis, the peak hour demand is estimated to be 4.5 times the average annual demand.

10.20.4.14 Historical Peaking Factors.

The water demands observed over the five years from 1995-1999 can also be expressed as a ratio to the annual average demand known as peaking factors. Although peaking factors vary significantly from user to user, these historical peaking factors are useful for comparing system-wide water use patterns in Grants Pass to other communities and for projecting future water use patterns. Table 10.20.14 identifies Grants Pass water demand patterns for 1995 to 1999 and shows peaking factors based on ratios to annual average demand. The identified peak hour demands for the system are not actual measurements, but rather estimates derived from the reservoir level analysis conducted during the summer of 1999 as described above. Tables 10.20.14 and 10.20.15 below summarize the average peaking factors for the system. These values are fairly typical for a Western Oregon community. In general, they are slightly higher than the peaking factors for Corvallis and slightly lower than values for Portland.

Table 10.20.14 Grants Pass Maximum Month, Peak Day, and Peak Hour Demand Ratio

Year	Annual Average Demand, (mgd)	Maximum Month Demand, (mgd)	Peak Day Demand (mgd)	Peak Hour Demand, (mgd)	Ratio of Maximum Month to Annual Average Demand	Ratio of Peak Day to Annual Average Demand	Ratio of peak Hour to Annual Average Demand
1995	3.73	6.48	8.32	17.01	1.74	2.23	4.56
1996	4.11	7.22	9.09	18.60	1.76	2.21	4.52
1997	3.97	6.82	8.83	18.10	1.72	2.22	4.56
1998	4.17	7.62	9.47	19.40	1.83	2.27	4.66
1999	4.50	7.79	9.35	19.20	1.73	2.08	4.26
5 Yr. Avg.	4.10	7.18	9.01	18.46	1.76	2.20	4.51

Source: Grants Pass Water Distribution System Master Plan, January 2001

Table 10.20.15 Peaking Factor Summary

Description	Factor
Maximum month demand	1.8
Maximum daily demand Average for city	2.2
Peak hourly demand Average for city	4.5

NOTE: The average demand multiplied by the peaking factor yields the respective peak demand.
Source: Grants Pass Water Distribution System Master Plan, January 2001

The following table identifies Grants Pass Water demand patterns for the years 1995 through 2006. The data used in this table was obtained from the City Water Treatment Plant supervisor in 2007 to provide updated information that was not included in the previous planning documents referenced in this chapter. This more recent data shows that water demand has continued to increase since the previous plans were completed. Note that the more recent data was not contained in the *2001 Water Distribution System Master Plan*, the *2002 Water Management Plan* or the *2004 Water Treatment Plant Facility Plan*. The capital improvement and implementation plans developed in these documents and referenced later in this chapter pre-dated this more recent data.

Table 10.20.16 Grants Pass Maximum Month, Peak Day and Peak Hour Data, 1995-2006

Year	Annual Average Flow (mgd)	Maximum Month Demand (mgd)	Peak Day Demand (mgd)	Peak Hour Demand (mgd)
	Finished	Finished	Finished	Finished
1995	3.93	6.48	8.32	17.69
1996	4.23	7.22	9.09	19.04
1997	4.03	6.82	8.83	18.14
1998	4.24	7.62	9.47	19.08
1999	4.56	7.79	9.35	20.52
2000	4.55	7.82	9.73	20.48
2001	4.86	7.74	9.25	21.87
2002	4.98	8.88	10.54	22.41
2003	4.81	9.21	10.31	21.65
2004	5.00	9.33	10.17	22.50
2005	4.75	9.57	10.52	21.38
2006	5.27	9.55	11.69	23.72
12-Yr. Avg.	4.60	8.17	9.77	20.71

NOTE: Annual average flow, maximum month demand and peak day demand obtained from Water Treatment Plant data; Peak hour demand is an estimate calculated by multiplying the average flow by the actual 1995 to 1999 peaking factor average of 4.5 (see Tables 10.20.14 & 10.20.15)
Data Source: Jason Canady, City of Grants Pass Water Treatment Plant Supervisor

10.20.4.15 Future Water Demand.

NOTE: The information presented in this section was excerpted from the *Grants Pass Water Distribution System Master Plan*, prepared by West Yost and Associates in January 2001.

Committed Service Areas.
The land use demand factors developed in the previous sections provide a basis for projecting future water demand in the Grants Pass service area. The land use demand factors can be used in conjunction with land development projections to estimate water demand.

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Although the timing of land use development within the UGB is unknown, information is available regarding the current zoning designation for all properties within the UGB. Table 10.20.17 summarizes the acreage of properties with the UGB according to land use, differentiating between properties that are were receiving water service as of January 2001 and those that will connect to the water distribution system in the future. Using the unit demand factors developed for these land use classifications, the table also projects average annual water demand at the UGB build-out condition. This analysis assumes the existing mix of residential/commercial properties will stay the same in the future.

Table 10.20.17 Land Use Based Water Demand Projections for UGB Build-Out

Land Use	Existing Acreage	Future Acreage	Total Acreage	Unit Demand, gallons/acre-day	Estimated Average Annual Demand, mgd
Commercial	1,146	598	1,744	1,400	2.4
Single-Family Residential	1,977	2,419	4,396	1,100	4.8
Multi-Family Residential	435	440	875	1,700	1.5
Total	3,558	3,457	7,015	4,200	8.7

Source: Grants Pass Water Distribution System Master Plan, January 2001

Based on this estimate of the UGB build-out average annual demand, the future maximum month, maximum day, and peak hour demand can be estimated using historical peaking factors. Table 10.20.18 summarizes water demand at the time of the *2001 Water Distribution System Master Plan* and projections for the build-out condition.

Table 10.20.18 Water Demand Summary for Grants Pass

	Current Water Demand, mgd	Future Total Water Demand, mgd
Average Annual	5.85	9
Maximum Month	10.13	16
Maximum Day	11.87	20
Peak Hour	23.72	40

Source: Grants Pass Water Distribution System Master Plan, January 2001 (Updated Jason Canady, 3/08)

Potential Service Areas Independent of UGB.

The City has also evaluated its potential to serve properties outside the Urban Growth Boundary, mainly related to the existing distribution system serving the Merlin / North Valley area, and some additional areas contiguous to the UGB.

One location with potential for future water system expansion is Merlin / North Valley. A portion of this area, including the North Valley Industrial Park and several residents along Merlin Road, is being serving by a service connection to the City at NW Highland Ave. near NW Vine St. (as of 2008.) The North Valley pump station was constructed at this connection to pump water from the City's Pressure Zone 3 to fill a 1.2 million gallon reservoir though an 8-inch pipeline in Highland Ave. The reservoir is located approximately one mile north of the North Valley pump station. The water coming out of the storage tank then feeds the Merlin / North Valley system by gravity through a 1.6-mile long 16-inch pipeline. The North Valley pump station consists of three booster pumps with a total capacity of 1,070 gallons per minute (gpm).

The existing North Valley Reservoir and Pump Station have the capacity to accommodate some additional demand in the area. Based on a Technical Memorandum issued by West Yost Associates in December 2007, in addition to serving existing users, there is adequate capacity to

serve the airport, Manzanita Rest area, the North Valley schools and Paradise Ranch. To expand service beyond these specific users, including adding any additional residential users, would require significant upgrades to the system, including pipeline improvements and an upgrade of the North Valley pump station. Because of limited space within the existing pump station vault, it may be necessary to relocate the pump station to accommodate any expansion. Additionally, water storage issues will need to be addressed if the service expansion is large enough. The City can continue to provide water service to specific properties within Merlin / North Valley through individual service agreements; however there are no additional obligations to provide service to properties other than those which are currently served (as of 2008.)

In addition to properties within the UGB, there are also properties contiguous to the UGB that are potential candidates to receive water service in the future. Based on information obtained by West Yost Associates from City staff in about 2001, a rough estimate of the area of these properties is 400 acres. Assuming that these properties largely fall within the single-family residential and industrial land use classifications, the additional acreage would increase the annual average demand on the system by approximately 0.5 mgd.

10.20.5 CITY OF GRANTS PASS WATER SYSTEM CAPITAL IMPROVEMENTS PROGRAM (CIP)

Based on the evaluation of existing system performance and future expansion requirements presented in Section 10.20.4, and upon projected improvements required for ensuring the Water Treatment Plant (WTP) continues to serve the City's needs for the next 20 years and beyond, this section integrates the projects into a staged Implementation Plan for WTP improvements and Capital Improvement Program (CIP) for water distribution system improvements (including pipes, pump stations, etc.)

Table 10.20.19 identifies the WTP facility improvements required to meet the build-out of the City of Grants Pass UGB. The Implementation Plan for WTP improvements was initially completed by MHW Americas, Inc. in 2004 as part of the *City of Grants Pass Water Treatment Plant Facility Plan.* The Plan was also included in the Technical Memorandum that was produced by Parametrix in March 2005 and adopted by City Council Resolution No. 4954 in May of 2005.

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Table 10.20.19 City of Grants Pass Water System
Implementation Plan for WTP Improvements (2003 Dollars)

Fiscal Year	Improvements	Estimated Project Costs ($1000)
2004/2005	Intake Modifications (Engr. And Permitting)*	400
2005/2006	Intake Modifications (Engr and Construction)*	500
	Filter Upgrades (Engr. And Construction*	200
	Basin Modifications (Engr. and Construction)	200
2006/2007	Intake Modifications (Construction)*	700
	Filter Upgrades (Construction)*	400
	Basin Modifications (Construction)	400
2008/2009	Filter Gallery Upgrades (Engr. And Construction)	480
	Solids Handling	800
2009/2010	Filter Gallery Upgrades (Construction)	510
	Chemical System Upgrades (Engr. And Const.)	53
2010/2011	Chemical System Upgrades (Construction)	138
	Sludge Removal Systems (Engr. And Const.)	80
	New Storage Building (Engr. And Construction)	27
2011/2012	Sludge Removal Systems (Construction)	239
	New Storage Building (Construction)	53
2012/2013	Emergency Generator for 5 mgd (Engr. and Const.)	319
2024	Expand Capacity to 30 mgd	7,813

*Completed as of September 2007, per Jason Canady, City of Grants Pass Water Treatment Plant Supervisor
Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005,
Adopted By Council Resolution No. 4954, 5/6/2005

In addition to the capital improvements presented above, the City should also implement the following efforts for the Water Treatment Plant (WTP) over the next few years:

- Continue to explore alternative coagulation options to reduce solids production, improve plant performance and reduce operating costs;
- Continue collecting Cryptosporidium samples from the Rogue River to determine "bin classification" according to the LT2ESWTR;
- Develop a DBP sampling program based on the proposed regulations, in conjunction with State of Oregon DHS, to monitor for trihalomethanes (THMs) and haloacetic acids (HAAs), to verify compliance with the proposed Stage 2 D/DBP Rule;
- Complete a Seismic Vulnerability Study; and
- Assess the viability and costs of the sludge handling and disposal program, of which evaluation and implementation has been ongoing for several years and is anticipated to continue for the next 3 to 5 years (as of 2007.)

As of November 2007, it is anticipated that over the next 2 to 3 years the City will verify that it can meet the LT2ESWTR and D/DBP Rule with the existing plant. LT2ESWTR requires 2 years of data collection. The data has been collected once and the WTP is currently completing a second round to ensure compliance with the rule. The stage 2 D/DBP rule requires extensive planning and testing to determine new sampling points before compliance can be determined. If compliance is ultimately determined to be unlikely, then the City may have to implement an alternative disinfection scheme at the WTP.

The City should periodically monitor plant performance and water demands over the next 10 years as it makes capital improvements and to verify that planned improvements are still required. An update of the WTP Facilities Plan should be completed in 5 to 10 years depending on water demands and regulations, including a review of plant expansion requirements.

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The WTP is capable of being expanded to approximately 30 mgd with major modifications. Based on current growth estimates, the plant expansion will not be required until the middle to end of decade 2020. The estimated project cost for a plant expansion to 30 mgd is $7.5 million dollars in 2003, which minimizes the use of additional footprint on the existing site. It is recommended that the City assess available property for a future new plant to expand and partially replace the existing plant within the next 50 years.

The following CIP identifies specific improvements for each of the first five years and for the full build-out of the City's service area. For each of the recommended projects, the CIP also presents cost estimates based on the unit construction costs identified on pages 7-1 of the *Grants Pass Water Distribution System Master Plan*, January 2001, West Yost & Associates. Cost estimates were adjusted by Parametrix in its Technical Memorandum dated March 2005 to reflect 2005 dollar amounts.

Table 10.20.20 City of Grants Pass Water System Capital Improvements Project List & Estimated Capital Costs 2000-2005 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Pump Stations	
Hilltop/Harbeck Heights Fire Pumps	60
Rogue Community College Pump Station	280
Subtotal	340
Pipelines	
Pressure zone boundary modifications	114
Pressure reduction valves	149
P-101 West Harbeck to Allen Creek	62
P-103 Leonard Street Looping	46
P-104 Lower River Road	86
P-105 Prospect Avenue Looping	16
P-106 Hawthorne to Crescent	427
P-107 9th to 10th at Midland	139
P-108 Sherman Lane to Tokay Heights	259
P-109 Marion Lane	131
P-110 C St to D St Loop	56
P-113 Bridge to Brownell	325
P-114 Lincoln Rd Looping	95
P-115 10th and Savage Tie-In	8
P-202 Redwood Ave Looping North	200
P-203 Redwood Ave Looping South	202
P-204 Rogue Community College Extension	4,372
P-207 Williams Hwy Extension	53
P-220 Southeast North Street Extension	26
P-221 Shannon Lane Extension	55
P-222 Lincoln Road Extension	118
P-223 Ament Road Extension	754
P-224 Starlite Connector	290
Subtotal	7,983
Total	8,323

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005; Subtotal and total dollar amounts have been adjusted by City staff due to mathematical errors that appear in original document.

NOTE: Several of the above-listed projects have been fully or partially completed. Uncompleted projects will continue to be identified and included in the work plan / budget for completion.

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Table 10.20.21 City of Grants Pass Water System Capital Improvements Projects List & Estimated Capital Costs 2005-2010 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Pipelines	
P-208 Williams Hwy Looping	336
P-214 Rogue River Hwy Extension	749
P-225 Starlite Extension	114
Subtotal	1,199
Pipeline Replacement	
12,500 feet total replacement	1,124
Subtotal	1,124
Total	2,323

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

Table 10.20.22 City of Grants Pass Water System Capital Improvements Projects List & Estimated Capital Costs 2010-2020 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Treated Water Storage	
Reservoir No 12	1,955
Reservoir No. 14	972
Reservoir No. 16	1,097
Reservoir No. 17	1,543
Reservoir No. 13 (replacement)	972
Subtotal	6,539
Pipelines	
P-206 Reservoir No. 12 Extension	488
P-209 Reservoir No. 17 Extension	111
P-215 Fruitdale Dr Extension	892
P-216 Cloverlawn Loop	64
P-218 Cloverlawn to Crestview Loop	162
P-219 Reservoir No.16 Extension	207
P-226 Greenfield Rd Loop	273
P-229 Reservoir No. 14 Extension	78
Subtotal	2,275
Pipeline Replacement	
25,000 feet total replacement	2,248
Subtotal	2,248
Total	11,062

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

Table 10.20.23 City of Grants Pass Water System Capital Improvements Projects List and Estimated Capital Costs 2020 (2005 Dollars)

Recommended Improvements	Capital Cost ($1000)
Treated Water Storage	
Reservoir No. 10	1,783
Subtotal	1,783
Pump Stations	
Treatment Plant Pumps	457
Subtotal	457
Pipelines	
P-217 Reservoir No 10 Extension	210
Subtotal	210
Total	2,450

Source: Capital Improvement Program, Parametrix Technical Memorandum Dated 4/25/2005, Adopted By Council Resolution No. 4954, 5/6/2005

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10.20.6 PRIVATE WATER UTILITIES

All domestic water services within the UGB area that are not served by the City of Grants Pass system derive their water supply from wells. Prior to requirements for subdivisions and other developments to connect to City water, developments were often served through private water systems. Private water companies supply water through small distribution systems mostly to subdivisions, motels, and mobile home parks. A list of private water systems and approximate populations served by each is available through the United States Environmental Protection Agency. Private utility companies have developed various areas within the City's Urban Growth Boundary, especially south of the Rogue River. Most remaining systems were constructed prior to requirements that they be constructed to municipal system standards. These systems rely upon wells for their supply, and their distribution piping systems consist of small diameter pipe and cannot be used effectively in the Grants Pass Water System. In order for the City to accept these private systems into theirs, the private companies are required to meet or exceed the City's standards. In most cases, the private companies cannot afford the capital improvement costs to comply and naturally resist connecting with the City System.

There are three existing private water systems that are scheduled to be connected to the City Water System within the next two years. These include Bluegrass Park Water Company, Meadow Creek Subdivision and Twilight View Estates7. Per EPA records, the total population served by these three private water companies is approximately 245. At least two additional private water systems, College Oaks and Willow Glenn, were built to City standards and may eventually be connected in the future. However, only the originally-mentioned three systems were planned for connection to City water as of September 2007.

City policy prevents the development of any new private water systems within the UGB.

10.20.7 URBAN SERVICE MASTER PLANS AND MANAGEMENT AGREEMENTS FOR WATER

1. The Grants Pass City Council prepared the *Grants Pass Water Distribution System Master Plan*, 2001; *Water Treatment Plant Facility Plan*, 2004; and the *City of Grants Pass Water Management and Conservation Plan*, 2002 all of which are hereby incorporated into the City of Grants Pass Comprehensive Plan by reference and furthermore, establish the Capital Improvement Program (CIP) and associated costs for keeping pace with build-out of the UGB and serving additional areas outside of the UGB, including portions of the North Valley.

2. The City-County Urban Service Policies, adopted with the UGB in August, 1979, require a public water system with fire flow capacities to serve urban levels of development. A Management Agreement initially adopted January 1981, set out interim development standards to determine domestic and fire requirements for utilizing wells and storage tanks prior to municipal system extension. On August 8, 1998, Josephine County, City of Grants Pass, Harbeck-Fruitdale Sewer District and Redwood Sanitary Sewer Service

7 Per Bob Hamblin and Kathy Mannon, City of Grants Pass Utilities Division, September 2007

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District signed an Intergovernmental Agreement for the Orderly Management of the Grants Pass Urban Growth Boundary Area. This Intergovernmental Agreement replaces each of the earlier agreements.

3. The City has additional agreements to serve specific properties in the North Valley. These include residential properties in the vicinity of the Merlin Landfill, the North Valley Industrial Park, and Paradise Ranch.

10.20.8 CAPITAL IMPROVEMENT PROJECT IMPLEMENTATION PLAN AND FUNDING MECHANISMS FOR WATER

Implementation and funding plans for the City of Grants Pass Water System are found in each respective Master Plan identified below.

10.20.8.1 Water Treatment Plant.

Please refer to the April 2004, *City of Grants Pass Water Treatment Plant Facility Plan*, Chapter 7 - Implementation Plan, pages 7-1 through 7-8 (MWH).

10.20.8.2 Water Distribution System.

Please refer to the *City of Grants Pass Water Distribution System Master Plan*, January 2001, West Yost & Associates, Chapter 7 – Cost of Recommended Capital Improvement Program, pages 7-1 through 7-7 and associated maps locating each improvement.

10.20.9 WATER SERVICES FINDINGS

10.20.9.1 Water Source.

1. Groundwater from the area's alluvial deposit yields a maximum 50 gallons per minute, which is insufficient for municipal supply. Problems of salt intrusion and a dropping water table further limit the groundwater resource. The only reliable source for the large quantities of potable water required for municipal purposes is the Rogue River.

2. The Rogue River yearly flow is effectively fully subscribed, and can support additional subscriptions only by impounding winter flow behind Lost Creek Dam for dry season release.

3. The City has one "perfected right" (priority date 1888) and three permits (priority dates 1960, 1965, and 1983) for withdrawing 12.5 cubic feet per second (cfs), 25 cfs, 25 cfs and 25 cfs, respectively, for a total of 87.5 cfs from the river for municipal purposes.

4. The following information regarding the Grants Pass Irrigation District (GPID) was included in the previously adopted version of this chapter, and should be updated when more current information can be obtained from GPID.

a. The Grants Pass Irrigation District has a "perfected right" of 96.7 cfs with a 1916 priority date, and in addition has a Fish and Game "transport right" of 83 cfs and an "as through" right for the turbine lifts of 800 cfs, also diverted at the Savage Rapids Dam Site. The GPID perfected right may be used for municipal purposes. One-third of this right could provide for 30,490 persons, and one-half could provide for 45,730 persons at maximum day demand levels.

b. The GPID canal and delivery system serves 400 exclusive farm use acres and 7000 urban-suburban acres. The District has 55 miles of major canals and laterals of which 85% are unlined or uncovered. Many of these canals and laterals serve as major drain ways of the City and urbanizing area, and have been incorporated into the Master Storm Drain Plan of the UGB area.

c. Some 2600 acres of irrigated lands have passed out of the District over the years, due mainly to urban level development and the silting and washout problems associated with winter drainage accommodation. The GPID has elected to continue supplying water through canals and laterals as development proceeds, citing as rationale that the improvements required by development often don't exceed "buy-out" costs, that the City's maximum day demand for water in the summer is thereby reduced, and that the irrigation water, being untreated and un-pressurized is cheaper for both the user and the provider.

10.20.9.2 Water Treatment.

5. The City began providing treated water for domestic use in 1931 (3.5/mgd), with expansions in 1950 (4.5 mgd), 1961 (11.5 mgd), and 1983 bringing the total current capacity to 20 mgd (as of 2007.) The WTP is capable of being expanded to approximately 30 mgd with major modifications. Based on a water demand increase of 2.5 to 3 percent per year, the plant expansion will not be required until the middle to end of decade 2020. The estimated project cost for a plant expansion to 30 mgd is $7.5 million dollars in 2003, which minimizes the use of additional footprint on the existing site. It is recommended that the City assess available property for a future new plant to expand and partially replace the existing plant within the next 50 years

10.20.9.3 Water Storage and Distribution.

6. Waters must be stored to allow for hourly fluctuation in demand ("equalizing storage" at 25% maximum daily demand), must meet fire flow demand when normal consumption is at the maximum daily rate ("fire storage" as per ISO tables), and must provide for water supply during a major disruption ("reserve storage," at 50% maximum day demand, or a 12 hour supply under maximum use conditions and 1 1/3 days supply under average use).

7. The annual average, monthly average, and maximum day water demand are calculated from analyzing WTP daily operational data. The analysis allows for the identification of annual average, monthly average, and maximum day water demand based on the period from 1995 to 1999. The average annual demand increased from 3.73 mgd to 4.50 mgd. The highest peak daily demand was 9.47 in August of 1998.

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facilities called for by the Water Facilities Plan, and such map shall be keyed to the computerized model of the distribution system.

10.2.4 The Development Code shall facilitate these water service policies, and shall contain a balanced mix of positive incentives (which may include density transfers, density bonuses, rapid review procedures, etc.) as well as exactive requirements (which may include dedication or easement requirements, system charges, development requirements, etc.) as needed to assure the realization of these policies.

10.2.5 The City and County shall ~~develop~~ ***maintain*** a Capital Improvement Program (CIP) ~~within 12 months of adoption of the Comprehensive Plan,~~ which ~~program~~ shall include timely and adequate funding to realize the development of facilities required by the Water Facilities Plan, and shown on the Water Facilities Plan Map.

10.2.6 The Water Facilities Plan shall be reviewed and updated~~, and revised if~~ ***periodically as*** necessary, ~~at one year intervals,~~ with major revisions at five year intervals. ~~The revisions to the Water Facilities Plan shall be used as a basis for revising these policies.~~

~~10.2.7 Within 24 months of adoption of the Comprehensive Plan, the City and County working with the Grants Pass Irrigation District shall explore the possibility of the municipal use of the District water right, and shall explore the most cost effective way, for public agencies and private individuals alike, to provide water to the UGB area for all purposes.~~

~~10.2.8 Urban level development shall require a public water system, or shall meet requirements of Interim Development Standards as provided by the Implementing Ordinances. Interim Development Standards shall allow development to proceed in a timely and economical manner, prior to full public water system extension, provided the requirements of public safety, health and welfare are met, and the future extension of the public water system is safeguarded.~~

10.3. Sewer Service Policies

10.3.1 ~~Within 12 months of adoption of the Comprehensive Plan,~~ The City and County shall ***follow adopted*** ~~develop and adopt~~ Sanitary Sewer Facility and Management Plans for the Redwood, Fruitdale-Harbeck and City service districts, including all parts of the Urban Growth Boundary area. The Sanitary Sewer Facility and Management Plans: ~~shall:~~

(a) determine the number, size, location and approximate costs of sanitary sewer facilities and improvements deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) base the facilities and improvements determination upon a thorough analysis of the Urban Growth Boundary service districts, including present treatment plan capacity, treatment levels and Department of Environmental Quality requirements, collection system age, construction and function, and infiltration and inflow characteristics of the system;

(c) recommend implementation and financing strategies for acquiring, developing and maintaining needed sanitary sewage facilities;

(d) demonstrate continuity with past sanitary sewer plans, as adopted and developed by the City and County;

(e) provide for adequate coordination between the City and County as needed in the expansion and maintenance of the sewer service districts;

(f) determine the areas of highest priority.

10.3.2 ~~Within 12 months of adoption of the Comprehensive Plan,~~ The City and County shall ~~adopt~~ ***maintain*** an official Sanitary Sewer Facilities Plan Map, showing the location, size and type of existing and future collection and treatment facilities called for by the Sanitary Sewer Facilities and Management Plan. The map shall also show Service District boundaries.

10.3.3 The Development Code and Development Standards shall act to facilitate these sanitary sewer service policies, and shall contain a balanced mix of positive incentives (which may include density transfers, public funding of oversized lines, rapid review procedures, etc.) as well as exactive requirements (which may include dedication or easement requirements, system charges, development requirements, etc.) as needed to assure the realization of these policies.

10.3.4 The City and County shall ~~develop~~ ***maintain*** a Capital Improvement Program (CIP) ~~within 12 months of adoption of the Comprehensive Plan~~, which ~~program~~ shall include timely and adequate funding to realize the development of facilities required by the adopted Sanitary Sewer Facility and Management Plans, and as shown on the Sewer Facilities Plan Map.

10.3.5 The Sanitary Sewer Facility and Management Plans shall be reviewed and updated~~, and revised if~~ ***periodically as*** necessary, ~~at one year intervals~~ with major revisions at five year intervals. The revisions to the Sanitary Sewer Facilities and Management Plans shall be used as a basis for revising these policies.

10.3.6 The City and County shall encourage sanitary sewer design that minimizes the cost of sanitary service extensions, and that minimizes the cost of maintaining such extensions.

~~10.3.7 Urban level development shall require a public sanitary sewer system, or shall meet the requirements of Interim Development Standards as provided by the Implementing Ordinances. Interim development Standards shall allow development to proceed in a timely and economical manner, prior to full extension of the sanitary sewer system, provided the requirements of public safety, health and welfare are met.~~

10.4 .Storm Drain Service Policies

10.4.1 The City and County shall follow the adopted Master Storm Drain Facilities and Management Plan for the Urban Growth Boundary area when extending the improving drainage service. Key factors to be utilized in growth management include:

8. There are eight water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999.

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10.30 SANITARY SEWER SERVICES INDEX

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10.30.1 PURPOSE AND INTENT

10.30.1.1 Purpose

The purpose of this section is to identify existing sanitary sewer service facilities and capacities, identify areas of immediate concern, project capacities needed through the planning period, present financial methods of paying for and regulating the service, and present policies of the orderly provision of services. Within and contiguous to the UGB, there are currently three systems providing sanitary sewer service: City of Grants Pass, Harbeck-Fruitdale Sanitary Sewer Service District, and the Redwood Sanitary Sewer Service District.

10.30.1.2 Intent

The intent of this section is to enact the following public facilities sanitary system master plans by ordinance as an update to the Public Facilities Element of the City of Grants Pass Comprehensive Plan:

1. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Final Report* (Parametrix, June 2001);
2. *Wastewater Facilities Plan Update, City of Grants Pass Water Restoration Plant, Appendices – Final Report* (Parametrix, June 2001)
3. *Collection System Master Plan, City of Grants Pass* (Parametrix, September 2004)
4. *Redwood Sanitary Sewer Service District Engineering Report* (Parametrix, April 1999, Revised November 1999) (**NOTE:** This report evaluated alternatives for either upgrading/expanding the Redwood Wastewater Treatment Plant, or conveying the waste water to the Grants Pass Water Restoration Plant (WRP) for treatment. The preferred alternative identified in the report was conveying the waste water to the WRP for treatment, and the work has since been completed.)

10.30.2 NATURAL ENVIRONMENT

The natural environment includes topography, geology, soils, climate, and water resources of the region. This section presents a brief discussion of these items in relation to sanitary sewer collection system planning and analysis. The information provided in this section was excerpted from the *City of Grants Pass Collection System Master Plan*, completed by Parametrix in September 2004.

10.30.2.1 Topography, Geology, and Soils

The topography, geology, and soils of a region can have a significant effect on the design and construction requirements of entire sanitary sewer system. Topography can determine the route and slope of sewer lines, as well as the need for and location of pumping stations. The geology and soil conditions in an area can affect construction costs for pipelines and determine locations for system components.

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10.30.2.2 Topography

Grants Pass lies in the Rogue River Valley in the Klamath Mountain Range of Oregon. The Rogue River Valley begins at the base of the surrounding hills and exists as a well-defined stream terrace some 10 to 15 feet above the bed of the Rogue River. The valley slopes toward the river at an average gradient of 1 to 2 percent. Elevations on the low-lying valley flow range from 880 to 1,100 feet above sea level. The Rogue River traverses the valley in a general east-west direction on an average slope of about 6 feet per mile. Away from the valley floor, the terrain grows steep relatively quickly. Beacon Hill (located northeast of the City) and Baldy Mountain (located southeast of the City) are 2,117 feet and 2,740 feet in elevation, respectively. The Siskiyou Mountains, part of the Klamath Mountains, lie to the south and west of Grants Pass. To the northeast, a spur connects the Klamath Mountains to the Cascade Range.

10.30.2.3 Geology

The service area contains several major geologic units, including alluvium deposits, diorites and granites, ultramafic and metavolcanic rocks, and gneisses and schists. The alluvium deposits, between 100 and 150 feet thick, formed the valley floor by eroding away from the surrounding rock units. The lava and metavolcanic rock composing Beacon Hill and Baldy Mountain does not weather easily. Its ruggedness has limited development in these areas. The softer granite of Dollar Mountain to the northwest, and various hills to the south and southwest of the city shows greater weathering. The rounded ridges and gentle slopes of these areas have encouraged development.

10.30.2.4 Soils

Weathering of the different geologic units has given the soils of this area a wide range of characteristics. The soils that underlie the developable portions of the Rogue River Valley are of the greatest importance to the collection analysis. Soils with poor drainage can increase the potential for infiltration and inflow (I/I) into the collection system, leading to increased flows to the Water Restoration Plant (WRP.) A survey conducted by the U.S Department of Agriculture (USDA, 1983) identified the soil types found in this area for agricultural purposes. A brief summary of the USDA survey with generalized engineering interpretations is presented below.

The most important soil types in the valley are Newberg fine sandy loam, Barron coarse sandy loam, and Clawson sandy loam. Newberg fine sandy loam is principal soil type in the floodplain and terraced areas of the valley. It occupies a strip along the Rogue River Valley that is generally about a mile in width; however, it narrows to about 2,500 feet at Grants Pass. The soil is well drained and presents no major problems for the collection system. Barron coarse sandy loam occupies extensive portions of the Rogue River Valley and underlies most of the city west of Gilbert Creek. The soil generally occurs upslope from Columbian fine sandy loam and extends as valley fill material into most of the minor tributary valleys. This soil has slightly higher clay content than the Newberg loam, but does not significantly impact drainage or increase I/I impacts.

Clawson sandy loam underlies a major portion of the city east of Gilbert Creek. This soil typically consists of about 1 foot of smooth-textured silt loam overlying a compact silty loam or clay loam subsoil. At a depth of about 30 inches, the subsoil assumes an extremely gritty texture, reflecting the presence of coarse granitic material. The subsoil terminates at shallow

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depths in coarse granitic rock. The soil is flat lying and poorly drained. Because of the impervious nature of the shallow bedrock, it is waterlogged during the winter and spring months. In some areas, the water table is less than 3 feet below the ground level well into the summer. The high groundwater conditions that accompany this soil type can be a problem when wastewater pipelines lying in the soil have cracks or leaks. Groundwater infiltrates into the cracks and leaks, significantly increasing the flow of liquid to the WRP.

10.30.2.5 Climate

Precipitation, temperature, and other climatic factors can significantly affect the design and construction of wastewater facilities. Rainfall is especially significant, because it can cause large flow increases in the collection system due to storm water runoff, illicitly connected roof drains, and raised groundwater levels.

10.30.2.6 General Climatic Conditions

The climate of Grants Pass is generally mild, although temperatures below freezing and above 100 degrees Fahrenheit occur for short periods annually. Climate is influenced by the Pacific Ocean, which is located about 60 miles west of the city. The intervening coastal mountains modify the effect of the marine air masses, causing this portion of the Rogue River Valley to receive less annual rainfall and to have fewer cloudy and rainy days than most other portions of Western Oregon. Monthly temperature and precipitation data for Grants Pass are summarized in Table 10.30.1.

Table 10.30.1 Climate Summary[a]

Month	Temperature, Degrees Fahrenheit					Precipitation in Inches	
	Mean Maximum	Mean Minimum	Mean	Highest Recorded	Lowest Recorded	Mean	Greatest Daily
January	47.4	31.1	39.3	69	13	5.0	3.4
February	54.1	32.7	43.4	76	12	4.4	2.2
March	59.8	34.1	47.0	81	22	3.7	2.2
April	65.6	35.8	50.7	93	24	2.0	1.4
May	73.1	40.5	56.8	102	26	1.2	1.5
June	80.8	45.4	63.1	106	33	0.5	1.0
July	88.8	49.5	69.2	109	39	0.4	1.3
August	89.0	48.9	69.0	110	36	0.5	0.8
September	82.7	43.0	62.9	108	29	0.9	2.9
October	70.4	37.4	53.9	98	20	2.1	1.7
November	53.3	34.6	44.0	77	12	5.1	4.8
December	45.7	31.3	38.5	67	-1	5.4	4.0
Annual	67.6	38.7	53.2	110	-1	31.2	4.8

[a] Source: Records of the Oregon Climate Services, 1971-2000

10.30.2.7 Precipitation

Nearly 75 percent of annual rainfall in Grants Pass occurs between the months of November through March. A majority of the annual precipitation is in the form of rain, although about 4 to 5 inches of snow falls each year. Seasonal snowfall rarely exceeds 10 inches, which usually melts immediately.

10.30.2.8 Temperature

Temperatures in Grants Pass usually remain moderate through the winter. Subfreezing temperatures may persist long enough to freeze water in aboveground facilities; however, it rarely lasts long enough to cause freezing in buried facilities. Summers are warm and dry. Temperatures exceed 100 degrees Fahrenheit on an average of 6 days a year. Nighttime temperatures are generally cool, averaging about 51 degrees Fahrenheit during July, the warmest month.

10.30.2.9 Other Climatic Factors

Sunshine is usually abundant during the spring, summer and fall, but the area is generally cloudy during the winter months. Early morning fog occurs frequently during November, December and January. Fog is less common in October and February and rarely occurs during the rest of the year. Wind speed and direction are not routinely measured at Grants Pass. The prevailing wind direction, however, is from the west, approximately parallel to the axis of the Rogue River Valley.

10.30.2.10 Water Resources

The principal water resources in the service area are surface water from The Rogue River and its tributaries and groundwater from the alluvium covering the river valley. The water resource most important for this Plan, the Rogue River, drains 2,460 square miles above Grants Pass before traversing the study area. The Rogue River is used for the City's potable water supply, irrigation, and recreation.

A number of individual wells rely on area groundwater. The alluvium is the major aquifer, with typical yield of 40 gallons per minute (gpm). Volcanic formations usually yield less water, but in a few highly fractured areas wells have yields as high as 60 gpm. Many of the high yields are not sustainable, as the aquifers are small and substantial drawdown occurs.

10.30.2.11 Water Quality

The *2004 Collection System Master Plan* discusses regulatory framework for both existing water quality and the quality of water discharged from the Water Restoration Plant. The City's NPDES Permit takes into account existing temperature and pollutant loading in setting discharge limits from the Water Restoration Plant. Oregon Administrative Rules (OAR), Division 41, Section 340-41-365, sets standards for water quality in the Rogue River basin. The rules cover dissolved oxygen concentrations, temperature increases, pH values, coliform counts, creation of tastes or odors, toxic conditions that harm aquatic life or affect drinking water, and the formation of sludge. The regulations and associated water quality information relating to wastewater treatment and disposal are discussed in Section 10.30.7.

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10.30.2.12 Water Quantity

The most recent 30 years of Rogue River flow data collected by the U.S Geologic Survey (USGS) are summarized in Table 10.30.2.

Table 10.30.2 Historical Rogue River Stream Flow Data

Month	Monthly Stream Flow Data (1973-2002)[a]		
	Maximum (ft^3/s)	Minimum (ft^3/s)	Mean (ft^3/s)
January	16,610	1,348	5,123
February	10,960	1,162	4,512
March	10,760	1,099	4,364
April	8,395	997	4,076
May	6,428	1,538	3,703
June	4,572	1,016	2,796
July	3,484	974	2,060
August	3,080	878	2,009
September	2,642	1,098	1,724
October	2,282	1,008	1,503
November	9,086	1,160	2,833
December	17,620	1,386	4,866

[a] Data from US Geologic Survey Rogue River Monitoring Station (14361500) at Grants Pass, Oregon.

Flows in the Rogue River can fluctuate widely from year to year. The largest recorded discharge, 152,000 cubic feet per second (cfs), occurred during a December 1964 flood. The lowest recorded discharge was minimum day of 606 cfs during 1968. Reservoirs have since been constructed in the Rogue River basin to provide storage of high wet weather flows for release during dry weather periods.

10.30.2.13 Flood Potential

It is necessary to identify flood-prone areas in order to safely locate wastewater collection and treatment system components. A detailed description of the flood history, mapped locations, and an evaluation of the degree of hazard are found in the Natural Hazards Element of the City's Comprehensive Plan, 1983. The City has adopted Ordinance 4471, which prohibits development in the floodway without a certified "No-Rise" Analysis. Development in the floodway fringe is permitted provided the main living floor is elevated at least one foot above the 100-year Base Flood Elevation, or for nonresidential structures, floodproof construction techniques are utilized.

10.30.3 DEMAND FACTORS

Sewerage demand is driven by factors such as population growth, land use, and to a lesser extent, infiltration from GPID. This section examines the factors driving demand. Unless otherwise noted, the information provided in this section was excerpted from the *City of Grants Pass Collection System Master Plan*. This plan, completed by Parametrix in September 2004, used a planning horizon through the year 2020 and a horizon of 2060 for sizing the distribution system.

10.30.3.1 Population

Consideration of population trends is crucial to long-term sewerage planning. In order to size new facilities and expansions, historical population trends must be examined to predict future population growth. The Grants Pass area has experienced steady population growth since the 1920s. This increase has been in line with the national population trend of people moving to the west and southwest from the northeastern and central states and to rural areas from urban areas. The population in Josephine County rose during the 1970s (5.11 percent annually), but slowed dramatically during the 1980s (0.6 percent annually). The City of Grants Pass grew more rapidly than the county during the last decade (1.5 percent annually).

10.30.3.1.1 Estimate of Population Equivalent for Sewer Service Area

The 1990 census lists the population of Grants Pass at 17,424. Population within the city limits in 2003 was estimated at 22,444. This is the base year population estimate used for the *2004 Collection System Master Plan.* Population estimates completed after the *2004 Collection System Master Plan* are presented in the Population element of the Comprehensive Plan.

The population within the sewer service area consists primarily of the population within the city limits, and the Harbeck-Fruitdale and Redwood areas outside the city limits. Prior to 2001, the City's Water Restoration Plant (WRP) treated sewage only from within the city and Harbeck-Fruitdale area. The *2004 Collection System Master Plan* estimated a 2003 population of 4,620 for Harbeck-Fruitdale, based on sewer connection and billing records. In addition, the City began serving the Redwood Sanitary Sewer Services District (RSSSD) in 2001, and has subsequently converted the Redwood Treatment Plan to a pump station and sewage is now pumped to and treated at the City's WRP. The *2004 Collection System Master Plan* used a 2003 estimated population of 5,714 for the Redwood area. Based on the estimates for the City, Harbeck-Fruitdale and Redwood, the sewer service area population was estimated to be 32,778 in 2003. It is also necessary to take into account the commercial and industrial contribution as a form of population equivalent. The *2004 Collection System Master Plan* assumed that commercial/industrial population equivalent is 35 percent of the total residential population, which equals 11,472. Therefore, the total sewer service area population equivalent was estimated to be 44,250 in 2003.

The *2001 Wastewater Facilities Plan Update* projected the future sewerage area to also include Merlin / North Valley. This area lies outside the UGB to the north of the city and has largely been dependent on on-site sewage treatment systems. The subsequent *2004 Collection System Master Plan* acknowledged that although this area may begin contracting with the City for wastewater services in the future, there are no current plans for service expansion. Therefore, consideration of flows from the North Valley area was not addressed in the *2004 Collection System Master Plan.* In summary, the *2001 Wastewater Facilities Plan Update* would provide sufficient capacity of the WRP to accommodate sewage from Merlin / North Valley. However, the *2004 Collection System Master Plan* does not plan for facilities to collect and convey sewage from this area to the WRP.

10.30.3.1.2 Projection of Population Equivalent for Sewer Service Area

The *2004 Collection System Master Plan* developed a future equivalent population projection for the service area by applying sub-area growth rates from the City's Comprehensive Plan to the base population estimates for the sub-areas and adding 35% to the resulting population for commercial/industrial population equivalent. Estimated growth rates were respectively: 1.5% for

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Grants Pass, 1.6% for Harbeck-Fruitdale and 3.1% for Redwood. Maintaining the commercial/industrial equivalent as 35% of the population share in 2020 resulted in an estimated 1.8% growth rate in commercial/industrial equivalent population. As of 2003, the Grants Pass WRP was serving an estimated population equivalent of 44,250 (including the 35 percent commercial/industrial equivalent.) After applying these growth rates, year 2020 service area population equivalent was estimated as 60,157. For years beyond 2003, it is likely that the city limits and UGB would have enlarged from what existed in 1997. The Redwood and Harbeck-Fruitdale populations would probably then be reflected in the city limit population.

A summary of the population projection used in the *2004 Collection System Master Plan* is shown in Table 10.30.3. In addition to projections for the years 2020 and 2025, the plan also created a projection assuming all available land was at saturation levels of development, which was referred to as Year 2060. The 2060 projections provide a good representation of the build-out conditions that are typically used to determine the size of long-term infrastructure improvements, such as collection system pipelines.

Table 10.30.3 Population Growth by Area

Area	1997 Population	Average growth rate (% per year)	Population Projection			
			2003	2020	2025	2060
City Limits	20,526	1.5	22,444	28,908	31,143	52,440
Harbeck-Fruitdale	4,200	1.6	4,620	6,053	6,550	11,417
Redwood	4,758	3.1	5,714	9,600	11,186	32,563
Commercial / Industrial Equivalent	10,319	1.8	11,472	15,596	17,107	33,747
Total Service Area Population Equivalent:	39,803		44,250	60,157	65,986	130,167

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004

10.30.3.2 Land Use

The *2004 Collection System Master Plan* used land use estimates to generate wastewater flow estimates. These estimates were used to determine the size of long-term collection system improvements. However, land use data was not used in determining plant capacity. Plant capacity was determined exclusively by utilizing the population projections found in the previous section, and applying a 35 percent commercial / industrial equivalency factor. The land use data used in developing the flow analysis can be found in Section 2 of the *2004 Collection System Master Plan.*

10.30.3.3 Grants Pass Irrigation District

The Grants Pass Irrigation District was formed in the early 1920s to supply irrigation water to land located between the town of Rogue River and the confluence of the Applegate and Rogue Rivers. Currently about 7,700 acres of agricultural and residential lands are irrigated. The district has water rights to divert up to 150 cfs from the Rogue River during irrigation season. A series of canals constructed by the district carries the water from Savage Rapids Dam throughout the area covered by the district. Many residents in the sewerage service area use water from the canals to irrigate landscaping and gardens. The canals are also used to carry storm water away from these lands.

6.5.3 Cost Estimates

A summary of the preliminary cost estimates, force-main length, and easements required of each treatment Alternatives 6 through 12 are presented in Table 6-7.

Table 6-7
Force Main Alternatives
Preliminary Cost Estimates

Key	Treatment Alternative	Pipeline Length	# Ease.	Pipeline	Pump Station	Contingency 30%	Engineering & Admin. 25%	Total[1]
Blue	6	26,500	1	$3,240,000	$1,182,000	$1,326,600	$1,105,500	$6,850,000
Green/ Blue	7	24,000	48	$2,960,000	$1,284,000	$1,273,000	$1,061,000	$6,580,000
Green	8	27,000	77	$2,780,000	$1,284,000	$1,219,200	$1,016,000	$6,300,000
Green/ Red	9	29,300	47	$3,000,000	$1,284,000	$1,285,200	$1,071,000	$6,640,000
Red/ Blue	10	28,600	3	$2,930,000	$1,284,000	$1,264,200	$1,053,500	$6,530,000
Red/ Green	11	31,100	32	$3,080,000	$1,284,000	$1,309,200	$1,091,000	$6,700,000
Red	12	33,400	1	$3,230,000	$1,284,000	$1,354,200	$1,128,500	$6,700,000

(1) Rounded to three significant figures.

Alternative 6 provides a single pump station at the Redwood WWTP. Alternatives 7 through 12 include two pump stations: One large pump station at the north end of Darneille Lane, at manhole RI-25, and a smaller one at the Redwood WWTP (RI-0). One disadvantage of this route is that flow in the existing 24-inch-diameter interceptor west of RI-25 would be significantly reduced, thereby resulting in poor scouring of the solids in this gravity interceptor pipe. The District would need to periodically clean this pipeline, using portable, jet-type sprayers.

Operation and maintenance costs for these alternatives are generally similar. Costs would include maintenance of the existing sewer collection system, electrical pumping costs, odor/chemical costs and equipment maintenance. Treatment costs at the Grants Pass WRP would also be paid by the users. The total probable annual O&M/treatment cost would be approximately $230,800. A breakdown of these costs is included in Appendix H.

6.5.4 Preliminary Screening of Alternatives 6 Through 12

Similar to the preliminary screening that was completed on Alternatives 1 through 5, a preliminary screening of Alternatives 6 through 12 was also conducted based on cost and potential impact to residents adjacent to the conveyance pipeline. Based on cost and impact, Alternatives 7 and 9 were selected for further comparison to Alternatives 1 and 4. At workshops with the District and the City of Grants Pass staff, Alternative 9 was further refined into Alternatives 9A and 9B which are described as follows:

Alternative 9A: The force main would be located along the existing easements to RI-25. Then follow the public right-of-way on Leonard Road to Dowell Road, then back onto the existing easement to West Park, the Pedestrian Bridge, and then to the Grants Pass WRP. From Figure 6-10, this alternative is referred to as the "green-red-green" route.

Alternative 9B: Same as above, but the force main would follow the public right-of-way on South River Road, then to Leonard Road.

6.6 PREFERRED ALTERNATIVE SELECTION

A final economic evaluation of Treatment Alternatives 1, 4, 7, 9A and 9B was made by comparing their present worth costs. The present worth costs include capital costs and operation and maintenance costs. Capital and O&M costs for the alternatives were presented earlier in Tables 6-5, 6-6, and 6-8, and in Appendix G and H. Annual O&M costs were converted to a present worth cost based on a return rate of 6 percent over a 21-year period (1999 to 2020). A summary of present worth costs for these alternatives is presented in Table 6-8.

Table 6-8
Treatment Alternatives 1, 4, 7, 9A, and 9B
Present Worth Cost Estimates (in $1,000s)

	Alternative 1 Contact Stabilization	Alternative 4 Complex Mix Effluent Filter	Alternative 7 Easement/ Lower River Road	Alternative 9A Easement/ Leonard Road	Alternative 9B Easement/ South River Road
Subtotal Construction	$7,280	$8,670	$5,520	$5,600	$5,510
Engineering and Admin.	$1,600	$1,910	$1,060	$1,160	$1,150
Total Project Cost	**$8,880**	**$10,580**	**$6,580**	**$6,760**	**$6,660**
Present Worth O&M Cost[1]	$6,870	$7,320	$2,710	$2,710	$2,710
Total Present Worth	**$15,750**	**$17,900**	**$9,280**	**$9,470**	**$9,370**

(1) Present worth based on 6 percent interest for 22 years.

For each of the conveyance alternatives, the City of Grants Pass omitted charging the District a connection charge or System Development Charge (SDC) for treatment at the City's plant. However, even if the City had charged a fee, it would not change the least cost alternative.

As an example, if the City were to charge District customers the SDC currently collected by the District for new developments ($1,966 per ERU), approximately four million dollars would be added to the capital and present worth cost of each conveyance alternative. For Alternatives 7, 9A, and 9B the present worth cost would be $13.28, $13.47, and $13.37 million, respectively. In comparison, the present worth cost of Treatment Alternatives 1 and 4 are $15.75 and $17.9 million, respectively, which are considerably greater. Still, the least cost would be any of the conveyance alternatives.

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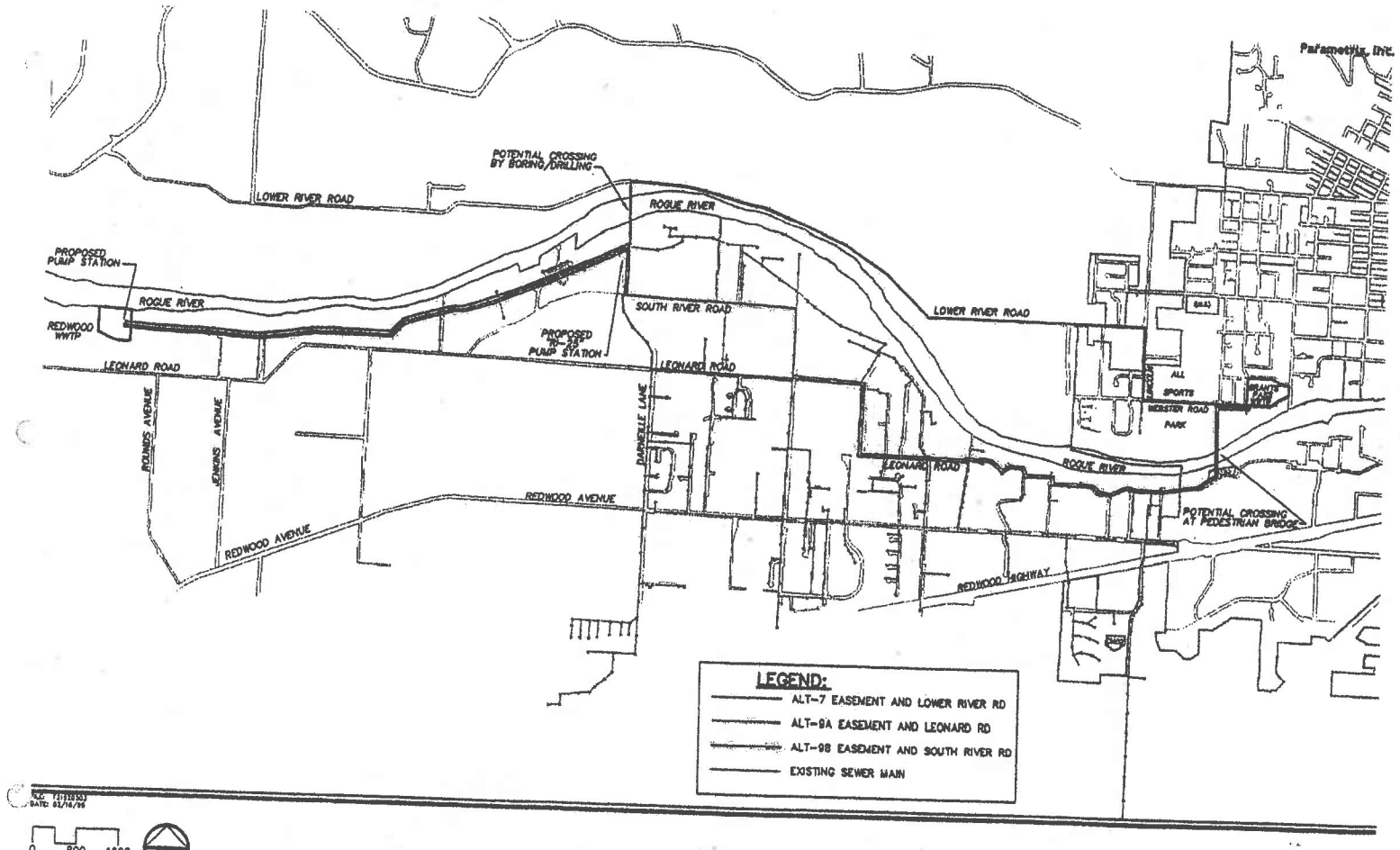

Figure 6-10
Treatment Alternatives 7, 9A, 9B

An important part of the evaluation process was a selection workshop held in Grants Pass on December 3, 1998. To facilitate the selection process, a list of selection criteria was generated and each criterion weighted in terms of importance. The criteria generally fell into four main categories, Cost Issues, Environmental Issues, Community Issues, and Operation Issues. Because cost is a major concern, 42 percent of the weighting was placed on this category.

The selection group was comprised of two representatives from the District, six representatives from the City of Grants Pass, and the City's Attorney. Each representative ranked the alternatives using an uncompleted version of the matrix shown in Table 6-9. The criteria for each alternative were given a rating of 1 to 10 (10 being the higher ranking). After the rankings were done individually then, as a group, all participants completed the matrix shown in Table 6-9. Because of higher costs, the treatment alternatives ranked much lower than the conveyance alternatives. The two top equally ranked alternatives were Alternative 7 - Lower River Road (Blue route) and Alternative 9B - South River Road (Red route).

The results of the selection process were then presented to the Grants Pass City Council, acting as the governing Board for the District, on December 14, 1998. Of the top two alternatives, the Board selected Alternative 9B - South River Road as the preferred treatment alternative.

Table 6-9
Treatment Alternative Selection Process
Evaluation Criteria and Selection Ranking

Criteria	Item Weight	Alt. 1 Contact Stab. without Filtration	Alt. 4 Anoxic Select with Filtration	Alt. 7 Easement/ Lower River Road	Alt. 9A Easement/ Leonard Road	Alt. 9B Easement/ South River Road
Cost Issues – 42%						
Capital Cost	20%	3	1	10	9	10
Present Worth O&M Cost	18%	3	1	10	10	10
Traffic Control	1%	10	10	3	7	8
Probable Utility Conflict	1%	10	10	5	3	4
Length of Pipeline Route	1%	10	10	3	1	1
Geologic Condition Rock	1%	10	10	5	10	10
Environmental Issues – 14%						
Natural Resource Concerns	1%	10	10	8	10	10
River Crossing Risk	5%	10	10	3	5	5
Permitting Issues	2%	10	10	5	8	8
Wetland Concerns	1%	10	10	9	8	8
Community Issues – 24%						
Customer Satisfaction	9%	1	1	5	8	8
Number of Easements Required	9%	10	10	7	4	5
Schedule of Compliance	7%	1	1	6	5	5
Flexibility to Meet Future Demands	1%	5	5	5	5	5
Business Impacts	1%	5	5	5	5	5
Jurisdictional Issues	2%	10	10	9	10	10
Operation Issues – 20%						
Operator Familiarity	1%	8	8	5	5	5
Operation of Existing Plant	1%	1	1	9	9	9
Potential for Odor	10%	5	5	8	5	5
Constructibility	1%	3	3	6	8	8
Potential for Noise	2%	1	1	9	9	9
NPDES Permit Issues	5%	1	3	8	8	8
Weighted Total (high score = Preferred Alternative)	100%	4.49	3.83	7.74	7.43	7.74

Each alternative was given a 1 to 10 rating for each criteria; a "10" being the best rating and "1.0" being the worst.

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6.7 ENVIRONMENTAL REVIEW

6.7.1 Introduction

The purpose of this environmental review is to summarize the potential impacts associated with the No Action Alternative and two build alternatives evaluated for improvements to the Redwood Sanitary Sewer District Wastewater facility. This section includes a characterization of the natural and human elements in the study area, economic considerations, and a summary of public participation. Natural and human elements considered in this chapter include land uses and zoning, historic and cultural resources, wetlands, floodplains, agricultural lands, wild and scenic rivers, fish and wildlife, threatened and endangered species, and other unique or sensitive environmental resources.

During the preparation of this Facilities Plan, 12 improvement alternatives were initially evaluated (please refer to sections 6.1 through 6.5 of this chapter). Ten of the 12 alternatives were eliminated from further consideration. The remaining two build alternatives were more extensively evaluated for potential environmental impacts and the findings are summarized in this section. The No Action Alternative and the two build alternatives are defined below.

6.7.1.1 No Action Alternative

The No Action Alternative is defined as continuing operation at the existing Redwood WWTP as in the past using composting to manage biosolids. Because the District is under court order to eliminate the biosolids composting, the No Action Alternative would be in violation of the court order and, therefore, is not a feasible alternative. This alternative was eliminated from further consideration for this environmental evaluation.

6.7.1.2 Preferred Alternative

The Preferred Alternative is shown as Alternative 9B in Figure 6-10. A smaller pump station would be located at the Redwood WWTP and would pump into a 6-inch force main in existing easements between the Redwood WWTP to RI-25. At RI-25, a second larger pump station would pump through dual force mains to the Grants Pass WRP. The force main route follows the public right-of-way on South River Road to Leonard Road to Dowell Road, then back on existing easements to West Park, then back on public right-of-way on the Pedestrian Bridge to the Grants Pass WRP.

This Alternative was selected by the District Board over the Second Preferred Alternative because they felt that the risks associated with crossing the Rogue River with a horizontal boring were greater than risks associated with the restricted access on easements east of Dowell Road and obtaining easements from property owners.

6.7.1.3 Second Preferred Alternative

The Second Preferred Alternative is shown as Alternative 7 in Figure 6-10. A smaller pump station would be located at the Redwood WWTP and would pump into a 6-inch force main in existing easements between the Redwood WWTP to RI-25. At RI-25, a second larger pump station would pump through dual force mains to the Grants Pass WRP. The force main crosses the Rogue River due north of the RI-25 pump station using a horizontal boring. The route then follows public right-of-way on Lower River Road, Lincoln and Webster Roads to the Grants Pass WRP.

6.7.2 Zoning and Land Uses

Between the Redwood WWTP and Darneille Lane, both alternatives use the same route. This route is zoned Rural Residential with 1-acre-minimum lots. The pipeline would be located in an existing 20-foot-wide easement that currently runs through property used for residential yards, fallow fields, small scale farming, and pastures for horses and cows.

After Darneille Lane, the Preferred Alternative continues through areas zoned for Rural Residential, and Low, Moderate and High Density Residential uses until the route ends at the Grants Pass WRP. All of the proposed pipeline in this section would be located in existing public road rights-of-way and easements. This route would cross the Rogue River using a pedestrian bridge that is under construction.

The Second Preferred Alternative turns north at Darneille Lane, crossing the Rogue River and continuing east through areas zoned for Rural Residential, Exclusive Farm Use, and Low to Moderate Density Residential uses until it ends at the Grants Pass WRP. With the exception of where the pipeline crosses the Rogue River, the pipeline would be located entirely within public road rights-of-way. A small easement would need to be acquired from the Rogue River to the Lower River Road right-of-way. Abutting property in this section of the alignment is used for agricultural, residential and recreational uses. This route would require a boring to cross underneath the Rogue River.

The proposed pipeline is a permitted use in all of the zoning districts. The land use compatibility requirements are apart of the Clean Water State Revolving Fund (CWSRF) and have been documented in the State Revolving Fund (SRF) loan files. Although construction of the proposed pipeline may cause temporary impacts, no adverse impacts to land uses as a result of operating the pipeline are anticipated.

6.7.3 Historic and Cultural Resources

A cultural resource inventory of the area of proposed improvements was conducted. Although no evidence of cultural materials or deposits were observed directly, because archaeological sites in the immediate vicinity of both alternatives have been previously recorded with the State Historic Preservation Office (SHPO), it is recommended that an archaeologist review the final

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project plans showing all anticipated areas of impact. Using the most current plans, archaeological monitoring is recommended for specific locations along the project route.

In addition, there is always the possibility that ground disturbance during construction activities along the alignment might expose buried cultural material or human burials that were not detected during the survey. If such an event should occur, Oregon State law (ORS 97.740, 97.760, 358.905, 390.235, and 358.955), and various federal laws and regulations, which may be applicable to this project, will require that work in the vicinity of such finds be suspended. The SHPO and the appropriate tribes would be notified, and a qualified archaeologist would be called in to evaluate the discovery and recommend subsequent courses of action in consultation with the tribes and SHPO.

6.7.4 Economic Considerations

Various project-funding scenarios were evaluated to determine the best way for the District to finance the projected project cost of $6.7 million for the Preferred Alternative. After reviewing several scenarios, a preferred funding alternative was to fund the project with approximately $5.4 million of State Revolving Fund (CWSRF) loan money and approximately $1.3 million of available cash from the District. The District has an additional $0.7 million cash that will be needed for project reserves. No sewer rates or system development charge increases would be necessary in the District in the next several years; therefore, there will be no adverse economic impact to service area users. The Preferred Alternative is $90,000 less than the Second Preferred Alternative in present worth costs.

6.7.5 Wetlands

The Preferred Alternative will not impact wetland or stream crossings. The proposed project will cross four streams: Sparrow Hawk Creek, Sand Creek, Allen Creek, and Darneille Creek. The boring method of construction will be used to avoid impacts to the systems. The staging area for construction equipment will be set back 25 feet on either side of the stream crossings and will not impact the riparian corridor.

The Preferred Alternative will not impact the Rogue River. The proposed pipeline will be attached to the City of Grants Pass' Pedestrian/Bikeway Bridge, currently under construction. A Nationwide Permit for the Pedestrian/Bikeway Bridge was received from the Army Corps of Engineers on March 22, 1999. The pipeline connection to the Pedestrian/Bikeway Bridge on the north and south sides of the Rogue River were reviewed by a biologist and no wetlands were identified.

The Second Preferred Alternative would cross three stream systems: Sparrow Hawk Creek, Sand Creek, and Darneille Creek. Because the boring method of construction would be used, there will be no wetland impacts. The staging area for the boring will be located 25 feet on either side of the stream crossing and would not impact the riparian corridor of these systems.

The Second Preferred Alternative proposes using the boring method under the Rogue River. No adverse impacts to the River would be expected from this method of construction.

6.7.6 Floodplains

Between the Redwood WWTP and Darneille Lane, both alternatives use the same route and cross flood zones A11, B, and C. Zone A11 is considered an area of special flood hazard (for the 100-year flood). Zone B is considered an area between the limits of the 100-year flood and the 500-year flood; or certain areas subject to 100-year flooding with average depths less than 1-foot or where the contributing drainage area is less than 1 square mile; or areas protected by levees from the base flood. Zone C is considered an area of minimal flooding. The pipeline would be located in an existing 20-foot-wide easement for a gravity sewer. Addition of the proposed pipeline to this existing easement will not alter or affect flooding in this area, nor would the proposed pipeline be affected by flooding.

After Darneille Lane, the Preferred Alternative continues through areas with flood zones C, X, and AE where the route ends at the Grants Pass WRP. Zone X is considered an area of 500-year flood; areas of 100-year flood with average depths of less than 1 feet or with drainage less than 1 square mile; and areas protected by levees from 100-year flood. Zone AE is considered a special flood hazard area inundated by 100-year flood. All of the proposed pipeline in this section would be located in existing public road rights-of-way and easements. This route would cross the Rogue River using a pedestrian bridge, which is under construction. Addition of the proposed pipeline to this existing easement and the public road rights-of-way will not alter or affect flooding in this area, nor would the proposed pipeline be affected by flooding.

The Second Preferred Alternative turns north at Darneille Lane, crossing the Rogue River and continuing east through areas with flood zones B, A11, A4, and AE where it ends at the Grants Pass WRP. Zone A4 is considered an area of special flood hazard by a 100-year flood. With the exception of where the pipeline crosses the Rogue River, the pipeline would be located entirely within public road rights-of-way. A small easement would need to be acquired from the Rogue River to the Lower River Road right-of-way. This route would require boring underneath the Rogue River to cross it. Addition of the proposed pipeline to the existing or proposed easements and public road rights-of-way will not alter or affect flooding in this area nor would the proposed pipeline be affected by flooding.

Although construction of the proposed pipeline may cause temporary impacts, no adverse impacts to land uses as a result of operating the pipeline are anticipated.

6.7.7 Agricultural Lands

Between the Redwood WWTP and Darneille Lane, the pipeline would be located in an existing 20-foot-wide easement that currently runs through property used for residential yards, fallow fields, small scale farming, and pastures for horses and cows. Because the pipeline would be located within an existing easement for a gravity sewer, construction could cause temporary

disruption of agricultural activities. However, after construction is complete, the operation of the proposed pipeline in this section of the route would not disrupt any agricultural activities. The Second Preferred Alternative turns north at Darneille Lane, crossing the Rogue River and continuing east within public road rights-of-way. Some abutting property in this section of the alignment is used for agricultural activities. Construction of the proposed pipeline could cause temporary disruption of agricultural activities. However, after construction is complete, the operation of the proposed pipeline in this section of the route would not disrupt any agricultural activities.

6.7.8 Wild and Scenic Rivers

A section of the Rogue River has been designated a wild and scenic river. The wild and scenic river designation begins at the confluence of the Applegate River and extends approximately 84 miles to the Lobster Creek Bridge. Because the project is not located within the wild and scenic area of the Rogue River, neither the Preferred Alternative or the Second Preferred Alternative will have impacts to the area designated as a wild and scenic river.

6.7.9 Fish and Wildlife

Correspondence with the Oregon Natural Heritage Program (ONHP), U.S. Fish and Wildlife Service (FWS), and National Marine Fisheries Service (NMFS) was initiated to identify any federally listed, proposed, or candidate threatened or endangered species that may potentially occur within the project area. Potential impacts to listed species identified in the agency correspondence along with the results from the October 1999 fish and wildlife survey will be addressed in a Biological Assessment (BA). The BA will be conducted after the route alignment is selected and final design is underway. Potential impacts to listed species will be the same for the Preferred Alternative and the Second Preferred Alternative.

6.7.10 Threatened and Endangered Species

Informal consultation with the NMFS, ONHP, and FWS was initiated as part of this process to identify the presence of any federally listed, proposed, or candidate threatened or endangered plant or animal species. A BA in compliance with Section 7 of the Endangered Species Act (ESA) will be prepared to determine potential effects of the project on listed, proposed, and candidate species and their habitats. The BA will be conducted after the route alignment is selected and final design is underway. The impacts to listed species will be the same for the Preferred Alternative and the Second Preferred Alternative. Based on preliminary review by biologists, impacts to listed species are not anticipated.

6.7.11 Other Unique or Sensitive Environmental Resources

No other unique or sensitive environmental resources were identified for either alignment alternative.

6.8 CONSTRUCTION TECHNIQUES, BEST MANAGEMENT PRACTICES, AND MITIGATION OF SELECTED NATURAL RESOURCE CONCERNS

6.8.1 Construction Techniques

The pipelines will be installed at 3- to 6-foot-deep and 3- to 7-foot-wide trench (wider for dual pipeline). The pipes will be in a bed of ¾-inch aggregate. Trenching and pipe installation will be done with a track hoe. Surfaces will be restored to match the existing conditions: pastures will either have topsoil replaced or hydroseeded (owner's preference), undeveloped areas will be hydroseeded, and roads will be gravel/paved. Where the pipeline alignment is near sensitive natural features, the construction plans will call for appropriate Best Management Practices. See the attached figure for pipeline installation and restoration details.

Dewatering

Where groundwater is encountered in excavations, the excavation will be dewatered either with a submersible pump or temporary well points established around the perimeter of the excavation. Water pumped from these will be sent to a two-stage temporary sedimentation pond. Water will flow into the first cell, then through a gravel filter to get into the second cell. From the second cell, clarified water will be discharged into the natural drainage system. This method has been successfully employed on several projects and is effective in removing turbidity.

Creek Crossings

Creeks will be crossed by horizontal borings (see below). The pipeline(s) will be in a steel casing, which will start and end 25 feet back from the creek banks, and the top of the pipe will be a minimum of 10 feet below the creek bottom.

Horizontal Borings

Boring is done using an auger in a steel casing. The casing is driven horizontally, and the auger in the casing rotates to bring earth to the bore pit, where it is removed to the surface. The casing is advanced using a horizontally oriented hydraulic jack. After the casing is installed, the "carrier" pipeline (containing the liquid being piped) is installed in the casing. Bore pits are excavated on either side of the obstacle (creek): a driving pit and a receiving pit. The pits are from 15 to 30 feet deep, depending on the depth of the creek. The pit excavations will be near vertical and steel trench shoring will be used to maintain the sides. The driving pit will contain the hydraulic jack. The receiving pit is typically dug only after the casing boring is completed to expose the casing and install the carrier pipeline. Fencing is provided around the bore pits during non-working hours for safety and erosion control measures are employed around the work site. The area around the boring is used to stage casing pipe and typically contains a track hoe for removing bored material. Dump trucks travel to and from the site.

Construction Staging

Equipment and material will be staged at the Redwood WWTP, the RI-25 Pump Station, the Josephine County Fairgrounds, and the Grants Pass WRP.

6.8.2 Best Management Practices

Erosion Control

Silt fences (reinforced with wire fencing as required) and straw bales are employed at the perimeter of the construction site to isolate it from natural areas. The silt fence must be installed to capture surface runoff. It must be inspected daily by the contractor. The engineer's inspector checks it upon installation and periodically thereafter for satisfactory condition and performance.

Water Disposal

Water removed from dewatering is typically returned to natural drainage. In the event the water does not meet standards, it will be discharged to the adjoining sanitary sewer or storm sewer, which parallel the entire new pipe alignment.

6.8.3 Mitigation of Selected Natural Resource Concerns

Tree Removal and Replacement

Approximately 20 ornamental cedar trees on the south boundary of All Sports Park will be removed and replaced in kind.

Noise

The area for the pipeline alignment is rural residential. As such, there is a significant amount of farming occurring thus, the sound of tractors operating is not unusual. There are numerous residential communities and several new residential developments currently under construction along the pipeline route. Thus, the sound of construction equipment is quite common.

After construction, the pump stations will operate continuously. The noise from the pumps and the emergency generator will be less than 45 decibels at the property line of each pump station.

6.9 PUBLIC PARTICIPATION

The Redwood Sanitary Sewer Service District has involved the public in a variety of ways in the process to evaluate alternatives and select a preferred alternative for the Redwood WWTP upgrade. The preferred alternative was selected on April 28, 1999, at a Sewer District Meeting. Ongoing communication between ratepayers and affected property owners has continued since the preferred alternative was selected. Copies of newsletters, meeting minutes, and public

notices related to this effort can be found in the Appendix I. The following listing identifies public participation activities conducted and planned.

Schedule of Public Involvement Activities

- Public Meeting Announcement, November 1998
- District Board Briefing, November 18, 1998
- Public Meeting, November 19, 1998
- District Board Briefing, December 10, 1998
- District Board Briefing/Selection of a Preferred Alternative, December 14, 1998
- District Board Briefing, March 8, 1999
- District Board Adopts Wastewater Facilities Plan Update, April 28, 1999
- Project Press Release, May 21, 1999
- District-Wide Project Update, Public Meeting Announcement, May 1999
- Letter to Affected Property Owners, May 24, 1999
- Affected Property Owner Meeting, June 1, 1999
- Conveyance Route Field Walk, June 2 and 3, 1999
- Pipeline Route Walk-Through/Talk with Property Owners, June 2 and 3, 1999
- Property Owner Update/Meeting Announcement, July 1999
- Affected Property Owner Meeting, July 20, 1999
- Individual Property Owner Meetings, July through October 1999
- District-Wide Project Update, planned for November 1999
- Public Meeting, To Be Determined

Individual Letters to Property Owners

- Ozust – Information regarding pipe, August 2, 1999
- Langevin – Information regarding pipe, August 6, 1999

6.10 WASTEWATER FACILITIES PLAN UPDATE

6.10.1 District Board Briefing – November 18, 1998

The Grants Pass City Council, acting as the governing board for the District (District Board), received a project update and briefing on November 12, 1998. District Board members were given a status report on the evaluation of the 12 alternatives being evaluated (5 treatment options and 7 conveyance options). District Board members were informed of the November 19, 1998, public meeting to solicit public input on the conveyance and treatment alternatives. In addition, the District Board was informed of a staff workshop on December 3, 1998, to evaluate alternatives.

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6.10.2 Public Meeting/Forum – November 19, 1998

A public forum and information meeting notice was sent to all ratepayers in the Redwood District in November 1998.

Twenty-eight people attended the District-wide meeting held on November 19, 1998 and the alternative solutions were presented. A questionnaire was distributed among meeting attendees to determine public comments on a preferred alternative. Written comments or testimony were accepted until November 20, 1998. Sixteen questionnaires were returned to the District. Of the alternatives presented, the public preferred the conveyance alternatives to the treatment alternatives. The rationale for supporting the conveyance option was cost.

6.10.3 District Board Briefing – December 10, 1998

District representatives briefed the District Board on December 10, 1998. District representatives presented the results of the November 19 public meeting and the December 3 staff workshop. The results revealed that the public showed a strong preference for the conveyance alternatives. Staff members had also selected the conveyance alternatives. The staff workshop narrowed the conveyance alternatives to two preferred alternatives. The two conveyance alternatives received identical scoring; therefore, a decision by the District Board was requested.

6.10.4 District Board Briefing, Preferred Alternative Selected – December 14, 1998

The District Board selected the preferred alternative at the meeting (open to the public) held on December 14, 1998.

6.10.5 District Board Briefing – March 8, 1999

A project briefing (open to the public) to the District Board was held to keep the Board and public informed of the status of the Wastewater Facilities Plan Update.

6.10.6 District Board Adopts Wastewater Facilities Plan Update – April 28, 1999

On April 28, 1999, the District Board adopted the Wastewater Facilities Plan Update. This reaffirmed the selection of the Preferred Alternative.

6.11 PREFERRED ALTERNATIVE IMPLEMENTATION

6.11.1 Project Press Release and District-wide Project Update

A project press release was sent to the local newspaper, and a District-wide project update was sent to ratepayers in the District. The purpose of the press release/newsletter was to inform the public of the preferred alternative and explain the next steps related to the project.

6.11.2 Letter to Affected Property Owners

A letter to all affected property owners (approximately 70 people) was sent on May 24, 1999. The letter explained the Pipeline Project and informed property owners that they would be directly impacted by the project. They were invited to attend a public open house on June 1, 1999, to learn more about the project, schedule, and impacts. In addition, a right-of-entry form was included in the letter for property owners to complete and return to the District. The right-of-entry form would allow surveying and selected field work on private property.

6.11.3 Affected Property Owner Meeting

The purpose of the first affected property owner meeting was to explain the project history, project purpose, and project plan. Because pipeline construction would occur across easements located on private property, the District thought it was necessary to meet with property owners and discuss the project. In addition, the meeting set the stage for obtaining right-of-entry forms and temporary construction easements. A property owner informational survey was distributed.

6.11.4 Conveyance Route Field Walk

On June 2nd and 3rd, District representatives walked the conveyance route to obtain field data and talk one-on-one with impacted property owners. District representatives discussed the project with approximately 50 percent of the property owners along the route. They also put flags in the projected location of the new pipeline. The purpose of the flags was to show residents where the new pipeline route and the existing sewer easement are located. In addition, the flags included the phone number of a District contact in case property owners had questions about the project.

6.11.5 Property Owner Update/Meeting Announcement and Property Owner Meeting

A second property owner update and meeting announcement was distributed in early July 1999. The property owner meeting was held on July 20, 1999. The purpose of the meeting was to discuss the issue of temporary construction easements. District representatives explained the need for the temporary construction easements and the rate schedule that would be used to compensate property owners for granting the temporary easements. The process for obtaining easements was also discussed.

6.11.6 Individual Property Owner Meetings

From July to October, District representatives spent time meeting with affected property owners individually to discuss the project, answer property owner questions, discuss restoration requirements, and obtain temporary construction easements. Letters were sent and follow-up phone calls were made to individuals who were unable to meet with District representatives.

In addition, letters were sent to two separate property owners addressing questions and concerns (see letters to Ozust and Langevin).

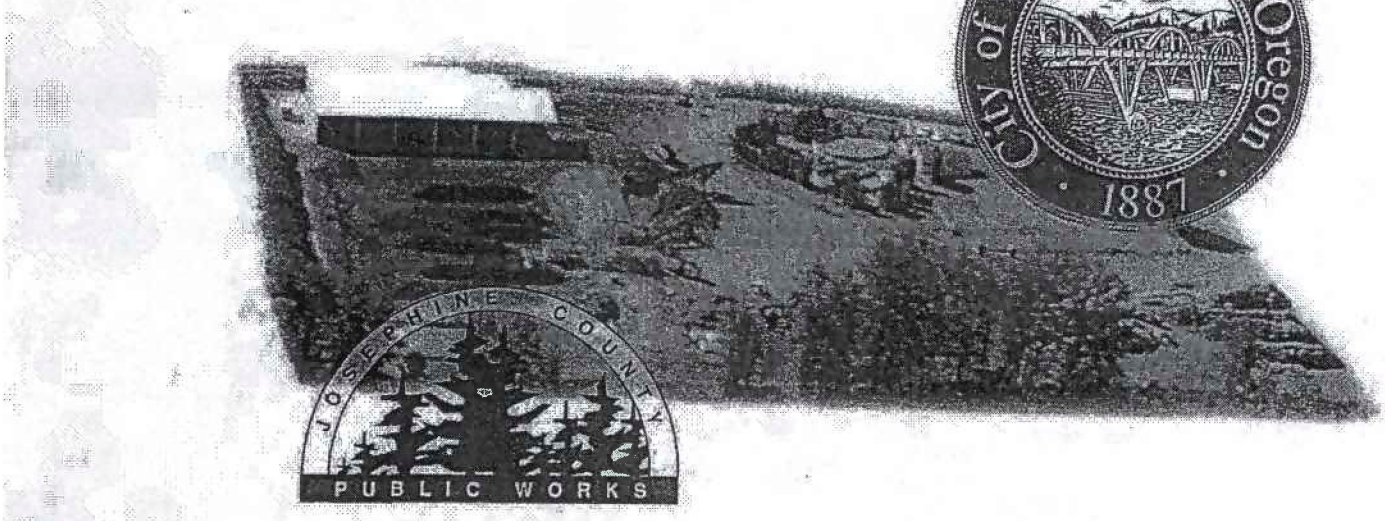
6.12 FUTURE PUBLIC INVOLVEMENT ACTIVITIES

The District plans to send out another District-wide Update in November 1999. A District-wide and affected property owner public meeting will be held prior to construction. In addition, the District will continue to update ratepayers and property owners of the construction schedule when construction begins in April 2000.

Preferred Alternatives

Redwood Sanitary Sewer Service District

Josephine County, Oregon



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7. PREFERRED ALTERNATIVE

A description of Treatment Alternative 9B – South River Road is presented in this chapter. This description is divided into the following sections:

- Pump Station Design Criteria
- Force Main Design Criteria
- Related Issues
  - Demolition of the Redwood WWTP
  - Transfer of Redwood WWTP NPDES Permit
  - Easements
- Project Schedule

In addition, a more detailed pre-design level presentation of Alternative 9B is contained in Appendix J and includes a more detailed description of each of the above topics.

7.1 PUMP STATION DESIGN CRITERIA

For Alternative 9B, two pump stations would move the wastewater from the Redwood WWTP to the Grants Pass WRP. A pump station near the existing manhole RI-25 would receive interceptor flow from all sewers east of this pump station plus flow from a smaller pump station (RI-0) to be located at the existing Redwood WWTP.

7.1.1 RI-25 Pump Station

The flow analysis projections presented in Section 5 are the basis of design for the RI-25 pump station. The peak-day design flow would be 4.2 mgd for the year 2020 and the minimum design flow at current conditions would be 0.28 mgd. Because the flow projections cover 21 years of growth, phase construction was considered rather than building a pump station for year 2020 flows at this time. Based on these parameters, the new RI-25 pump station design criteria would be as follows:

• Peak Flow (Phase 2)	4.2 mgd
• Head Based on Dual 12-inch Mains	181 feet
• Peak Flow (Phase 1)	3.1 mgd
• Head Based on Dual 12-inch Mains	130 feet
• Minimum Flow (Existing)	0.28 mgd

Pump selection would be based on the ability to meet high-flow conditions. As indicated in Section 6.3, variable speed drive control is recommended for the pumps. With variable speed drives, the pumps can better match the incoming flow rate. This ensures two things: 1) the pumps will cycle on and off less often (thereby prolonging pump life), and 2) flow pumped to the Grants Pass WRP would be maintained at a fairly uniform flow rate. A uniform flow would make it easier to operate the Grants Pass WRP, particularly during high-flow conditions.

7.1.2 RI-0 Pump Station

Sewer flow generated west of RI-25 would flow through the existing interceptor to the current Redwood WWTP. At this plant, the existing influent pump station would be modified with new pumps. The capacity of these pumps would normally be based on available flow data. However, since there is no wastewater flow measurement data from this area, pump capacity is not yet finalized. The District is currently installing flow monitoring equipment downstream of manhole RI-25 and comparing it to Redwood WWTP influent flow to determine flow contribution between the two points. Accurate design criteria cannot be established until data is available later this year. For this report, however, flow estimates were made based on population and zoning. Design criteria for the RI-0 pump station are shown below:

- Peak flow (Phase 2) 0.4 mgd
- Head based on one 6-inch main 72 feet
- Peak flow (Phase 1) 0.3 mgd
- Head based on one 6-inch main 40 feet
- Minimum flow (existing) 0.08 mgd

The existing pump station has two pumps in a dry-well-package-type pump station. The wet well for these pumps is manhole RI-0. A drawing of the existing pump station is included in Appendix K. Converting the existing pump station into a force main pump station would require new pumps, because the existing pumps are not suitable for the new flow and head requirements. The most cost-effective approach to meet the new conditions would be to retrofit the existing pump station and place two new submersible pumps into RI-0. Each pump would be rated for maximum flow. In an extremely high-flow situation, both pumps could then be operated if necessary.

7.2 FORCE MAIN DESIGN

Based on the pump station design criteria presented in the previous Section, the two proposed force main systems to accompany each pump station were evaluated. The criteria for each force main system are as follows:

- Provide a minimum scour velocity of 3.0 feet per second at low flow to minimize solids buildup in the pipeline.
- Reduce total force main hydraulic loss to under 200 feet (ideally 100 feet) at peak-design flow.

7.2.1 RI-25 Pump Station Force Main

The range of flows is predicted to vary from a minimum of 0.28 mgd to a maximum of 4.2 mgd. A force main diameter is selected based on the design velocity limits. For the range of flows anticipated, however, two pipelines would be needed to maintain the flow velocities within the recommended range and the head below 200 feet. As velocities increase, so does the pipeline pressure (head loss). There is an upper practical limit of 150 to 200 feet of head for solids-handling centrifugal pumps.

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Based on these two key criteria, a dual force main system would be the best alternative. To provide capacity at peak flow, either a dual 12-inch-diameter force main system could be used or a 12- and 14-inch-diameter system.

A dual 12-inch-force-main system would meet the two key criteria plus have a cost advantage over the 12- and 14-inch-force-main system. Since the greater capacity provided by a 14-inch main would not be needed and the costs for this alternative is greater, it was not further considered.

Based on a minimum pump station RI-25 water surface elevation of 870 and a maximum pipe elevation at the pedestrian bridge of 920, the maximum static head for the system would be 50-feet. During peak flow, the dynamic head loss with both pipes in operation would be approximately 122 feet. Adding the static head loss and loss through fittings would give a total head requirement of approximately 182 feet. Few available wastewater pumps can meet these standards and pass 3-inch solids. For wastewater pumping, this is a relatively high head situation. Pumps that could meet this condition could potentially have problems with rags getting lodged in their 5-inch discharge piping. The best way to avoid this problem is to use Wemco Hidrostal pumps which are not prone to plugging.

For a more detailed summary of pipeline velocity and head loss calculations, refer to Appendix L.

7.2.2 RI-0 Pump Station Force Main

The smaller pump station to be located at the existing wastewater treatment plant would serve less than 15 percent of the existing service area. The maximum estimated flow capacity for year 2020 is 350 gpm. Three force-main sizes were considered for this design flow. Because the head loss created in a 5-inch force main would be in excess of 400 feet, this option was not further considered. An 8-inch force main would require approximately 64 feet of pumping head. The major drawback to this option would be reaching a scour velocity. Scour velocity would require a flow of 470 gpm. This high flow would adversely impact downstream pumping and treatment facilities.

Based on problems with the other alternatives, the 6-inch force main would be the best choice for this installation. A 6-inch force main would require almost 176 feet of pumping head. To provide future flexibility at a minimal increase in capital costs, variable speed drives would also be included with this alternative. The force main would be constructed in the existing interceptor sewer easement.

Bridge Crossing

The preferred alternative includes attaching one 14-inch force main to the proposed pedestrian bridge over the Rogue River. A preliminary drawing of the pedestrian bridge is included in Appendix K. The bridge would be the high point of the entire force main system. The pipe would be placed on one side of the bridge with air release valves located at the high points to release air when the pipe is being filled. To control odor, the air released would pass through carbon filters to scrub the air before it is released.

Interceptor Storage Capacity

The existing interceptor sewer that conveys wastewater from the District to the Redwood WWTP is already used occasionally during high flow events to equalize flow when the influent pump capacity is exceeded. The last 420 feet of interceptor before the Redwood WWTP is 27-inch-diameter pipe. The next 1,080 feet of interceptor is 24-inch pipe. If the last 1,500 feet of this interceptor pipe were used for flow equalization, there would be approximately 38,000 gallons of storage available at RI-0. Based on the projected peak-hour flows for the District, this would not be a significant volume of storage. Future flow conveyed through this pipe, however, will only come from a small portion of the District and this storage capacity could theoretically hold several hours of flow. Selection of the RI-0 pumps will be based on using this volume.

7.3 RELATED ISSUES

There are several issues related to the preferred alternative. Three of these issues are discussed in this Section.

Demolition of the Redwood WWTP

One relatively small cost that should be included with the preferred alternative is the cost of abandoning most of the facilities at the existing Redwood WWTP. The facilities that would be abandoned are:

- Influent pump station
- Headworks
- Aeration basin
- Clarifier
- Chlorine contact chambers
- Aerobic digester
- Paved area at compost facility

The compost building would be dismantled and relocated to a new compost facility planned by the City of Grants Pass.

Although most of the existing facilities would not be used with the preferred alternative, several of the facilities could be. The facilities that would be kept for use with the preferred alternative would include:

- Control building
- Blower/generator building
- Outfall pipeline

The control building would be used to provide a restroom and shower for maintenance staff. The blower/generator building would be used to house the motor control center and instrumentation and controls for the new pumps. The outfall would be used as an emergency outfall to the river in the event of a catastrophic pump station failure. None of the existing facilities will be used for wastewater storage.

Transfer of Redwood WWTP NPDES Permit

The existing Redwood WWTP's NPDES discharge permit includes an allocation for maximum lbs/day BOD and TSS loads to the Rogue River. Because wastewater treatment is being transferred to the Grants Pass WRP, this allocation should be transferred to the Grants Pass WRP. The existing NPDES permit for the Redwood WWTP includes wasteload BOD and TSS limits as shown in Table 7-1.

Table 7-1
Transfer of BOD and TSS Wasteload Capacity (lbs/day)

	Monthly Average	Maximum Week	Maximum Day
May 1 to October 31			
BOD	80	120	160
TSS	80	120	160
November 1 to April 31			
BOD	120	180	240
TSS	120	180	240

These discharge loads should be transferred by ODEQ to the Grants Pass WRP NPDES permit once the preferred alternative is operational. A request for a transfer of mass BOD which follows ODEQ administrative procedures will be made later by the City of Grants Pass.

Property Easements

Before force main final design begins, the final force main route would need to be established. Much of the proposed pipeline route lies within the existing gravity collection interceptor easement.

In general two types of easements are used:

- Permanent easements to house the utility. These allow staff access for operation and maintenance.
- Temporary (construction) easements to provide an area for the construction contractor to conduct work.

The existing (permanent) Redwood Intercepter easement is 20 feet wide. This easement is sufficient for the one force main between Redwood WWTP and RI-25; however, it is just barely sufficient for the anticipated dual force mains needed between RI-25 and the Grants Pass WRP.

For this reason, we evaluated three different approaches to the installation of dual force mains in this area.

A. Limit construction within the 20-foot existing easement. This is the least desirable approach because it will likely cost more money to limit the contractor's work space. There may be short sections of the project, however, that require this approach.

B. Provide a new 20-foot-wide temporary construction easement adjoining the existing permanent easement. This is the preferred approach.

C. Provide a new 10-foot-wide permanent easement to install the new pipelines, plus provide a new 20-foot-wide temporary construction easement adjoining the new permanent easement. This would provide the contractor with the most room to work and allow more room for future access. However, it would be the most costly to the District and disruptive to property owners; therefore, it was not evaluated further.

It was assumed that the existing 20-foot-wide easement cannot alone be used for construction of the new pipelines, and an additional 20-foot-wide temporary construction easement (Approach B) should be provided, where possible, to allow the contractor sufficient room to work, for receipt of deliveries, and for staging equipment and materials.

The cost of easements can be significant. The costs include not only compensation to the owner for the property, but also fees to engineers, surveyors, real estate negotiators, and attorneys to prepare and process needed paperwork.

7.4 PROJECT SCHEDULE

Based on the criteria presented in this section for the preferred alternative, a preliminary project schedule has been prepared and is shown in Figure 7-1. The main tasks included in the project schedule are as follows:

- Submit facility plan to ODEQ for approval
  Request additional SRF funding
- Engineering design
- ODEQ review and approval of design
- Advertise, review bids, and award construction contract
- Complete construction and startup force main

Based on this schedule, the earliest that the new pump stations and force mains could be put into operation would be mid-October 2000. This is also the projected completion date for the pedestrian bridge over the Rogue River.

7.4.1 Plant Operation During Construction

The construction process of the preferred alternative should have little or no impact on operation of either the Redwood WWTP or Grants Pass WRP. This is because very little, if any, of the actual construction activities would occur at either site.

The only exception to this may be during two construction procedures:

- When the RI-0 pump station modifications are being built, a temporary pump station to bypass RI-0 will be required. This would consist of installing a temporary submersible pump into manhole RI-1 and using a temporary aboveground force main system to the existing headworks.

- When the dual force mains are connected to the Grants Pass WRP influent pump station, it may require a short-term shutdown of this station to make the final connection

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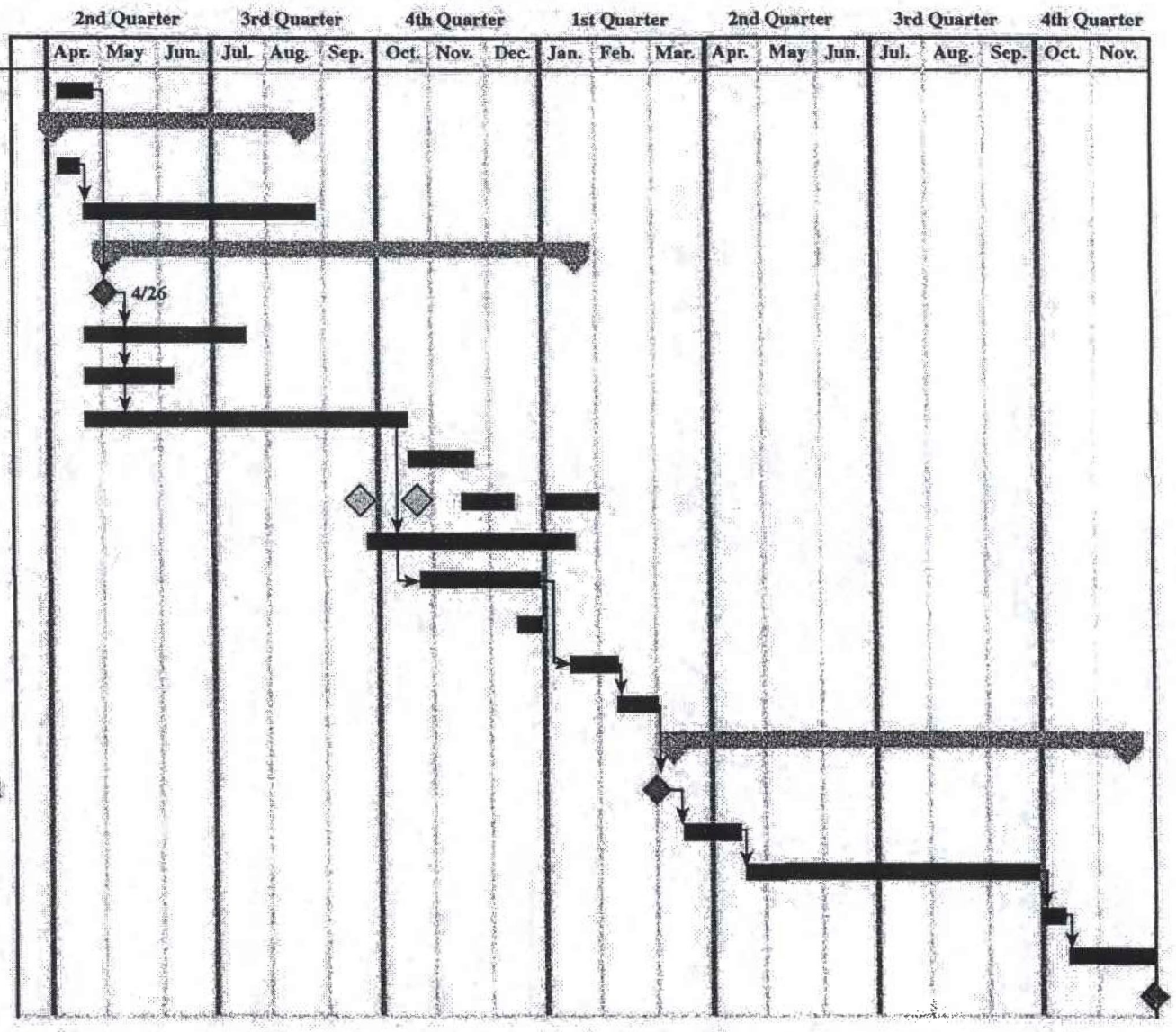
ID	Task Name
1	**Submit Facility Plan Update**
2	**Project Funding**
3	Request Funding
4	Coordinate w/Agencies
5	**Engineering Design**
6	Notice to Proceed
7	Const. Elements
8	Survey & Geotech
9	60% Design Submittal
10	Owner Review
11	DEQ Review
12	Permit Process
13	Final Design
14	Plan of Operation
15	Bid Project
16	Award Project
17	**Construction**
18	Issue Notice to Proceed
19	Submittals
20	Construction
21	Force Main Start-Up
22	Restoration/Clean-up
	Project Completion



Redwood Waste Water Treatment Plant
#27-2192-05.02/99

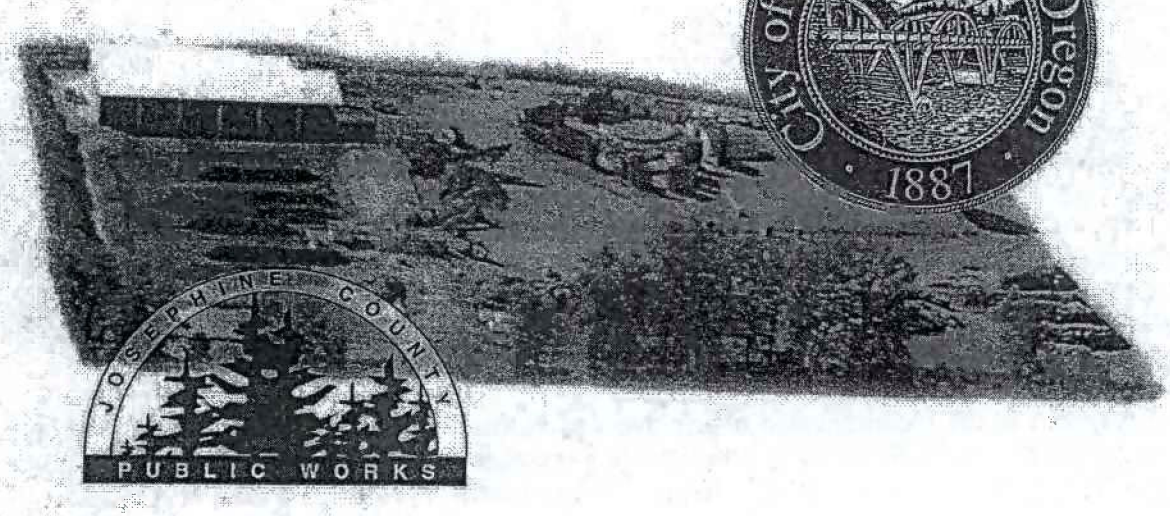
Summary Milestone Task ODEQ Submittal of 60% Design

Figure 7-1
Project Implement Schedule

Financing Plan

Redwood Sanitary Sewer Service District

Josephine County, Oregon



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8. FINANCING PLAN

Various project funding alternatives exist for the District to implement the preferred treatment alternative. The purpose of this section is to select the best option from those alternatives available. This financing plan includes an evaluation of present and future revenues and costs. The project construction, engineering, administrative, and operation and maintenance (O&M) costs used in the evaluation were taken from Section 5. Revenue projections are based on existing rates, system development charges (SDCs), and the population growth projections presented in Section 2.

The financial analysis consists of two parts:

- **Capital Cost Review** – We reviewed the capital improvement costs to ensure all known, related costs were included.

- **Revenue Requirements Analysis** – We analyzed needs of the utility based on current budget and financial information, projected future operating and maintenance costs, and ongoing capital outlays. We added to this the cost impact of the capital improvement program in terms of direct user charge support and debt service support. Alternative financing approaches and combinations were then examined.

8.1 BACKGROUND

The Wastewater Facility Plan completed in November 1994 included a project financing section. The four funding sources included in this evaluation were the State Revolving Fund (SRF), Community and Economic Development, the Farmers Home Administration, and Revenue Bonds. A very thorough evaluation of each of these funding alternatives was performed, including alternative combinations. The best options were a combination of SRF loans and bond funding, or 100 percent SRF project funding. Because there were not enough SRF loan monies available to finance the entire project, the conclusion reached was that the SRF loans would be augmented with revenue bonds or general obligation bonds.

Also included in the recommended alternative was an increase in system development charges (from $922/ERU to $1,966/ERU), and a gradual increase in monthly sewer rates ($2 to $3 per year). These changes were implemented in 1995 and have been revised on a regular basis.

8.2 EXISTING FINANCES

Before discussing the various project financing alternatives, a review of the existing finances and costs is presented. A summary of the District's operating revenue and expenses for 1998/1999 is presented in Table 8-1. Under "revenues," the System Development Charge (SDC) shown is for managing the accounts. Most SDC revenues are placed into Account Numbers 594 and 595. Under "expenses," the intergovernment payments include billing by the City of Grants Pass, inspection services, and other services provided by the County. Account Number 596 expenses are for capital works improvements to the system. The current sewer debt level is very low. The total current debt payments per year are only $12,800.

Table 8-1
General District Operating Revenue and Expenses

Rate of Growth:	3.1%/Year
Rate of Inflation:	4%/Year
Operational Cost Increase:	5%/Year

Operating Revenues	Budget 1998/1999	Projected 1999/2000	Projected 2000/2001
Use Fees	$515,000	$530,965	$547,425
Permit Fees	2,240	2,309	2,381
Connection Fees	5,600	6,005	6,438
System Development Charges	6,384	6,582	6,786
Interest 590	13,290	13,702	--
Fund 594.34 Revenue	20,000	33,908	34,959
Fund 595.34 Revenue	52,000	103,377	106,582
Fund 595.34 Revenue	2,000	2,062	2,126
Interest 394, 395, and 396	56,100	57,839	--
Total Revenue	**$672,614**	**$756,749**	**$706,697**
Operating Expenses			
Salary/Wages	$111,000	$116,550	$18,558
Benefits	37,899	39,415	6,495
Supplies and Chemicals	29,991	31,191	63,510
Power	--	--	11,341
Collection System		--	62,891
Services	89,815	93,408	200
Intergovernment Payment for WWTP	101,540	105,602	91,221
Fund 594.34 Expend	--	--	--
Fund 595.34 Expend	--	--	--
Fund 596.34 Expend	103,746	51,873	51,871
Debt Service	12,800	12,800	12,800
Total Costs	**$486,791**	**$318,886**	**$278,302**

The last day of compost facility operation will be in October 1999. Currently, the biosolids generated at the Redwood WWTP are used as base material for compost generation. The current annual JoGro costs for materials, equipment, and services are approximately $40,000. For evaluation purposes, it was assumed that these costs would equal the new costs associated with transporting the biosolids to the Grants Pass WRP.

8.3 SYSTEM DEVELOPMENT CHARGES

System Development Changes (SDCs) are billed to new customers so they can pay their share of the sewer system capital costs. SDCs are imposed on new customers connecting to the system as a condition of service, in addition to any costs to connect the customer.

The SDC consists of two components: 1) A reimbursement fee that constitutes a buy-in to existing facilities already in place, and 2) an improvement fee that represents the additional cost of system expansion as planned by the utility. Improvement fees can only be used for capacity-expanding improvements, while reimbursement fees can be used for any planned capital improvements. Separate accounts for each fee component and budget-specific expenditures are in place. Reimbursement SDCs go to Account Number 594 and Improvement SDCs go to Account Number 595.

One benefit of this project being delayed from 1994 to 1999 while the compost facility issues were being resolved, is that the District has been able to accrue a large cash balance. This cash has been set aside until the facility upgrade capital works projects are ready to go. According to the District's 1998/1999 budget, the total cash balance from the four sewer accounts is approximately $2,050,990.

8.4 FUNDING ALTERNATIVES

The ODEQ's Wastewater Finance Office has already approved a $3,040,000 State Revolving Fund (SRF) Loan for the District's wastewater project. The District applied for this loan several years ago, and it was approved based on a 20-year payment period at 3.89 percent interest. Access to additional SRF funding is considered very likely. Based on this relatively low interest rate, and no 25 to 35 percent debt service coverage requirements, the District has decided that the project will be totally funded with SRF money. Below is a summary of what SRF funding includes:

> **State Revolving Fund** – This loan program is the successor to the Federal Clean Water Act grant program. Federal and State governments provide money to a revolving loan pool at a lower than market rate. The loans are for planning, designing, and construction of wastewater treatment facilities.

The District could complete a new SRF loan application for new funding, or they could ask for additional funding based on their approved application. A new funding request would need to go through the entire application selection process and compete with all other new applications. Applying for additional SRF funding based on their approved loan should be a relatively simple process, and would provide the greatest chance of success. Because the District is already approved for a $3.04 million loan, they have a high priority for receiving additional funding.

Although an SRF loan does not require debt coverage like bond funding, a special reserve account would be necessary. This reserve money, called a Required Loan Reserve, is placed in an account for the entire loan period and is used if the District misses a loan payment. The Required Loan Reserve would be equal to the average annual payment over the 20-year period.

Although this entire amount must be kept on reserve, it could earn interest. SRF loan payments are due on June 1 and December 1 of each year.

8.5 FINANCING ALTERNATIVES

This section presents a cash flow analysis of two project financing scenarios. The first scenario will be based on the same O&M cost and population growth rates used in the previous portions of this report. This is called the base case scenario. The second scenario assumes a reduction in the cost for wastewater treatment. The capital and O&M costs used for both scenarios were taken from Section 6 and assumes no increases in sewer fees or SDCs.

8.5.1 Base Case Scenario

The assumptions used for scenario one "Base Case" project finance cash flow spreadsheet are as follows:

- ERU growth rate of 3.1 percent per year
- An inflation rate of 4.0 percent per year
- A salary increase of 5 percent per year
- An SRF loan rate of 3.89 percent paid over a 20-year period

Based on this scenario, the District would be able to finance a project costing up to $6.79 million, including loan fees. A summary of the Base Case is presented in Table 8-2. The complete spreadsheet is included in Appendix M.

Table 8-2
Base Case Scenario
Total Project Funding Capability

Required Reserve Funds	
District Held Reserves	$305,000
SRF Loan Reserve	398,816
Total Reserves to be Held[1]	**$703,816**
Available Funds for the Project	
Available Cash	$1,347,175
SRF Loan Proceeds	5,440,000
Total Available Funding[2]	**$6,787,175**

(1) Total cash reserves that shall be held for the entire period.
(2) Total funding available to use for project design, construction, administration, etc.

8.5.2 Alternative One Scenario

The "Base Case Scenario" used an annual treatment cost of $88,500 to be paid to the City of Grants Pass for treatment at the Water Restoration Plant. This cost was based on a ratio of District customers to Grants Pass City WRP customers, and assumed all customers would pay an equal amount per gallon treated. In the Alternative One Scenario, however, the treatment

cost has been reduced to $38,500. This is the estimated incremental cost increase at the Grants Pass WRP to treat District wastewater and is lower than the average cost per gallon treated. Under this scenario, the District would be able to finance a project costing up to $7.4 million. A summary of the Alternative One Scenario is presented in Table 8-3.

Table 8-3
Alternative One Scenario
Total Project Funding Capability

Required Reserve Funds	
District Held Reserves	$225,000
SRF Loan Reserve	436,938
Total Reserves to be Held[1]	**$661,938**
Available Funds for this Project	
Available Cash	$1,389,052
SRF Loan	5,960,000
Total Available Funding[2]	**$7,349,052**

(1) Total cash reserves that shall be held for the entire loan period.
(2) Total funding available to use for project design, construction, administration, etc.

8.6 PROJECT FUNDING SUMMARY

Two project financing alternatives were reviewed for the District. In either, the District could finance sufficient monies to implement the preferred treatment alternative. The consensus by the District Board of Commissioners was, however, that the District's share for wastewater treatment costs at the Grants Pass WRP should be $88,500 per year, not $38,500 per year as evaluated in the Alternative One Scenario.

Based on this, the District has already began the process of requesting an additional $2,400,000 of SRF money. This money will be added to the original $3,040,000 that was approved in 1995. A preliminary project schedule, included at the end of Section 7, outlines how the funding acquisition needs to proceed.

900986

RESOLUTION NO. 4954

A RESOLUTION OF THE COUNCIL OF THE CITY OF GRANTS PASS ADOPTING A CAPITAL IMPROVEMENT PROGRAM FOR THE CITY'S WATER UTILITY.

WHEREAS:

1. In January, 2001, a Water Distribution Master Plan was completed and in May, 2004 a Water Treatment Plant Facility Plan was completed; and

2. Financial Consulting Solutions Group, Inc. and Parametrix, Inc. have reviewed the capital improvements recommended by the two plans, and have recommended priorities and schedules for construction; and

3. A technical memorandum has been prepared by Parametrix, Inc. on behalf of the City that summarizes the recommended capital improvement plan (CIP) for the Water Treatment and Water Distribution divisions; and

4. The recommended capital improvements must be adopted by Council to allow the adoption of new SDCs in the near future.

NOW, THEREFORE, BE IT RESOLVED by the Council of the City of Grants Pass, that the CIP recommended in the attached technical memo be adopted.

EFFECTIVE DATE. This resolution shall be effective immediately upon its passage by the City Council and approval by the Mayor.

ADOPTED by the Council of the City of Grants Pass, Oregon, in regular session this 4th day of May, 2005.

SUBMITTED to and Approved by the Mayor of the City of Grants Pass, Oregon, this 4th day of May, 2005.

Len Holzinger, Mayor

ATTEST:

Joanne M. Stamp
Administrative Services Director

Date Submitted to Mayor: 5/4/05

EXHIBIT 10
to UAIC Staff Report 98-19

900988

RESOLUTION NO. 4955

A RESOLUTION OF THE COUNCIL OF THE CITY OF GRANTS PASS ADOPTING A CAPITAL IMPROVEMENT PROGRAM FOR THE CITY'S WASTEWATER UTILITY.

WHEREAS:

1. In June, 2001 a Wastewater Facilities Plan Update was completed and in September, 2004 a Collection System Master Plan was completed; and

2. Financial Consulting Solutions Group, Inc. and Parametrix, Inc. have reviewed the capital improvements recommended by the two plans, and have recommended priorities and schedules for construction; and

3. A technical memorandum has been prepared by Parametrix, Inc. on behalf of the City that summarizes the recommended capital improvement plan (CIP) for the Wastewater Treatment and Collection System divisions; and

4. The recommended capital improvements must be adopted by Council to allow the adoption of new SDCs in the near future.

NOW, THEREFORE, BE IT RESOLVED by the Council of the City of Grants Pass, that the CIP recommended in the attached technical memo be adopted.

EFFECTIVE DATE. This resolution shall be effective immediately upon its passage by the City Council and approval by the Mayor.

ADOPTED by the Council of the City of Grants Pass, Oregon, in regular session this 4th day of May, 2005.

SUBMITTED to and Approved by the Mayor of the City of Grants Pass, Oregon, this 9 day of May, 2005

Len Holzinger, Mayor

ATTEST:

Joanne M. Stumpf
Administrative Services Director

Date Submitted to Mayor: 5/6/05

EXHIBIT 11
to UAPC, Staff Report 989

000990

Item: Resolution adopting a capital improvement program for the city's water utility. Date: May 4, 2005

RECOMMENDED ACTION:

It is recommended the Council adopt the capital improvement program for water utility.

PROCEDURE:

Follow procedure for a Resolution.

BACKGROUND:

A Water Distribution Master Plan was completed in January, 2001 and a Water Treatment Plant Facility Plan was completed in May, 2004. Each plan provides a list of capital construction projects and recommends timing for completion of the projects. The projects are designed to improve system efficiency, replace worn out and substandard equipment and piping, and accommodate service population growth.

To assure the ability to finance the projects, a study of water user rates and system development charges (SDCs) has been completed by Financial Consulting Solutions Group, Inc. and Parametrix, Inc. The construction projects have been combined into a capital improvement program (CIP), prioritized, and scheduled for construction. This information is summarized in a Technical Memorandum prepared by Parametrix, Inc. and is attached to this document.

New SDCs cannot be approved until the City Council adopts an updated CIP. Staff recommends adoption of the proposed Water Utility CIP.

RELATIONSHIP TO COUNCIL GOALS:

This action directly implements Council Goals of **PUBLIC SAFETY** and **MANAGEMENT** by providing for the upgrade, efficient operation, and expansion of the City's Water Treatment and Distribution System.

COST IMPLICATION:

Financing strategy for the Water System CIP will be considered at a future Council meeting where new water user rates and SDCs will be recommended for adoption.

ITEM: 2.b. RESOLUTION ADOPTING A CAPITAL IMPROVEMENT PROGRAM FOR THE CITY'S WATER UTILITY.

EXHIBIT 12 99:

000992

Item: Resolution adopting a capital improvement program for the city's wastewater utility. Date: May 4, 2005

RECOMMENDED ACTION:

It is recommended the Council adopt the capital improvement program for wastewater utility.

PROCEDURE:

Follow procedure for a Resolution.

BACKGROUND:

The Wastewater Facilities Plan Update was completed in June, 2001 and the Wastewater Collection System Master Plan was completed in September, 2004. Each plan provides a list of capital construction projects and recommends timing for completion of the projects. The projects are designed to improve system efficiency, replace worn out and substandard equipment and piping, and accommodate service population growth.

To assure the ability to finance the projects, a study of sewer user rates and system development charges (SDCs) has been completed by Financial Consulting Solutions Group, Inc. and Parametrix, Inc. The construction projects have been combined into a capital improvement program (CIP), prioritized, and scheduled for construction. This information is summarized in a Technical Memorandum prepared by Parametrix, Inc. and is attached to this document.

New SDCs cannot be approved until the City Council adopts an updated CIP. Staff recommends adoption of the proposed Wastewater Utility CIP.

RELATIONSHIP TO COUNCIL GOALS:

This action directly implements Council Goals of **PUBLIC SAFETY** and **MANAGEMENT** by providing for the upgrade, efficient operation, and expansion of the City's Wastewater Treatment and Collection System.

COST IMPLICATION:

Financing strategy for the Wastewater System CIP will be considered at a future Council meeting where new sewer user rates and SDCs will be recommended for adoption.

ITEM: 2.c. RESOLUTION ADOPTING A CAPITAL IMPROVEMENT PROGRAM FOR THE CITY'S WASTEWATER UTILITY.

EXHIBIT 13 in IWR Staff Report 993

000994

Parametrix ENGINEERING • PLANNING • ENVIRONMENTAL SCIENCES

TECHNICAL MEMORANDUM

Date: April 25, 2005
To: City of Grants Pass
From: Steven C. Gilbert, P.E.
Subject: Sewer and Water Utility Capital Programs
cc:
Project Number: 216-3416-028
Project Name: Sewer and Water Rate Analysis

INTRODUCTION

As part of the Sewer and Water SDC and Rate Analysis Task Order No. 21, the Sewer and Water Utility Capital Programs need to be identified to determine the appropriate Sewer and Water System Development Charge (SDC). The purpose of this Technical Memorandum is to present a Capital Improvement Program (CIP) for the following components of each of these utilities:

- Wastewater Treatment Plant, referred to as the "Water Restoration Plant" (WRP).
- Sewer Collection System, including transmission and pump/lift stations.
- Water Treatment Plant.
- Water Distribution System, including reservoirs, transmission mains, and water booster pump stations.

Much of the information presented has been taken from previous studies conducted by the City of Grants Pass on these utilities between 2001 and 2004. These studies will be referenced as the information is presented.

In addition, in order to determine the appropriate SDC for each utility, the ability of the present utility (asset value of the utility) to adequately serve the community for 20 years must be determined. Also the CIP for each utility must be further defined. For each element of the CIP it must be determined whether that element is required to support future growth within the UGB or is required to replace and/or repair the existing element due to deterioration. This Technical Memorandum will also document this information.

SEWER UTILITY

The sewer utility includes the sewer collection system that collects sewage from homes and commercial establishments located within the Urban Growth Boundary (UGB) of the City of Grants Pass. This system currently includes a total of about 0.8 million feet of sewer pipelines ranging in size from 4 inches

EXHIBIT 14 995
to IAPC, Staff Report

to 42 inches in diameter, two sewage lift stations (Webster No. 1 and No. 2), and the Bridge Street sewage pump station.

The utility also includes the Water Restoration Plant with a peak capacity of 27 million gallons per day (mgd) and a secondary treatment capacity of 16 mgd. This plant treats wastewater collected from the UGB and from the Redwood Sanitary Sewer Service District located on the west edge of the city. Treated wastewater is discharged to the Rogue River.

The JO-GRO compost facility located at the previous Merlin Landfill site is also part of the sewer utility. This facility is used to compost all of the biosolids generated at the Water Restoration Plant, together with green waste collected from the Grants Pass community.

COLLECTION SYSTEM CIP

In September 2004, the City of Grants Pass completed a Collection System Master Plan (Parametrix, Inc. 2004) which identified a Capital Improvement Plan for the system for the next 20 years. This plan identified $18.08 million dollars ($M) of improvements that were needed in the collection system in the next 20 years in order to adequately serve the community. Provided these improvements were completed, the existing collection system would be adequate to serve the community for the next 20 years. This Capital Improvement Plan is shown in Table 1.

Table 1. Capital Improvement Plan – Grants Pass Collection System

Project	Schedule	Cost ($1M)
Pine Street Sewer	2004–2006	$2.55
Western Avenue Sewer	2006–2009	$1.46
Pine Street Structural Repair	2009–2011	$1.11
5th Street Structural Repair	2010–2012	$2.13
7th Street Structural Repair	2013–2015	$1.46
Lawnridge-Washington Structural Repair		$1.64
Mill Street Sewer	2016–2018	$3.08
Subbasin B/C Structural Repair		$1.23
7th Street Relief System	2019–2021	$1.40
Subbasin H Structural Repair		$1.21
Nebraska Avenue Sewer	2022–2024	$0.81
Total Collection System Capital Improvement Plan:		**$18.08**

Two different types of elements were identified in the CIP, individual pipeline improvement projects (as an example Pine Street Sewer) and structural repair projects (as an example Pine Street Structural Repair).

All structural repair projects were required to repair/replace existing sewer pipelines that were severely deteriorated in the city. These projects were not required to support growth in the community.

The individual pipeline improvement projects, on the other hand, were necessary both to repair badly deteriorated existing sewer pipelines in the city and also to support growth in the community. To handle the additional wastewater flow capacity in the future, the diameter of these new pipelines is larger than the existing pipeline. The following table summarizes and rationalizes the cost percentage of each of these projects attributable to supporting growth in the community for the next 20 years.

Table 2. Percentage of Anticipated Project Costs Attributable to Growth Collection System Capital Improvement Plan

Project	Cost ($1M)	Percentage	Rationale
Pine Street	$2.55	54	Increase pipe size from 12 to 21 inches
Western Avenue	$1.46	86	Increase pipe size from 8 to 18 inches
Mill Street	$3.08	70	Increase pipe size from 12 to 18 inches
7th Street Relief	$1.40	86	Increase pipe size from 8 to 18 inches
Nebraska Avenue	$0.81	25	Increase pipe size from 15 to 18 inches

WATER RESTORATION PLANT CIP

In April 2000, the City of Grants Pass completed a Facility Plan Update for the Water Restoration Plant (Parametrix, Inc., 2000) which identified a Capital Improvement Plan for the plant for the next 20 years. This plan identified $17.18M of improvements that were needed at the plant in the next 20 years in order to adequately serve the community. This CIP is shown in Table 3.

Table 3. City of Grants Pass – Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001/2004	2005/2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping	$560,000		$560,000	
Raw Sewage Pipeline to Ballasted Sedimentation	$240,000		$240,000	
Screening Odor Control	$390,000	$390,000		
Mechanical Bar Screen No. 2	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000		$3,510,000	
Modify Existing Primary to Combination Clarifier/Thickener	$610,000		$610,000	
Yard Piping	$270,000		$270,000	
SECONDARY TREATMENT				
Aeration Basin Fine Bubble	$530,000	$530,000		
Aeration Basin Selector	$260,000	$260,000		
Blowers and DO Control	$870,000	$870,000		
Rehabilitate Existing Clarifiers	$770,000	$770,000		
New Secondary Clarifiers	$2,400,000	$1,200,000		$1,200,000
Yard Piping	$770,000	$400,000		$370,000
Motorized Gates	$340,000	$340,000		
FINAL TREATMENT				
Outfall Diffuser	$540,000	$540,000		
OTHER PLANT IMPROVEMENTS				
Lab Improvements	$100,000	$100,000		
Operation Building Repairs and Office	$130,000	$130,000		
SCADA System Expansion	$670,000	$250,000	$250,000	$170,000
Equipment Improvements from Audit Results	$250,000	$100,000	$75,000	$75,000
Plant Landscaping	$100,000	$40,000	$30,000	$30,000
Public Education Program	$50,000	$20,000	$20,000	$10,000

(Table Continues)

Table 3. City of Grants Pass – Water Restoration Plant Capital Improvement Plan (Continued)

Project Item	Probable Cost	Construction Schedule		
		2001/2004	2005/2006	2010-2011
SOLIDS THICKENING AND DIGESTION				
Rehabilitate Existing Digester	$740,000	$740,000		
Dewatering Centrifuge	$1,000,000	$1,000,000		
SOLIDS HANDLING OFF-SITE				
Co-composting Facility	$1,850,000	$1,850,000	$	$
TOTALS	**$17,180,000**	**$9,530,000**	**$5,795,000**	**$1,855,000**

Between 2000 and 2004, part of these planned CIP improvements were completed in the Phase 1 Upgrade of the Water Restoration Plant (Parametrix, Inc., 2001). However, the remaining planned improvements to the plant as shown in the following table were never completed.

Table 4. City of Grants Pass – Remaining Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001/2004	2005/2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping	$657,000		$657,000	
Raw Sewage Pipeline to Ballasted Sedimentation	$282,000		$282,000	
Mechanical Bar Screen No. 2	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$4,121,000		$4,121,000	
Modify Existing Primary to Combination Clarifier/Thickener	$716,000		$716,000	
Yard Piping	$317,000		$317,000	
SECONDARY TREATMENT				
Rehabilitate Existing Clarifiers	$904,000	$904,000		
New Secondary Clarifiers	$1,409,000			$1,409,000
Yard Piping	$434,000			$434,000
OTHER PLANT IMPROVEMENTS				
SCADA System Expansion	$494,000		$294,000	$200,000
Equipment Improvements from Audit Results	$313,000		$225,000	$88,000
Plant Landscaping	$90,000		$60,000	$30,000
Public Education Program	$35,000	$	$23,000	$12,000
TOTALS	**$10,002,000**	**$904,000**	**$6,925,000**	**$2,173,000**

Amounts shown in Table 3 represent year 2000 cost. In Table 4, the anticipated cost of each remaining item to be completed has been increased to reflect year 2005 costs using the Engineering News Record (ENR) Construction Cost Index increase between 2000 and 2005.

In the development of the CIP for the Water Restoration Plant presented above, the impacts to the plant as a result of a Total Maximum Daily Load (TMDL) study of the Rogue River were unknown. At that time, the study had not yet been started; however, it was known that this study would be initiated in about 2005 and that it would result in impacts to the plant. Possible impacts to the plant might include more stringent

BOD and TSS limitations to the plant effluent and even new nutrient and temperature limitations in the plant effluent were possible. Any of these possible impacts were impossible to predict at that time.

For this reason, any additions or changes in the existing aeration basin at the plant to accommodate these more stringent or new limitations were not included in the CIP.

Recently, however, the TMDL study has been started in 2005. In a meeting with the Oregon Department of Environmental Quality, preliminary information on the potential impacts of the TMDL on the Water Restoration Plant has now been described to the City of Grants Pass. This information has resulted in a need to expand the previous CIP for the Water Restoration Plant as presented in the 2000 Facility Plan to include three new projects. Also, a fourth new project for the plant CIP has now been identified regarding the JO-GRO Compost Facility that must be added. These additional CIP projects, their cost, and anticipated schedule are shown in the following table. Costs shown in this table represent year 2005 dollars.

Table 5. Additional Capital Improvement Plan Projects
Grants Pass Water Restoration Plant

Project Item	Probable Cost	Construction Schedule 2007-2009 Cost
Aeration Basin Expansion	$2,400,000	$2,400,000
Effluent Filtration	$1,700,000	$1,700,000
Plant Biosolids Improvements	$700,000	$700,000
JO-GRO Expansion	$800,000	$800,000

These projects need to be added to the above Table 4 to provide a complete CIP for the Water Restoration Plant for the next 20 years. Together these improvements will provide a wastewater treatment plant capable of adequately serving the Grants Pass community for the next 20 years.

The improvements described in Tables 4 and 5 are all needed to support growth in the city. Certain components of the existing plant are also capable of supporting this growth and do not require new or additional expansion or upgrade. The components of the existing WRP that can accommodate growth for the next 20 years and the cost attributable to this growth are identified in Table 6.

Table 6. Cost of Existing WRP Treatment Components Attributable to Growth
Grants Pass Water Restoration Plant

Project Item	Total Cost ($M)	Cost Attributable to Growth ($M)
WRP Offices and Laboratory	$2.5	$0.5
Biosolids Dewatering	$2.0	$0.5
UV Disinfection	$2.5	$1.5
Outfall Diffuser	$1.0	$0.8
JO-GRO Facility	$2.2	$1.0

The capacity of the existing WRP without the CIP improvements described in Tables 4 and 5 can only accommodate the growth in the City of Grants Pass for the next 3 to 5 years provided careful attention is given to plant operation to maximize treatment removal efficiency. To maximize treatment, removal efficiencies may include installing chemical addition to the existing secondary clarifiers. This may be needed to enhance solids settling during peak flow and waste load events.

WATER UTILITY

The water utility includes the Water Treatment (filtration) Plant that treats water pumped from the Rogue River for the residential and commercial establishments in the City of Grants Pass. The plant removes suspended particulates; removes and inactivates pathogens; and produces a non-corrosive, palatable water meeting all regulations and Federal and State drinking water standards. The plant capacity is 18.0 million gallons per day (MGD), adequate to serve the community until about the year 2024. Of that capacity, the existing customer base uses approximately 10.5 MGD on a peak day basis.

Solids from the water filtration process are collected and discharged to a settling pond located next to the skate board park.

The water utility also includes a water distribution system that consists of about 150 miles of water pipelines. The distribution system includes eight reservoirs varying in size from 0.8 million to 1.5 million gallons that supply fire storage to customers, four water booster pumping stations which serve eight different pressure zones in the system, and 1,378 fire hydrants for fire protection.

Water Treatment Plant CIP

In May 2004, the City of Grants Pass completed a Facility Plan for the water treatment plant (CDM, 2004) that identified $4.335M in improvements required at the plant. These improvements were necessary in order for the plant to supply water to the city for the next 20 years. Provided that these improvements are completed, the plant will be adequate to serve the community for 20 years depending on the outcome of current monitoring programs. The Capital Improvement Plan identified in this Facility Plan is shown in the following table.

Table 7. Near-Term Implementation Plan for WTP Improvements

Fiscal Year	Improvements	Estimated Project Costs
2004/2005	1. Intake Modifications (Engr. and Permitting)	$400,000
2005/2006	1. Intake Modifications (Engr. and Construction)	$500,000
	2. Filter Upgrades (Engr. and Construction)	$200,000
	3. Basin Modifications (Engr. and Construction)	$200,000
2006/2007	1. Intake Modifications (Construction)	$700,000
	2. Filter Upgrades (Construction)	$400,000
	3. Basin Modifications (Construction)	$400,000
2008/2009	1. Filter Gallery Upgrades (Engr. and Construction)	$480,000
	2. Solids Handling	$800,000
2009/2010	1. Filter Gallery Upgrades (Construction)	$510,000
	2. Chemical System Upgrades (Engr. and Const.)	$53,000
2010/2011	1. Chemical System Upgrades (Construction)	$138,000
	2. Sludge Removal Systems (Engr. and Const.)	$80,000
	3. New Storage Building (Engr. and Construction)	$27,000
2011/2012	1. Sludge Removal Systems (Construction)	$239,000
	2. New Storage Building (Construction)	$53,000
2012/2013	1. Emergency Generator for 5 mgd (Engr. and Const.)	$319,000
2024	1. Expand Capacity to 30 mgd	$7,813,000
	TOTAL	$13,312,000

001000

In addition to these improvements, recently the plant has tested a bagged dewatering process to remove solids from the settling pond. This process has proven to be very costly and a different approach to dewatering solids from the pond is necessary in the near future. It is estimated that the cost of a new dewatering process will be $800,000 and this CIP project has been added to the CIP shown in Table 7. It is anticipated that a new solids dewatering process will be necessary in the year 2008.

Water Distribution System CIP

In January 2001, the City completed a Water Distribution System Master Plan Update (West Yost and Associates, 2001). This plan identified over $20.0M in improvements through the year 2020 to support expansion of the distribution system; to replace old, undersized cast iron water mains; and to address service areas that do not meet the City's performance standards.

The water distribution system CIP from this plan is shown in Tables 8 through 11. Provided these improvements are completed in the next 20 years, the distribution system, including reservoirs and booster pumping station, is adequate to serve the community.

Table 8. Estimated Capital Costs for CIP Projects (2001 Dollars) – Period 2000–2005

Recommended Improvements	Capital Cost $1,000
Pump Stations	
Hilltop/Harbeck Heights Fire Pumps	$60
Meadow Wood Pump Station	255
Laurelridge Pump Station	245
Rogue Community College Pump Station	245
Subtotal	$805
Pipelines	
Pressure Zone Boundary Modifications	$100
Pressure Reduction Valves	130
P-101 West Harbeck to Allen Creek	54
P-102 Ringuette Street	129
P-103 Leonard Street Looping	40
P-104 Lower River Road	149
P-105 Prospect Avenue Looping	54
P-106 Hawthorne to Crescent	748
P-107 9th to 10th at Midland	122
P-108 Sherman Lane to Tokay Heights	227
P-109 Marion Lane	115
P-110 C Street to D Street Loop	49
P-111 Hamilton Lane Looping	134
P-112 Agness to Gladiola	119
P-113 Bridge to Brownell	431
P-114 Lincoln Road Looping	83
P-115 10th and Savage Tie-In	7

(Table Continues)

Table 8. Estimated Capital Costs for CIP Projects (2001 Dollars) – Period 2000–2005 (Continued)

Recommended Improvements	Capital Cost $1,000
P-201 Redwood Avenue Extension	370
P-202 Redwood Avenue Looping North	319
P203 Redwood Avenue Looping South	221
P-204 Rogue Community College Extension	577
P-205 Allen Creek Connector	869
P-207 Williams Highway Extension	152
P-210 West Harbeck Road Connector	318
P-212 Pedestrian Bridge Connector	353
P-220 Southeast North Street Extension	113
P-221 Shannon Lane Extension	48
P-222 Lincoln Road Extension	103
P-223 Ament Road Extension	660
P-224 Starlite Connector	290
P-227 Valley View Road	25
Subtotal	$7,109
TOTAL	**$7,914**

Table 9. Estimated Capital Costs for CIP Projects (2001 Dollars) – Period 2005–2010

Recommended Improvements	Capital Cost $1,000
Pipelines	
P-208 Williams Highway Looping	$294
P-214 Rogue River Highway Extension	655
P-225 Starlite Extension	100
Subtotal	$1,049
Pipeline Replacement	
12,500 Feet Total Replacement	$1,124
Subtotal	$1,124
TOTAL	**$2,173**

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Table 10. Estimated Capital Costs for CIP Projects (2001 Dollars) – Period 2010–2020

Recommended Improvements	Capital Cost $1,000
Treated Water Storage	
Reservoir No. 12	$1,710
Reservoir No. 14	850
Reservoir No. 16	960
Reservoir No. 17	1,350
Reservoir No. 13 (replacement)	850
Subtotal	$5,720
Pipelines	
P-206 Reservoir No. 12 Extension	$479
P-209 Reservoir No. 17 Extension	97
P-215 Fruitdale Drive Extension	780
P-216 Cloverlawn Loop	56
P-218 Cloverlawn to Crestview Loop	142
P-219 Reservoir No. 16 Extension	181
P-226 Greenfield Road Loop	239
P-229 Reservoir No. 14 Extension	68
Subtotal	$2,042
Pipeline Replacement	
25,000 Feet Total Replacement	$2,248
Subtotal	$2,248
TOTAL	**$10,010**

Table 11. Estimated Capital Costs for CIP Projects (2001 Dollars) – Period Post 2020

Recommended Improvements	Capital Cost $1,000
Treated Water Storage	
Reservoir No. 10	$1,560
Subtotal	$1,560
Pump Stations	
Treatment Plant Pumps	$400
Subtotal	$400
Pipelines	
P-217 Reservoir No. 10 Extension	$184
Subtotal	$184
TOTAL	**$2,144**

A number of these projects have already been completed since the plan was prepared in 2001. In addition, the costs shown in Tables 8 through 11 represent year 2001 construction cost. In Tables 12 through 15, the projects already completed have been removed and project costs increased to year 2005 construction cost based again on the ENR Construction Cost Index increase between 2001 and 2005. The current Capital Improvement Plan for the water distribution system is shown in Tables 12 through 15.

Table 12. Estimated Capital Costs for CIP Projects (2005 Dollars) – Period 2000–2005

Recommended Improvements	Capital Cost $1,000
Pump Stations	
Hilltop/Harbeck Heights Fire Pumps	$60
Rogue Community College Pump Station	280
Subtotal	$305
Pipelines	
Pressure Zone Boundary Modifications	$114
Pressure Reduction Valves	149
P-101 West Harbeck to Allen Creek	62
P-103 Leonard Street Looping	46
P-104 Lower River Road	86
P-105 Prospect Avenue Looping	16
P-106 Hawthorne to Crescent	427
P-107 9th to 10th at Midland	139
P-108 Sherman Lane to Tokay Heights	259
P-109 Marion Lane	131
P-110 C Street to D Street Loop	56
P-113 Bridge to Brownell	325
P-114 Lincoln Road Looping	95
P-115 10th and Savage Tie-In	8
P-202 Redwood Avenue Looping North	200
P203 Redwood Avenue Looping South	202
P-204 Rogue Community College Extension	4,372
P-207 Williams Highway Extension	53
P-220 Southeast North Street Extension	26
P-221 Shannon Lane Extension	55
P-222 Lincoln Road Extension	118
P-223 Ament Road Extension	754
P-224 Starlight Connector	290
Subtotal	$7,983
TOTAL	**$8,288**

Table 13. Estimated Capital Costs for CIP Projects (2005 Dollars) – Period 2005–2010

Recommended Improvements	Capital Cost $1,000
Pipelines	
P-208 Williams Highway Looping	$336
P-214 Rogue River Highway Extension	749
P-225 Starlite Extension	114
Subtotal	$1,199
Pipeline Replacement	
12,500 Feet Total Replacement	$1,124
Subtotal	$1,124
TOTAL	**$2,323**

001004

Table 14. Estimated Capital Costs for CIP Projects (2005 Dollars) – Period 2010–2020

Recommended Improvements	Capital Cost $1,000
Treated Water Storage	
Reservoir No. 12	$1,955
Reservoir No. 14	972
Reservoir No. 16	1,097
Reservoir No. 17	1,543
Reservoir No. 13 (replacement)	972
Subtotal	$6,539
Pipelines	
P-206 Reservoir No. 12 Extension	$488
P-209 Reservoir No. 17 Extension	111
P-215 Fruitdale Drive Extension	892
P-216 Cloverlawn Loop	64
P-218 Cloverlawn to Crestview Loop	162
P-219 Reservoir No. 16 Extension	207
P-226 Greenfield Road Loop	273
P-229 Reservoir No. 14 Extension	78
Subtotal	$2,275
Pipeline Replacement	
25,000 Feet Total Replacement	$2,248
Subtotal	$2,248
TOTAL	**$11,062**

Table 15. Estimated Capital Costs for CIP Projects (2005 Dollars) – Period Post 2020

Recommended Improvements	Capital Cost $1,000
Treated Water Storage	
Reservoir No. 10	$1,783
Subtotal	$1,783
Pump Stations	
Treatment Plant Pumps	$457
Subtotal	$457
Pipelines	
P-217 Reservoir No. 10 Extension	$210
Subtotal	$210
TOTAL	**$2,450**

Provided the improvements shown in Tables 12 through 15 are completed in the next 20 years, the distribution system, including reservoirs and booster pumping, is adequate to serve the community.

From: "MICHAEL SNIDER" <MSNIDER@co.josephine.or.us>
To: "Voice, Jared" <jvoice@grantspassoregon.gov>
CC: "DeJanvier, Charles" <CDEJANVIER@co.josephine.or.us>
Date: 6/3/2008 2:11 PM
Subject: Re: Public Facilities Plan amendments

Jared,

I talked to Chuck DeJanvier at public works and he says the water and sewer elements are not of direct concern to them; they will continue to focus on "surface" issues such as drainage, erosion, road improvements, etc., as those issues come up.

If your willing, you can forward final electronic copies of the elements to our departments when they are adopted.

Thanks,

Michael

>>> "Jared Voice" <jvoice@grantspassoregon.gov> 6/2/2008 10:09 AM >>>

Nora / Michael:

Good morning! I just wanted to verify that the County has no comments regarding the updated Water and Sewer sections of Grants Pass Comprehensive Plan Element 10 (Public Facilities and Services.) This proposal was on the Site Plan Review agenda for April 29, 2008 and is scheduled for hearing at the Urban Area Planning Commission on June 11, 2008.

Please advise me as to whether Josephine County will be providing comment on the proposed amendments.

Thank you-

Jared Voice
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EXHIBIT 15
to UAPC, Staff Report 10/10/17

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The irrigation season typically lasts from about April 15 to October 1. Examination of flows received at the treatment plant shows that the influent flows increase during those months, even though little precipitation occurs during that period. It appears that one source of infiltration and inflow in the Grants Pass sewage treatment plant is irrigation water that has seeped into the ground and infiltrated the sewer system. Although there is a court order to remove Savage Rapids Dam (now underway), GPID will continue to function as it does now through the installation of pumps that will deliver water to the district's canal system.

10.30.3.4 Hydraulic and Biologic Loading

The *2001 Wastewater Facilities Plan Update* made flow projections based on existing wastewater flow, plus a typical flow per capita assumed for all future connections. These future flow factors in gallons per capita per day (gpcd) are shown in Table 10.30.4. The per capita flow assumed for future connections was typical for new construction at the time. Table 10.30.5 shows the wastewater flow projections made through the 2020 planning period. **NOTE:** The population figures shown in Table 10.30.5 were extracted from the *2001 Wastewater Facilities Plan Update* and include the Harbeck-Fruitdale Sewer Service District, the Redwood Sanitary Sewer Service District, and the Merlin / North Valley area.

Table 10.30.4 Future Flow Factors

Flow Type	Contribution (gpcd)
Average Flow	105
Maximum Month Dry Weather Flow (MMDWF)	100
Maximum Month Wet Weather Flow (MMWWF)	130
Peak Day	275
Peak Wet Weather Flow	360
Summer Average	85
Winter Average	125

Source: Waste Water Facilities Plan Update, Parametrix, 2001

Table 10.30.5 Wastewater Flow Projections

	YEAR					
Types of Flow	1999 (Pop. 42,900)		2010 (Pop. 52,200)		2020 (Pop. 62,700)	
	Calculated (mgd)	Calculated (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)	Design (mgd)	Design (Gal/cap-day)
Summer Max. Month	7.3	173	8.4	160	9.4	150
Winter Max. Month	10.8	256	12.2	233	13.5	216
Peak Day	21.5	514	24.6	471	27.5	438
PWWF	30.1	715	34.0	652	37.8	603
Summer Average	5.0	120	6.0	113	6.8	109
Winter Average	6.4	155	7.80	149	9.1	145
Annual Average	5.7	137	6.9	131	8.0	127

Source: Waste Water Facilities Plan Update, Parametrix, 2001

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Table 10.30.6 Organic and Solids Loading (pounds per day)

Type of Loading	Year		
	1999	2010	2020
Average BOD_5	7,490	9,400	11,400
Maximum Monthly BOD_5	10,985	13,700	16,800
Average NH_3-N	327	390	450
Maximum Monthly NH_3-N	636	760	870
Average TSS	8,564	10,500	12,700
Maximum Monthly TSS	11,623	14,300	17,300

Source: Waste Water Facilities Plan Update, Parametrix, 2001

Because the Rogue River is used for salmon spawning, fish passage, and rearing Coho, wastewater discharge requirements are strict. To properly evaluate upgrading/expanding the Grants Pass WRP, an analysis of the facilities wastewater discharge permit limit is necessary. Table 10.30.6 presents the anticipated year 2020 BOD_5 and TSS treatment requirements as mandated by OAR 340-41-375. These values represent the dry weather season BOD_5 and TSS effluent concentrations that the plant should meet monthly.

Table 10.30.7 Anticipated Year 2020 BOD_5 and TSS Treatment Requirements Based on OAR 340-41-375

	Flow (mgd)	Effluent Concentration (mg/L)			Mass Discharge, lbs/day		
		Monthly	Weekly	Daily	Monthly	Weekly	Daily
Permit Limits Based on Effluent Quality							
Summer, Dry Weather	8	10	15	-	670	1,000	1,300
Winter, Wet Weather	10	30	45	-	2,500	3,750	5,000

Source: Waste Water Facilities Plan Update, Parametrix, 2001

The DEQ has determined that the Rogue River is a high priority concern for implementing management strategies to attain compliance with water quality standards. The first step in this process is to develop the total maximum daily load (TMDL) pollutant loading allocations. This TMDL will include both point and non-point sources.

The TMDL process is now underway and is currently scheduled to be completed in the next several years. Following TMDL allocation, new discharge permit requirements may be established for the Grants Pass WRP that will modify Table 10.30.7. These new permit requirements may also impact the capacity of the existing plant.[1]

[1] Per Steve Gilbert, Parametrix, June 2007

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10.30.4 CITY OF GRANTS PASS SANITARY SEWER SERVICES

10.30.4.1 Service Area

Situated near the Rogue River, the Grants Pass Water Restoration Plant (WRP) serves the City and the Harbeck-Fruitdale area. In the fall of 2000, the plant began receiving sewage from the Redwood area.

The *2004 Collection System Master Plan* estimated that the Grants Pass WRP had a service area population equivalent of approximately 44,250 in 2003. With estimated growth rates of 1.5 percent for Grants Pass and 1.6 percent for Harbeck-Fruitdale, and the addition of Redwood at a 3.1 percent population growth rate, it was estimated that the Grants Pass WRP would be serving an equivalent of 60,157 people by the year 2020.

10.30.4.2 Treatment Plant

Since 1935, the City of Grants Pass WRP has been operating at its current site. Subsequent plant additions occurred in 1953 and 1962. In 1974, the treatment plant was renovated and expanded. More improvements occurred from 1994-96 in response to the Department of Environmental Quality's concerns about the effluent toxicity, and the discharging directly into the Rogue River. The improvements consisted of a fourth raw sewage pump, a temporary belt filter press to improve wet weather solids disposal, two rectangular primary clarifiers, a computer based supervisory control and data acquisition system, and an ultraviolet disinfection system. In January 1999, Brown & Caldwell completed a facilities plan for the Grants Pass Water Restoration Plant (WRP). Also in 1999, the addition of a second Ultraviolet channel was implemented. This allowed the increase of disinfection capacity from 21.5 million gallons per day (mgd) to 43 mgd.[2]

10.30.4.3 Treatment Level

At the time of the *2001 Wastewater Facilities Plan Update*, the Grants Pass WRP treated 4.5 mgd of average dry weather flow (ADWF), with a record peak storm flow of 26.5 mgd. More recent data has also been compiled. As of April 2008, the WRP treated an ADWF of 5.5 mgd and peak wet weather flow of 30.0 mgd. The WRP is comprised of numerous unit treatment processes for both liquid and solid streams, a control/laboratory building, and a maintenance shop. There is a 27 mgd hydraulic capacity for influent pumping[3], screening, and primary treatment, a 13 mgd hydraulic capacity for secondary treatment, and a 43 mgd hydraulic capacity for UV disinfection. Flow exceeding the secondary treatment capacity receives only primary treatment and disinfection. This occurs only a few days a year during wet weather storm conditions.

10.30.4.4 Biosolids Handling and Disposal

Currently, the Grants Pass WRP sends the Class B Biosolids to JO-GRO™, a green waste and biosolid waste composting plant.

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[2] Information from this section was excerpted from the *2001 Wastewater Facilities Plan Update*. Additional WRP upgrades have occurred since that time (see Water Restoration Plant Capital Improvement Plan, Table 10.30.11).

[3] As of January 2008 a project was completed to increase the hydraulic capacity for influent pumping to 45 mgd.

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To produce Class B biosolids, processing must occur. The process starts with a primary clarifier. Solids, which are thickened in the gravity thickener and the gravity belt thickener, are dewatered by the secondary clarifier to produce waste activated sludge. The primary and secondary sludge are then combined and sent to the 50-foot-diameter anaerobic digester. Dewatered biosolids are loaded into a dumpster and hauled to the Merlin Landfill for the JO-GRO™ composting process.

10.30.4.5 Collection System

Per the *2001 Wastewater Facilities Plan Update*, it was believed that prior to 1927 sewer pipes were constructed of vitrified clay, although records are not available to confirm this. From 1927 to 1964, sewers were constructed of non-reinforced concrete pipe with bell and spigot joints caulked with cement mortar. Since 1964, sewers have been constructed of concrete pipe with bell and spigot joints and rubber ring gaskets. The concrete pipes are caulked at the joints with cement mortar. Over time, the cement caulking dissolves leaving the joints vulnerable to cracking and resulting in infiltration of groundwater and penetration of tree roots.

At the time of the *2001 Wastewater Facilities Plan Update* the City's collection system consisted of approximately 110 miles of gravity sewers, one force main (approximately 2,000 feet long), and three pumping stations. As of 1983, three hundred fifty acres of the downtown area were still served by vitrified clay pipe installed prior to 1927. The portion of the collection system constructed prior to 1964 has shown severe deterioration in both pipe materials and joint integrity. From 1992 to 2001, the city reported 17 sinkholes due to pipe failure.

The City of Grants Pass completed a *Collection System Master Plan* in 1983 (James Montgomery, 1983). A number of the 1983 recommendations for improving the collection system have been implemented. The City has, however, experienced sewer line failures and occasional overflows due to sewer line obstructions. The City has also expanded its collection system to include the Redwood Sanitary Sewer Service District (RSSSD) collection system located west of the City. The Oregon Department of Environmental Quality made completion of an updated Collection System Master Plan a provision in the National Pollutant Discharge Elimination System (NPDES) permit for the City's Water Restoration Plant, issued in December of 2000.

The *2004 Collection System Master Plan* is the most recent in a series of reports/studies focusing on wastewater infrastructure. In 2000, the City commissioned a *Wastewater Facilities Plan Update* (Parametrix, 2001) for improvements to the City WRP. The *Facilities Plan* focused on the WRP but also provides some analysis of the existing collection system to assess the impact of infiltration and inflow (I/I) on peak flow events at the WRP. Additional collection system analyses that were presented in the *RSSSD Facilities Plan* (prepared in 1999 by Parametrix) assessed the RSSSD wastewater collection system. In September 2004, Parametrix prepared the City's latest *Collection System Master Plan*, which provides for the capital improvement programming (see tables 10.30.11, 10.30.14, and 10.30.15) to accommodate orderly and cost-effective methods to operate, maintain and expand the collection system while reducing the risk of system failures.

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10.30.4.6 Pump Stations

The topography of the City's service area is such that most of the system is operated under gravity flow conditions. As such, there are only a few pump stations in the collection system. The Webster No. 1 Lift Station, Webster No. 2 Lift Station, and Bridge Street Pump Station are all located in the southwestern portion of the city. Under an intergovernmental agreement, the City also operates two pump stations serving the RSSSD: the RSSSD pump station, located at the site of the abandoned RSSSD wastewater treatment plant at 4960 Leonard Road, and the Darneille Pump Station at 3100 South River Road.

10.30.4.6.1 City Pump Stations: The collection system includes three pumping stations, two of which are simply lift stations. Design data for all three pump stations is included in Table 10.30.8. The lift stations, Webster Lift Station 1 and Webster Lift Station 2, are both located on Webster Lane and serve the mobile home park in the western section of Basin A. Each station is equipped with two vertical, non-clog, and centrifugal pumps. The pumps in Station 1 are 7.5 hp pumps, each with a capacity of 100 gallons per minute (gpm) at 23 feet total dynamic head (TDH). Station 2 pumps are 3 hp pumps, each with a capacity of 100 gpm at 10 feet TDH.

The third station, designated the Bridge Street Station, is located at the intersection of Bridge Street and Tami Court. Dual (4- and 8-inch diameter) force mains travel east from the pump station along Bridge Street about 1,900 feet and discharge in Manhole B111. The station is equipped with two submersible non-clog centrifugal pumps. Each of the 20 hp pumps has a capacity of 650 gpm at 75 feet TDH. Air injection is also provided for the force main to control sulfides. A 50 kW natural gas fueled engine generator is provided standby power.

10.30.4.6.2 Redwood Sanitary Sewer Service District (RSSD) Pump Stations: The Redwood Conveyance System, which transfers all flow from the old Redwood Wastewater Treatment Plan (WWTP) to the Grants Pass WRP, includes the Redwood Pump Station, the Redwood force main, the Darneille force mains and Darneille Pump Station, and a gravity sewer which brings the transferred flow to the Grants Pass WRP. The Redwood Pump Station is located at the old Redwood WWTP. It is a duplex submersible pump station with a capacity of 0.48 mgd, based on one pump in operation. The station has a Bioxide chemical injections system with a 3,000-gallon chemical storage tank. From the Redwood Pump Station, flow is routed through approximately 10,300 feet of 6-inch-diameter force main to an influent manhole at the Darneille Pump Station.

The Darneille Pump Station accepts the majority of the flow from the RSSSD, as well as the flow pumped from the Redwood Pump Station. The Darneille Pump Station has a firm capacity of 4.2 mgd, based on operation of two of three pumps. The station is a wet well/dry well type station with above-grade electrical panels, generator, and chemical feed system. The chemical feed system is identical to that provided at the Redwood Pump Station, except that the chemical feed pumps are slightly larger. From the Darneille Pump Station, flow is pumped through approximately 17,740 feet of dual 12-inch force main and the 1,000 feet of single 14-inch force main. From the pump station, the dual force mains are routed south to South River Road; then east through road rights-of-way and easements to the south side of the Pedestrian Bridge. The dual 12-inch force mains join into a single 14-inch force main that crosses the Rogue River on the Pedestrian Bridge and then discharges into a gravity sewer, which flows to the WRP. Table 10.30.8 provides pump station data.

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Table 10.30.8 Pump Station Data

Location	Year Built	No. of Pumps	Pump Type	Horsepower (hp)	Drive Type	Capacity/Head (gpm)/(feet)
Webster No. 1 Lift Station: East edge of Roguelea Estates	1967	2	Self-priming, vertical close-coupled, non-clog centrifugal	7.5	Constant Speed	100/23
Webster No. 2 Lift Station: All Sports Park -Basin A	1967	2	Self-priming, vertical close-coupled, non-clog centrifugal	3	Constant Speed	100/10
Bridge Street Pump Station: Bridge Street and Tami Court – Basin A	1994	2	Submersible, non-clog centrifugal	20	Variable Speed	650/75
Redwood Pump Station: 4960 Leonard Road – RSSSD	2000	2	Submersible, non-clog, centrifugal	40	Variable Speed	335/163
Darneille Pump Station: 3100 South River Road - RSSSD	2000	3	Immersible dry-pit, screw centrifugal	110	Variable Speed	1,460/180

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004

10.30.5 TREATMENT ALTERNATIVES CONSIDERED

In January 1999, Brown and Caldwell (BC) completed a Facilities Plan (FP) for the Grants Pass Water Restoration Plant (WRP). A concern regarding the future flow and population projections prompted a Value Engineering (VE) Workshop in June 1999. During this VE Workshop with Parametrix, Inc. and the City, these projections were recalculated and other treatment alternatives for both the liquid and solid streams were conceptualized. Also during the VE Workshop, the projected population, flow, Biological Oxygen Demand (BOD), and Total Suspended Solids (TSS) loadings were recalculated. These recalculations and treatment alternatives were incorporated into the *2001 Grants Pass Wastewater Facilities Plan Update* (WFP).

To select the best method for meeting Grants Pass treatment needs in the year 2020, the WFP evaluated three liquid stream alternatives and four solids streams alternatives. The liquid stream treatment includes a conventional expansion alternative, a ballasted sedimentation alternative, and a Zenon process alternative. The biosolids disposal and handling alternatives propose a Merlin Landfill co-compost facility, hauling dewatered biosolids to the Dry Creek Landfill, land application of Class B biosolids, and using an Aerobic Thermophilic Pretreatment (ATP) component. The alternatives are summarized below.

10.30.5.1 Liquid Stream Alternatives

10.30.5.1.1 Upgrades Common to the Liquid Stream Alternatives. All of the liquid stream treatment alternatives have several component upgrades in common. At the headworks, an additional mechanical bar screen would be installed with a 23.5 mgd capacity. This provides redundancy and a maximum capacity of 47 mgd, which is well beyond the projected future flows. An outfall diffuser would be added to improve and reduce the ammonia toxicity into the

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Rogue River. Several miscellaneous plant improvements, which existed in the Facility Plan, have also been included in the value engineering alternatives. These improvements include laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion.

10.30.5.1.2 Alternative One – Conventional Expansion. Alternative One is the recommended improvement from the WFP using a conventional expansion approach. By replicating the existing components, this option allows continuing ease of operation due to staff familiarity of the process. This alternative has been modified to treat the peak wet weather flow. A summary of the upgrades include:

- Additional mechanical bar screen with a capacity of 23.5 mgd.
- Removal of the four existing influent pumps and replacement with three 19 mgd pumps
- Additional rectangular primary clarifier. Rehabilitate existing primary clarifiers.
- Two additional aeration basins. Rehabilitate existing aeration basins.
- Two additional 115-foot secondary clarifiers. Rehabilitate existing secondary clarifiers.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, and instrumentation and control system expansion.

10.30.5.1.3 Alternative Two – Ballasted Sedimentation. Alternative Two uses ballasted sedimentation to treat peak flows greater than 13.5 mgd. It is proposed to convert the existing gravity thickener into the ballasted sedimentation tank. The peak flows would be conveyed to this system and then recombined with the main treatment train to receive UV disinfection. Other upgrades to the Grants Pass WRP include:

- Install additional influent pumping capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.
- Odor containment at the influent pump station and mechanical bar screen area,
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary clarifiers would be rehabilitated.
- Addition of a bioselector in the aeration basin to provide filamentous bacteria control, which would improve the settling performance in the secondary clarifiers. The aeration basin would also be modified with fine bubble diffusers, dissolved oxygen control, and motorized gates.
- Two additional 90-foot secondary clarifiers. Rehabilitate the secondary clarifiers.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements: laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

10.30.5.1.4 Alternative Three – Zenon Process. Alternative Three incorporated the use of Zenon for secondary treatment. The wastewater can flow directly into the primary clarifiers, thereby eliminating the need for secondary clarifiers. Zenon is a microfiltration membrane system located in a suspended growth biological reactor. For this alternative, the membranes would be placed into the aeration basin to serve as a biological reactor. Other upgrades to the Grants Pass WRP for this alternative include:

- Additional influent pump to meet the projected firm capacity.
- Additional mechanical bar screen with a 23.5 mgd capacity.
- Odor containment at the influent pump station and mechanical bar screen area.
- For peak overflows greater than 13.5 mgd, the existing gravity thickener would be converted into a ballasted sedimentation tank.
- Converting the circular primary clarifier to a combination primary clarifier/gravity thickener.
- The existing primary and secondary clarifiers rehabilitated for enhanced performance.
- Outfall diffuser in the Rogue River.
- Miscellaneous improvements include: Laboratory upgrades, operations building and modifications, instrumentation and control system expansion, plant equipment audit, additional plant landscaping, public education program, and yard piping upgrades.

10.30.5.1.5 Liquid Stream Alternatives Cost Estimate Comparison. A preliminary cost estimate for these liquid stream alternatives is summarized in Table 10.30.9. Engineering, administration, and contingency are included in these costs.

Table 10.30.9 Comparison of Liquid Stream Alternatives

Alternative	Capital Cost (millions)
One- Conventional Expansion	$12.23
Two- Ballasted Sedimentation	$13.59
Three- Zenon Process	$23.96

Source: City of Grants Pass Water Restoration Plan Update, June 2001, Parametrix Inc.

10.30.5.2 Biosolids Disposal and Handling Alternatives

The following subsections generally describe the alternatives for biosolids disposal and handling proposed at the Merlin Landfill co-compost facility, for hauling dewatered biosolids to the Dry Creek Landfill, for applying to land the Class B biosolids, and for using an Aerobic Thermophilic Pretreatment (ATP) component.

10.30.5.2.1 Alternative One– Merlin Landfill Co-compost Facility. Alternative One would co-compost digested primary and raw secondary biosolids. The existing digester would be rehabilitated and used only for digesting primary biosolids. A new component for dewatering digested primary biosolids to 15 percent solids would be installed in an existing building located on site. The biosolids would be hauled to the co-composting facility located at the Merlin Landfill. This facility would produce Class A biosolids, which would be available for public purchase.

10.30.5.2.2 Alternative Two- Dry Creek Landfill. Alternative Two hauls raw primary and secondary dewatered biosolids to the Dry Creek Landfill. These biosolids would be dewatered at the Grants Pass WRP to 15 percent solids using a belt filter press (BFP). The existing press and one new additional BFP would be installed in an existing building located on site. Extra hauling equipment would be required for handling the transport of all the biosolids to the landfill.

10.30.5.2.3 Alternative Three- Land Applying Class B Biosolids. Alternative Three is the recommended alternative from the WFP. It is a continuation of the current biosolids management program of long hauling Class B biosolids to be land applied. However, because of an increase in future biosolids production, contracting with landowners of large parcels of land in

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Eastern Oregon to expand the land application area would be necessary. Biosolids would be dewatered to 15 percent, hauled to the site in large tractor/trailers, and applied with a manure spreader. During the winter, the biosolids would be stored near the application site. An additional gravity thickener, gravity belt, anaerobic digester, and belt filter press are necessary to meet the future solids production. The existing anaerobic digester would need to be rehabilitated. Class A biosolids can be produced by adding low cost aeration equipment to the future storage building to create a pilot-scale co-composting facility.

10.30.5.2.4 Alternative Four- Aerobic Thermophilic Pretreatment (ATP). Alternative Four is to produce Class A biosolids and distribute them to the public as fertilizer. To accommodate increasing loads in the future, an ATP would be installed instead of adding a new digester. To reduce plastics in the biosolids, a Muffin Monster would be added prior to the ATP. The existing digester would need to be rehabilitated to enhance performance, increase capacity, and repair deficiencies.

10.30.5.2.5 Biosolids Alternatives Cost Estimate Comparison. Preliminary cost estimates for the four biosolids disposal alternatives are in Table 10.30.10. These values include engineering, administration, and contingency costs.

Table 10.30.10 Comparison of Biosolids Disposal and Handling Alternatives

Alternative	Capital Cost (millions)
One – Merlin Landfill Co-compost Facility	$3.60
Two – Dry Creek Landfill	$1.40
Three – Land Apply Class B Biosolids	$11.00
Four – Aerobic Thermophilic Pretreatment (ATP)	$2.30

Source: City of Grants Pass Water Restoration Plan Update, June 2001, Parametrix Inc.

10.30.5.3 Miscellaneous Plant Improvements

- Plant Equipment Audit. A full analysis of the existing component conditions would be conducted. This audit would analyze the remaining life span of each component and develop an operations and maintenance schedule for the 20-year planning period.
- Additional Plant Landscaping. Currently a substantial amount of landscaping at the treatment plant has occurred to promote a good-neighbor environment. However, to continue this effort, additional landscaping would be necessary.

10.30.6 PREFERRED TREATMENT ALTERNATIVES

10.30.6.1 Biosolids Handling and Disposal

For the biosolids handling and disposal, the *2001 Wastewater Facilities Plan Update* found the Merlin Landfill Co-compost Facility (Alternative One) to be the preferred alternative. This alternative would consist of the existing digester, a new dewatering device, and a co-composting facility. Under this solid waste handling system, rehabilitated digesters would be used to treat only primary clarifier solids. The secondary clarifier solids would be combined with the digested primary solids, dewatered in a new dewatering component, and trucked to the new co-composting facility located at the Merlin Landfill.

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10.30.6.2 Liquid Stream Treatment

The *2001 Wastewater Facilities Plan Update* found the ballasted sedimentation alternative to be the preferred alternative for the liquid stream treatment. Table 10.30.11 lists the anticipated costs and time-line of when the components or upgrades would occur at the Water Restoration Plant, and a proposed construction schedule follows. Both are excerpted from the *2001 Wastewater Facilities Plan Update*.

10.30.6.3 Capital Improvements Water Restoration Plant

Table 10.30.11 City of Grants Pass
Water Restoration Plant Capital Improvement Plan

Project Item	Probable Cost	Construction Schedule		
		2001-2004	2005-2006	2010-2011
PRELIMINARY AND PRIMARY TREATMENT				
Influent Pumping*	$560,000		$560,000	
Raw Sewage Pipeline to Ballasted Sedimentation	$240,000		$240,000	
Screening Odor Control*	$390,000	$390,000		
Mechanical Bar Screen No. 2*	$230,000		$230,000	
Modify Gravity Thickener to Ballasted Sedimentation	$3,510,000		$3,510,000	
Modify Existing Primary to Combination Clarifier/Thickener	$610,000		$610,000	
Yard Piping*	$270,000		$270,000	
SECONDARY TREATMENT				
Aeration Basin Fine Bubble*	$530,000	$530,000		
Aeration Basin Selector*	$260,000	$260,00		
Blowers and DO Control*	$870,000	$870,000		
Rehabilitate Existing Clarifiers*	$770,000	$770,000		
New Secondary Clarifiers*	$2,400,000	$1,200,000		$1,200,000
Yard Piping*	$770,000	$400,000		$370,000
Motorized Gates*	$340,000	$340,00		
FINAL TREATMENT				
Outfall Diffuser*	$540,000	$540,000		
OTHER PLANT IMPROVEMENTS				
Lab Improvements*	$100,000	$100,000		
Operation Building Repairs and Office*	$130,000	$130,000		
SCADA System Expansion*	$670,000	$250,000	$250,000	$170,000
Equipment Improvements from Audit Results*	$250,000	$100,000	$75,000	$75,000
Plant Landscaping*	$100,000	$40,000	$30,000	$30,000

Public Education*	$50,000	$20,000	$20,000	$10,000
SOLIDS THICKENING AND DIGESTION				
Rehabilitate Existing Digester*	$740,000	$740,000		
Dewatering Centrifuge*	$1,000,000	$1,000,000		
SOLIDS HANDLING OFF-SITE				
Co-composting Facility*	$1,850,000	$1,850,000		
COLLECTION SYSTEM				
Pine Street*	$1,050,000	$1,050,000		
2nd Street	$700,000	$700,000		
Western Avenue	$580,000	$580,000		
Master Plan*	$170,000	$170,000		
TOTALS	$19,680,000	$12,030,000	$5,795,000	$1,855,000

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc. *(1999-2000 dollars)
*Item has been completed or partially completed as of April 2008 (Per Public Works Dept.)

Year 2000

- Install odor containment and control at the influent pump station and mechanical screening areas of the plant.
- Add an anoxic selector basin to the aeration basin.
- Modify the existing aeration basin, convert the existing aeration system to fine bubble diffusers, and add dissolved oxygen control and motorized gates. This would require new or modified aeration blowers.
- Rehabilitate the existing secondary clarifiers to correct short circuiting and flow distribution problems.
- Add a third secondary clarifier.
- Begin laboratory upgrades and improvements.
- Begin operations building repairs and modifications.
- Begin instrumentation and control system expansion.
- Conduct a plant equipment audit.
- Continue to install plant landscaping.
- Develop and institute a public education program.

Year 2005

- Install additional influent pumping capacity.
- Add a second mechanical bar screen.
- Convert the existing circular primary clarifier to a combination primary clarifier/gravity thickener.
- Modify the existing gravity thickener to a ballasted sedimentation tank to treat peak flow.

Year 2010

- Add a fourth secondary clarifier.

ODC114

10.30.7 RECOMMENDED COLLECTION SYSTEM IMPROVEMENTS

The recommended collection system improvements presented below are based upon deficiencies in the pipeline hydraulic capacity that were identified in the Hydraulic Analysis found in Section 6 of the *2004 Collection System Master Plan* (Parametrix) and the needed collection system improvements that were identified in the Maintenance and Reliability Analysis presented in Section 7. A Capital Improvement Program (CIP) for the collection system was developed based on a priority analysis of these improvements.

10.30.7.1 Collection System Goals

Three goals were used in developing a CIP for the collection system, identifying the improvements required and a schedule for implementation.

- Service to Saturation-Level Populations: All of the needed improvements were selected to serve the 2060 populations that could occur in the collection system service area.
- Attention to Critical Improvements: Attention was given to collection system pipelines and sub-basin service areas that City staff identified as problems areas. City staff experience in the frequency of maintenance of various pipelines and witnessing surcharged pipelines (hydraulic capacity deficiencies) have been used, particularly to schedule needed improvements.
- Distribution of Capital Expenditures: In the selection and scheduling of the required collection system improvements, the goal was to develop a CIP that is financially viable for the City of Grants Pass.

10.30.7.2 Hydraulic Capacity Improvements

Based on the hydraulic analysis conducted on the wastewater collection system serving the City of Grants Pass, five capital improvement projects have been identified that need to be completed in the next 20 years. These improvements are necessary to maintain adequate conveyance system capacity in the collection system and prevent sewer system overflows. These five projects are found in Section 8 of the *2004 Collection System Master Plan.*

10.30.7.3 Maintenance and Reliability Improvements

Very old and small pipelines serve the downtown area of Grants Pass. The pipelines are greater than 60-years-old, and often only 6-inches in diameter. Based on the Maintenance and Reliability Analysis conducted on the wastewater collection system, six areas of the City are served by these old small-diameter collection lines need further investigation and will likely require repair or replacement. These areas are described in Section 8 of the *2004 Collection System Master Plan.*

10.30.7.4 Estimated Cost of Improvements

By using the gravity sewer pipeline construction cost unit prices, total project costs for each of the recommended pipeline improvement projects are listed in the following table.

Table 10.30.12 Cost of Recommended Pipeline Improvement Projects (in $1M)

Project	Length (feet)	Preliminary Diameter (inches)	Construction Cost ($/foot)	Base Construction Cost	Construction Contingency 40%	Engineering Legal and Administration 30%	Total Estimated Project Cost
Pine Street*	7,010	24	$200	$1.40	$0.56	$0.59	$2.55
Western Avenue	4,720	18	$170	$0.80	$0.32	$0.34	$1.46
Mill Street	9,140	21	$185	$1.69	$0.68	$0.71	$3.08
7th Avenue	4,530	18	$170	$0.77	$0.31	$0.32	$1.40
Nebraska	2,710	18	$170	$0.46	$0.18	$0.17	$0.81

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004
*Item has been completed or partially completed as of April 2008 (Per Joey Wright, Public Works Dept.)

Similarly, the total project costs for the recommended structural repair areas have been prepared. A minimum diameter of 8-inch sewer pipeline has been assumed to estimate project cost. This is because replacing existing 6-inch-diameter sewer pipe with 6-inch does not meet generally accepted sewer design criteria. Given that different repair/replacement technologies will be implemented in these areas, rather than just assume replacement of all old pipelines with new pipelines to estimate the total project costs in each of these areas, it was assumed that total cost would equal the cost to replace one-half of all pipelines in these areas. These costs are developed and presented in Table 10.30.13 below.

Table 10.30.13 Cost of Recommended Structural Repair Areas (in $1M)

Structural Repair Area	Pipeline Diameter (inches)	Length (feet)	Construction Cost ($/foot)	Base Construction Cost	Construction Contingency 40%	Engineering (Legal and Administration) 30%	Total Estimated Cost*
Pine Street (*completed*)	8 10 12	7,667 402 48	$150 $150 $155	$1.22	$0.49	$0.51	$1.11
5th Street	8 10 12	11,672 1,244 2,606	$150 $150 $155	$2.34	$0.94	$0.98	$2.13
7th Street	8 10 12 18	9,326 400 692 260	$150 $150 $155 $170	$1.61	$0.64	$0.67	$1.46
Subbasin B/C	8 10 12	7,409 945 644	$150 $150 $155	$1.35	$0.54	$0.57	$1.23
Subbasin H	8 10	7,919 1,005	$150 $150	$1.34	$0.53	$0.56	$1.21
Lawnridge-Washington	8 10 12	11,362 612 16	$150 $150 $155	$1.80	$0.72	$0.76	$1.64

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004
*Total of base cost, construction contingency, and engineering divided by two

10.30.7.5 Collection System Capital Improvement Program

Using the prioritization of Collection System Improvements described in Section 8.4 of the *2004 Collection System Master Plan* (Parametrix) and the estimated cost of these improvements presented in Table 10.30.12 and Table 10.30.13, a recommended Capital Improvement Program for the City collection system has been developed and is presented in Table 10.30.14.

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Table 10.30.14 Recommended Capital Improvement Program
Grants Pass Collection System

Project	Schedule	Cost ($1M)
Pine Street Sewer*	2004-2006	$2.55
Western Avenue Sewer	2006-2009	$1.46
Pine Street Structural Repair*	2009-2011	$1.46
5th Street Structural Repair	2010-2012	$1.11
7th Street Structural Repair	2013-2015	$1.46
Lawnridge-Washington Structural Repair		$1.64
Mill Street Sewer	2016-2018	$3.08
Sub-basin Structural Repair		$1.23
Nebraska Avenue Sewer	2022-2024	$0.81
Total Collection System Capital Improvement Program:		**$18.08**

Source: City of Grants Pass Collection System Master Plan, Parametrix Inc., 2004
*Item has been completed or partially completed as of April 2008 (Per Joey Wright, Public Works Dept.)

10.30.8 REGULATORY AND PROCEDURAL ISSUES

Federal, state, and local regulatory agency policies and procedures affect the installation, upgrades, and operation of the City's wastewater collection system and treatment facility. The impacts of these policies and procedures on wastewater management planning in the Grants Pass area are described below. The discussion of regulations presented is not exhaustive and is focused on those regulations and laws that are relevant to wastewater conveyance and treatment.

10.30.8.1 FEDERAL POLICY

Federal policies that will affect the planning process include the Federal Water Pollution Control Act/Clean Water Act; Safe Drinking Water Act; and the proposed Capacity, Management, Operation and Maintenance Rules.

10.30.8.1.1 Federal Water Pollution Control Act/Clean Water Act

Since its enactment, the Federal Water Pollution Control Act, also known as the Clean Water Act (CWA), has formed the foundation for regulations detailing specific requirements for pollution prevention and response measures. The CWA requires states to adopt water quality standards consistent with federal limitations on pollutant and thermal loading. The standards are to take into consideration the use of the waters for public water supplies; propagation of fish and wildlife; recreational purposes; and agricultural, industrial, and other beneficial uses.

The City's collection system conveys wastewater to the City WRP, where it is treated before discharge to the Rogue River. State policies specifically regulate pollutant and thermal loading to comply with federal policy detailed in the CWA.

10.30.8.1.2 Safe Drinking Water Act

The Safe Drinking Water Act (SDWA) authorizes the Environmental Protection Agency (EPA) to set national health-based standards for drinking water to protect against contaminants that may be found in drinking water. Wastewater flows that are collected in or travel through substandard collection systems have the potential to contaminate drinking water systems. State and local

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regulations are designed to comply with the SDWA, and prohibit activities that could cause an adverse impact on existing or potential beneficial use of groundwater.

10.30.8.1.3 Proposed Capacity, Management, Operation, and Maintenance Rule

EPA is proposing rules that will govern the manner in which municipalities and special service districts manage and operate wastewater collection systems. The proposed Capacity, Management, Operation, and Maintenance (CMOM) Rule, depending on its final promulgated form, may have a significant affect on collection system development and operation and maintenance (O&M) for the City.

Under the proposed rule, sanitary sewer overflows (SSOs) would be prohibited unless caused by severe natural conditions such as widespread flooding, earthquakes, or other natural disasters. Owners of collection systems would be required to provide adequate capacity for peak flows in all parts of the system, monitor and report on SSOs, and make the SSO control program and reports available for public review.

There are two aspects of the proposed rule that are under close scrutiny of the reviewing community. First, satellite sewer systems would be operated under separate NPDES permits. Satellite sewer systems are loosely defined in the proposed CMOM rule as any agency that conveys wastewater to another agency for additional conveyance and final treatment and discharge. For Grants Pass, the RSSSD would be considered a satellite system requiring its own NPDES permit.

Second, the proposed CMOM rule is vague on the design threshold to which SSOs must be controlled. For instance, the proposed rule is silent on the recurrence interval of a storm event above which SSOs would be allowed (e.g., would SSOs be permitted during storms greater than 1-in-5 year event?). As such, EPA offers wastewater agencies little or no clear guidance regarding the amount of additional pipeline and pump station construction that would be required under CMOM, nor understanding about the amount of additional maintenance effort required to ensure elimination of SSOs.

As of February 2008, the CMOM rule had not yet been implemented. Based on an April 2004 statement on the EPA SSO web page, "SSO Proposed Rule was with-drawn from publication in the Federal Register," the timeline for implementation of the CMOM rule is uncertain. How the CMOM rule will eventually be interpreted and applied in Oregon is also uncertain. One possibility is that Oregon's "bacteria rule," will be used to set a minimum threshold for SSO prevention.

10.30.8.2 STATE POLICY

10.30.8.2.1 National Pollutant Discharge Elimination System

Section 402 of the CWA provides the legal basis for the NPDES permit program, which regulates point and non-point source discharges. Oregon Department of Environmental Qualtiy (ODEQ) is authorized by EPA to administer the NPDES program through Oregon Revised Statute 468B and associated OARs. These rules and statutes include regulations for wastewater collection, treatment, control, and disposal. Under the conditions of the NPDES permit, permittees are allowed to construct, install, modify, or operate these systems only in conformance with the Federal Clean Water Act and the above-mentioned State statues that set forth requirements, limitations, and conditions for such activities.

The Grants Pass WRP plant operates under NPDES Permit Number 101985 issued December 29, 2000.

10.30.8.2.2 Bacterial Control Management Plan

As noted previously, EPA is currently considering proposed CMOM rules that will limit the number of allowable SSOs. While the proposed CMOM rule is silent on collection system design criteria, Oregon has already adopted a rule that addresses the "bacteria rule," therefore, indirectly offers some guidance to design engineers and collection system owners. The OAR seeks to protect receiving waters and drinking water sources by prohibiting discharge of untreated wastewater to waters of the state except during the following conditions:

- During the period November 1 though May 21, except during a storm event greater than the 1-in-5-year, 24-hour duration storm.
- During the period of May 22 through October 31, except during a storm event greater than the 1-in-10-year, 24-hour duration storm.

The State Environmental Quality Commission may approve a change to these rules on a case-by-case basis as described in OAR 340-41-120. Determining the causes and preventing against SSOs usually requires a municipality to thoroughly evaluate both the collection and treatment system to determine the extent of extraneous weather-related flow, system structural condition and reliability, system hydraulic and treatment capacity, and the efficiency of operation and maintenance practices. The City has already performed a thorough analysis of the treatment and collection system.

10.30.8.2.3 Groundwater Regulation

The Federal SDWA requires that state underground injection control programs be established to ensure that underground injection will not endanger drinking water sources. In Oregon, groundwater regulations, including regulatory requirements for injection controls, are administered by ODEQ. On-site drain fields and septic systems that serve 20 or more persons are considered injection wells by ODEQ. The City Development Code requires that all new development within the service area be connected to the wastewater collection and treatment system. Existing development using septic systems is required to connect to the public sewer system at such time as repair or replacement of existing facilities is necessary, if public sewer is within 300 feet of the property.

10.30.8.3 LOCAL ORDINANCES, POLICY, AND MANAGEMENT AGREEMENT

Local requirements of particular concern to the planning process are related to the City Municipal Code, City Development Code, and the Sanitary Sewer Lateral Replacement Policy.

10.30.8.3.1 City of Grants Pass Municipal Code

City Ordinances 4861 and 5028 have been adopted by the City as Chapter 8.50 of the City Municipal Code. Chapter 8.50 is intended to protect public health and safety; protect the environment; and ensure compliance with all applicable state and federal laws as they pertain to wastewater collection, conveyance, treatment, and discharge.

Sewer use requirements set forth in Chapter 8.50 include general and specific prohibited discharge standards. The general prohibitions state that no user shall introduce or cause to be introduced into the City WRP any pollutant or wastewater which causes pass-through or interference, or which will cause the WRP to violate its NPDES permit or harmfully impact the receiving water quality standards. Chapter 8.50 also sets forth procedures for the allowance of intentional occurrences, and the reporting of unanticipated bypass.

10.30.8.3.2 City of Grants Pass Development Code

Title 10 of the City Municipal Code may also be cited as the City Development Code. The purpose of the Development Code is to coordinate City regulations governing the development and use of land. Standards for sewer and septic systems are set forth in the Development Code to ensure compliance with state and federal statutes, policies, and laws designed to prevent harmful impact to receiving waters.

10.30.8.3.3 Sanitary Sewer Lateral Replacement Policy

Substandard or combined sewer laterals discovered during public sewer, water, or storm drain projects are required to be replaced. The City considers the sewer lateral to be the responsibility of the private property owner from the point of connection to the main to the building being served. Replacement of substandard sewer laterals may often include work within the public right-of-way, with the possibility of additional costs such as pavement patching, traffic control, and other construction items not usually associated with work within private property boundaries. To assist the property owner in the cost of lateral replacement, the City has adopted a Sanitary Sewer Lateral Replacement Policy. Under this policy, the City Manager can authorize payment of 50 percent of the cost of replacing failed or otherwise substandard laterals.

10.30.8.3.4 Urban Growth Boundary Management Agreement

The City-County Urban Service Policies, adopted with the UGB in August 1979, require a public sewer system with capacities to serve urban levels of development. On August 8, 1998, Josephine County, City of Grants Pass, Harbeck-Fruitdale Sewer District and Redwood Sanitary Sewer Service District signed an Intergovernmental Agreement for the Orderly Management of the Grants Pass Urban Growth Boundary Area.

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10.30.9 FINANCING PLAN

Various funding alternatives exist for the City of Grants Pass to implement the Capital Improvement Plan (CIP). The purpose of this section is to determine the best option for financing. Unless otherwise noted, the information from this section was excerpted from the *2001 Wastewater Facilities Plan Update*.

As part of the work in developing a financing plan, a computer model was created to assist the City in future modifications to both the CIP and operation/maintenance funding. This model is the basis for the conclusions presented herein.

10.30.9.1 Capital Costs

The CIP includes three major elements: Treatment Plant (liquid treatment), Biosolids, and Collection System. The proposed phasing plan is shown in Table 10.30.15.

Table 10.30.15 Capital Improvement Plan (Year 2000 dollars)

	YEAR 2001-2004	Year 2005-2006	Year 2010-2011
Treatment Plant	$5,940,000	$5,795,000	$1,855,000
Biosolids	$3,590,000		
Collection System	$2,500,000		
TOTAL	$12,030,000	$5,795,000	$1,855,000

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc.

In addition to capital costs, the City must be able to fund the ongoing cost of operating and adequately maintaining its sewer utility. A breakdown of these operation and maintenance costs is shown in Table 10.30.16.

Table 10.30.16 Operation and Maintenance Expenses (Year 2000 dollars)

Wastewater Collection Services	$333,488
Wastewater Treatment Services	$887,513
Customer Services	$167,910
General Program Operations	$361,393
TOTAL	$1,750,304

Source: City of Grants Pass Water Restoration Plan, June 2001, Parametrix Inc.
NOTE: Updated operation and maintenance expenses can be obtained from annual city budgets.

10.30.9.2 Current Funding

The City of Grants Pass has two main revenue sources: monthly user charges and system development charges. The monthly user charge is collected for all system customers, and has increased since the *2001 Wastewater Facilities Plan Update*.

All new connections to the system pay a Sewer System Development Charge (SSDC). This charge is designed so new customers pay their share of wastewater collection and treatment infrastructure costs. At the time of the *2001 Wastewater Facilities Plan Update*, SSDCs averaged about $1,000 per new connection. SSDCs have increased substantially since that time,

and vary depending upon use and location (properties within the Redwood Sanitary Sewer District are subject to a different SDC schedule than other properties connecting to the system.)

10.30.9.3 Capital Funding Mechanisms

To fund the CIP, the City will need to consider other funding mechanisms. These other funding mechanisms include: Revenue Bonds; Low interest loans – State Revolving Fund (SRF); and, Grants.

A revenue bond is a very common tool to fund capital improvements. Rates are determined by market conditions and currently are around 5 to 6 percent. Due to the significant financing cost and associated coverage, this funding method is normally a last resort if other funds are no available.

Grants used to be the only way to finance wastewater treatment improvements. Many of the upgrades to secondary treatment across the United States were funded by grants, sometimes up to 75 percent of the costs. Wastewater construction grants, while still available, are insufficient compared with the current demand.

One of the best governmental programs available is the State Revolving Fund loan program. This fund, seeded by money from the Environmental Protection Agency (EPA), is giving state governments, including Oregon, funds to loan to municipalities for treatment and collection system improvements. These loans, administered by the Oregon Department of Environmental Quality Wastewater Finance Office, are "low interest" with current rates at less than 4 percent. Unlike bonds, normally these loans have a smaller reserve amount and no coverage requirements. Due to federal funding, cities like Grants Pass seeking SRF funding must comply with EPA requirements for a Facility Plan, thus this plan has been developed according to said requirements. The City secured approximately $7M in SRF loans in 2003-2004 for work on Phase I of the CIP. The work was completed as of July 2004, and the loans were in the process of being repaid as of January 2008.[4]

10.30.9.4 Projected Cash Flow

The major components of both revenue and expenses in a simplified view of the wastewater utility are:

- Revenue: Monthly Sewer Service Charges, SDCs, Loan Proceeds, and Interest
- Expenses: Operation and Maintenance, Capital Expenditures, Debt Service, and Reserve Amounts

In reality, these revenues and expenses are tracked in separate accounts; however, for the purpose of this simplified analysis, all revenue and expenses will be considered as one amount. Table 10.30.17 projects the annual cash flow for the Grants Pass wastewater utility. This projection is for the period of the CIP. With the funding from the SRF, and the current cash on hand (based on the City's construction fund only), it appears that the City will be able to fully fund the CIP. In addition, the City may be able to fund an ongoing sewer rehabilitation program. This amount, shown in the cash flow during non-CIP funded years, is $250,000. It will have to be determined if this amount is sufficient to address the long-term needs of the City.

[4] Per Joey Wright, City of Grants Pass Public Works Department, January 2008

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Table 10.30.17 Simplified Cash Flow Analysis

	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009	2010	2011
Beginning Balance	$6,800,000	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	1,401,552$	$931,076
Rate Revenue	$2,650,000	$2,689,750	$2,730,096	$2,771,048	$2,812,613	$2,854,803	$2,897,625	$2,941,089	$2,985,205	$3,029,983	$3,075,433	$3,121,565
SDC Revenue		$100,000	$101,500	$103,023	$104,568	$106,136	$107,728	$109,344	$110,984	$112,649	$114,339	$116,054
O&M Cost	$1,750,000	$1,802,500	$1,856,575	$1,912,272	$1,969,640	$2,028,730	$2,089,592	$2,152,279	$2,216,848	$2,283,353	$2,351,854	$2,422,409
Loan Funding		$2,000,000	$2,000,000		$2,000,000							
Debt Service		$145,435	$290,871	$290,871	$290,871	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306	$436,306
Reserve Account		$150,000	$150,000			$150,000						
Capital Cost		$6,195,400	$6,195,400	$100,000[a]	$100,000[a]	$2,547,000	$2,547,000	$100,000[a]	$100,000[a]	$100,000[a]	$921,000	$921,000
Investment Income		$290,000	$243,728	$111,106	$56,396	$82,298	$92,184	$50,301	$19,036	$34,521	$48,912	$46,653
Ending Balance	$7,700,000	$4,486,415	$1,068,894	$1,750,927	$2,363,994	$2,245,196	$269,835	$681,984	$1,044,057	$1,401,552	$931,076	$435,633

Source: 2001 Grants Pass Wastewater Facilities Plan Update, Parametrix

[a] Balances reflect both Capital and Operations Accounts

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10.30.9.5 Conclusion

The financial situation of the City's wastewater utility is good. Given the ability to secure loans from the SRF program, and the sound fiscal management of the utility, the City will be able to fully fund the CIP as outlined in the Facilities Plan.

10.30.10 SANITARY SEWER SERVICES FINDINGS

10.30.10.1 Existing Sewer Capacity

- At the time of the *2001 Wastewater Facilities Plan Update*, the Grants Pass Water Restoration Plant (WRP) treated 4.5 mgd of average dry weather flow (ADWF) to a record peak storm flow of 26.5mgd. More recent data has also been compiled. As of April 2008, the WRP treated an ADWF of 5.5 mgd and peak wet weather flow of 30.0 mgd. The plant is comprised of numerous unit treatment processes for both liquid and solid streams, a control/laboratory building, and a maintenance shop. The Grants Pass WRP has a 27 mgd hydraulic capacity for influent pumping[5], screening, and primary treatment. There is a 13 mgd hydraulic capacity for secondary treatment. In addition, there is a 43 mgd hydraulic capacity for UV disinfection. Flow exceeding the secondary treatment capacity receives only primary treatment and disinfection. This occurs only a few days a year during wet weather storm conditions. For the final phase of disposal, dewatered biosolids are loaded into a dumpster and hauled to the Merlin Landfill for the JO-GRO composting process.

- The Harbeck-Fruitdale Sewer Service District uses the Grants Pass WRP plant for the processing of its waste. The sewer collection system has a capacity that can accommodate the population equivalent of approximately 14,000 persons.

- The Redwood Sanitary Sewer Service District uses the Grants Pass WRP for the processing of its waste. The sewer collection system has a capacity that can accommodate the population equivalent of approximately 16,000 persons.

10.30.10.2 Future Need

- The City and County have already established pumping of Redwood's wastewater to Grants Pass WRP and are not likely to extend services to the North Valley during this planning period.

- The *2004 Collection System Master Plan* developed a future equivalent population projection for the service area by applying sub-area growth rates from the City's Comprehensive Plan to the base population estimates for the sub-areas and adding 35% to the resulting population for commercial / industrial population equivalent. As of 2003, the Grants Pass WRP was serving an estimated population equivalent of 44,250. Estimated growth rates are

[5] As of January 2008 a project was completed to increase the hydraulic capacity for influent pumping to 45 mgd.

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respectively: 1.5% for Grants Pass, 1.6% for Harbeck-Fruitdale and 3.1% for Redwood. By the year 2020, it was anticipated that the Grants Pass WRP could be serving a total population of 60,157. Year 2060 service area population, assumed by the *2004 Collection System Master Plan* to represent build-out, was estimated at approximately 130,167.

- The *2001 Wastewater Facilities Plan Update* identified a need for several improvements: At the head works, an additional mechanical bar screen to be installed with a 23.5 mgd capacity this provides redundancy and a maximum capacity of 47 mgd, which is well beyond the projected future flows); An outfall diffuser to be added to improve and reduce the ammonia toxicity into the Rogue River; Several miscellaneous plant improvements, which existed in the Facility Plan, were also included in the value engineering alternatives. These improvements include laboratory upgrades, operations building repairs and modifications, and instrumentation and control system expansion. All of these improvements have since been completed.

- Plant Equipment Audit. The *2001 Wastewater Facilities Plan* recommended that a full analysis of the existing component conditions be conducted. This audit would analyze the remaining life span of each component and develop an operations and maintenance schedule for the 20-year planning period. The audit has since been completed.

- Additional Plant Landscaping. As of the *2001 Wastewater Facilities Plan*, a substantial amount of landscaping at the treatment plant had occurred to promote a good-neighbor environment. However, to continue this effort, the *WFP* recognized that additional landscaping would be necessary. The installation of additional landscaping has been ongoing.

- The City of Grants Pass continues upgrading its sanitary sewer system facilities to provide the appropriate level of services for the impending growth and development of its service area. Through its master facility planning, the City is prepared to meet its growth challenges in compliance with federal, state and local regulations, provided it continues to implement the respective Capital Improvements Plans as scheduled.

- Feasibility for Effluent Reuse, (Chapter 7 – *Grants Pass Facilities Plan*, Parametrix, 1999) concluded that reclaimed water (treated effluent) from the Grants Pass Water Restoration Plant (WRP) could be used beneficially instead of being discharged. Low river levels coinciding with the period of peak crop water use make irrigation a potentially feasible alternative for the City of Grants Pass and local agricultural water users. Reclaimed water can irrigate agricultural land, parks, highway landscaping, and golf courses. It can be used to grow wood fiber for fuel, pulp, or lumber. Other potential uses include increased flow to wetlands and storage in reservoirs or other impoundments specifically designated for effluent reuse only.

    - An extra incentive to "get out of the river" during the summer months is provided by concerns over meeting temperature requirements, potential future nutrient limitations, and listing of certain fish species as endangered. Reclaimed WRP

effluent can also be viewed as a valuable resource that could help bridge the gap created by the planned removal of the Savage Rapids Dam. Dam removal has sparked a controversy about the availability of irrigation water, and new costs associated with pumping irrigation water from the Rogue River.

- Effluent reuse is complex and requires further planning to address implementation issues of effluent reservoir and storage siting, piping route selection, right-of-way acquisition, system ownership, financing and management.

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10. PUBLIC FACILITIES & SERVICES

Goal

To provide needed facilities and services for the Urban Growth Boundary area in a timely, orderly, efficient, economic and coordinated manner.

Policies

10.1 General Service Policies

10.1.1 Urban levels of development shall require urban levels of service, as defined by the Implementing Ordinances.

10.1.2 Those who benefit most from the extension of urban services shall be those who pay most of the cost of service extension. Citizens in the developed areas with a full range of services already provided should pay little if any of the costs of extending urban services. Various techniques should be utilized to mitigate the economic impact of service extension to those residents in developing areas who already provide certain of their own services, and to mitigate the economic impact of service extension to those persons on fixed and/or low incomes.

10.1.3 Services shall be provided in an orderly and economic manner. Services provided at public expense should be provided first to those areas most heavily committed to urban development and those areas most actively developing, before extension to less committed areas or to those areas less actively developing. The extension of services with similar physical and/or programmatic requirements should be coordinated where economies will result. The involvement of the private sector is essential in the provision of services, and will determine to a great extent the timing, location and financing means of service extensions.

10.1.4 The division of lands and development of property within the Urban Growth Boundary shall be in accordance with the phased provision of urban services, as provided in the Implementing Ordinances. The type, location and phasing of public facilities and services shall be used by the City and County in a coordinated fashion as factors to direct urban expansion, and to implement land use policies.

10.1.5 Neither the City nor the County shall create special districts within the Urban Growth Boundary for the provision of water, sewer, storm drainage or street improvement services, unless approved by both parties and managed by either the City Council or the Board of County Commissioners. Overlapping and competing layers of political control of the provision of services shall be discouraged.

10.1.6 Services shall be resource effective. Services shall not be extended past the carrying capacity of the resource base of that service, and shall utilize the resource in the most effective way practicable.

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10.1.7 The City and County recognize that the provision of necessary services to accommodate the projected growth and land use allocations is a mutual responsibility. The City and County will continue to cooperate with other and with the private sector in the development and use of financial mechanisms and programs that are effective, efficient and equitable. The County recognizes its need to develop new techniques and resources for financing urban level public facilities.

10.1.8 ***The City and County will develop, adopt and maintain*** ~~A six-year~~ Capital Improvements Program***s to meet the needs of the service area.*** ~~will be developed jointly by the City and County by January, 1984, and be maintained on an annual basis. thereafter. This~~ ***These*** program***s*** will be used as a guide in the decision making process regarding the expenditures of local public funds on capital projects as well as seeking State and Federal funds.

~~10.1.9 Prior to first periodic review, the City and County shall complete a public facilities plan for those areas within the Urban Growth Boundary which were not included within the adopted facilities plans at the time of acknowledgment.~~

10.2 Water Service Policies

10.2.1 The City and County shall follow the adopted Water Facilities Plan for the Urban Growth Boundary area when extending and improving water service. Key factors to be utilized in growth management include:

(a) the number, size, location and approximate costs of water treatment, storage and distribution facilities deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) water sources and treatment and distribution modes;

(c) continued input from all segments of the community;

(d) implementation and financing strategies for acquiring, developing and maintaining needed water treatment, storage and distribution; and

(e) determination of the areas of greatest need, including techniques of funding and prioritization for these areas of need.

10.2.2 The City and County shall maintain a continuously updated computerized model of the municipal distribution system. This model shall be available for use at cost by public agencies and private organizations in order to determine questions of service capacity, improvement requirements and improvement cost.

10.2.3 The City and County shall adopt an official Water Facilities Plan Map, showing the location, size and type of existing and future water treatment, storage and distribution

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(a) the number, size, location and approximate costs of storm drainage facilities and improvements deemed necessary to serve the expected population within the Urban Growth Boundary;

(b) the analysis of the UGB drainage basins, using generally accepted runoff projection techniques, including appropriate computer modeling, if possible;

(c) implementation and financing strategies for acquiring, developing and maintaining needed storm drainage facilities;

(d) maintaining continuity with past drainage plans, as adopted and developed by the City and County; and

(e) determination of the areas of highest priority, including techniques of funding and prioritization for these high priority areas.

10.4.2 The City and County shall adopt an official Storm Drainage Facility Map showing the location, size and type of existing and future storm drainage facilities called for by the Storm Drainage Plan. The Storm Drainage Map shall be used to determine service district jurisdiction, and the location of future storm drainage facilities and improvements.

10.4.3 The Development Code shall act to facilitate these storm drainage policies, and shall contain a balanced mix of positive incentives (which may include density transfers, public funding of oversized lines, rapid review procedures, etc.), as well as exactive requirements, system charges, development requirements, etc.), as needed to assure the realization of these policies.

10.4.4 The City and County shall develop a Capital Improvement Program (CIP) within 12 months of adoption of the Comprehensive Plan, which program shall include timely and adequate funding to realize the development of facilities required by the adopted Storm Drainage Plan, and shown on the Storm Drainage Facilities Map.

10.4.5 The Storm Drain Plan shall be reviewed and updated, and revised if necessary, at one year intervals, with major revisions at five year intervals. The revisions to the Storm Drain Plan shall be used as a basis for revising these policies.

10.4.6 The City and County working with the Grants Pass Irrigation District shall explore an agreement that will ensure that the storm drainage use of, and the necessary repairs, improvements and maintenance of the irrigation canal system, are made in a manner consistent with the Storm Drain Plan, and in a timely and cost-effective manner.

10.4.7 The City and County shall encourage storm drainage design that minimizes storm water runoff, including retention areas or devised, use of vegetative open space, and the preservation of natural waterways.

10.4.8 The City and County shall coordinate the provision of storm drain facilities with the provision of open space called for by the Park Facilities Plan, wherever possible, and to the

extent practicable. This coordination shall include retaining drainage channels as close as possible to their natural state, and the use of plan materials and maintenance techniques in storm water retention.

10.4.9 Urban level development shall require urban levels of storm drainage, as provided in the Implementing Ordinances. Interim Development Standards shall allow development to proceed in a timely and economical manner, prior to full extension and development of the storm drain system, provided the requirements of public safety, health and welfare are met.

10.5 .Solid Waste Service Policies

10.5.1 The City and County shall encourage the collection of solid waste within the Boundary area by private, commercial collection services.

10.5.2 The City and County Agreements with the commercial franchise service managing the solid waste landfill at the Merlin site shall include measures to successfully reduce leachate produced at the landfill site, such as uphill trenching and draining, and importation of suitable topsoil to reduce erosion and promote revegetation.

10.5.3 Within 16 months of adoption of the Comprehensive Plan, the City and County shall adopt a Solid Waste Management Implementation Plan, including relevant sections of the Solid Waste Management Plan (1975), which plan shall include:

(a) an ongoing assessment of landfill disposal techniques, with provisions for correction of those techniques as required.

(b) a yearly estimate of landfill capacity and the rates of solid waste generation, including all areas within the landfill site service district as well as the UGB area, and an estimate of when landfill site capacity will be reached.

(c) a recommendation of financing strategies for adequately maintaining and preparing the landfill site, as well as providing for alternative methods of solid waste disposal.

10.6 .Police Protection Service Policies

10.6.1 Urban levels of development shall require urban levels of police protection. As the urbanizing area converts from rural to urban levels and intensities of land use over time, police protection should be increased to meet the increased service need.

10.6.2 The City and County shall explore an agreement establishing responsibility for the provision of police protection services within the Urban Growth Boundary over time. This agreement shall consider the costs and benefits of various methods of providing police protection, and shall include financing techniques to mitigate the costs of increased service.

10.7 .Fire Protection Service Policies

10.7.1 Municipal water systems shall provide water at fire flow capacities.

10.7.2 Urban levels of development shall require urban levels of fire protection as stipulated by the Implementing Ordinances. The minimum urban level of fire protection for fully developed residential, commercial and industrial areas shall be that qualifying for the insurance underwriters relative classification rating of 5. Provision of fire protection should be phased over time as urban level development proceed without a minimum of a Class 8 rating, nor shall commercial industrial development proceed without a minimum of a Class 9 rating.

10.7.3 The City and County shall explore an agreement establishing responsibility for the provisions of fire protection services within the Urban Growth Boundary area over time. This agreement shall consider the costs and benefits of various methods of providing fire protection, and shall include financing techniques to mitigate the costs of increased service.

10.8 .Health Services

10.8.1 Health services should be provided by the private sector. The City and County shall encourage the provision of health services in appropriate locations throughout the Boundary area.

10.9 .School Service Policies

10.9.1 The City and County shall maintain an open, ongoing dialogue with the School Districts in a manner that will facilitate the planning efforts of all agencies.

10.9.2 The City and County shall notify the respective School Districts of all residential land use actions within that district in a timely and complete manner, and make development data available to the districts on a regular basis.

10.9.3 The School Districts shall be notified in a timely manner regarding revisions and updates to the Comprehensive Plan that may affect the Districts, and shall be encourage to participate in the revision process.

Water Distribution System Master Plan (2001)

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CHAPTER 1

EXECUTIVE SUMMARY

This master plan presents the results of the water distribution system planning effort conducted for the City of Grants Pass. The plan summarizes the components of the existing water distribution system, analyzes local water demand patterns, evaluates the performance of the water system with respect to critical service standards, and identifies the improvements necessary to remedy system deficiencies and accommodate future growth. Based on this analysis, the study recommends specific projects for inclusion in the water distribution system Capital Improvement Plan (CIP). These projects will ensure that the water distribution system continues to provide adequate and reliable service to the Grants Pass community.

Existing System

The existing water distribution system consists of a water treatment plant, nine booster pumping stations, eight reservoirs, two pressure reducing valves, and five altitude valves. The service area contains eight pressure zones which are summarized in Table 1-1 along with their respective static pressure ranges.

Table 1-1. Pressure Zone Ranges

Zone	Elevation (feet)	Pressure (psi)
1	900 – 1,020	36 – 90
2	1,020 – 1,140	41 – 95
2A	960 – 1,035	61 – 94
2B	1,000 – 1060	35 – 60
3	1,140 – 1,280	36 – 100
4	1,280 – 1,420	42 – 104
5	1,420 – 1,560	41 – 104
NV	995 – 1,165	101 – 177

Water Demand

Analysis of historical water production and water demand data allowed for the identification of the water use patterns that characterize the City of Grants Pass. These water use patterns provided the basis for estimating future water demand in the community when development has filled the urban growth boundary (UGB). Table 1-2 summarizes existing water demand in terms of average annual, maximum month, maximum day, and peak hour demand and provides projections for the UGB build-out condition.

Table 1-2. Water Demand Summary for Grants Pass

	Current Water Demand, mgd	Future Total Water Demand, mgd
Average Annual	4.5	9
Maximum Month	8	16
Maximum Day	10	20
Peak Hour	20	40

Water Distribution System Service Standards

The City of Grants Pass maintains benchmarks for service quality that are used to measure performance of the water utility. These benchmarks include service standards for water quality, quantity, and pressure, as well as the minimum supply levels for fire protection. For example, the Grants Pass water distribution system was analyzed to ensure that service pressures never fall below 35 psi during normal demand scenarios and fire flows (of up to 4,000 gpm) are available without dropping system pressures below 20 psi. The service standards set forth in this master plan are derived from regulations, rules, and recommendations established by a variety of sources including the Oregon State Health Division (OSHD), the Environmental Protection Agency (EPA), the American Water Works Association (AWWA), the Insurance Services Office (ISO), and the Uniform Fire Code (UFC).

Distribution System Modeling

A computer based hydraulic model of the Grants Pass water distribution system was developed as part of the master planning effort to evaluate the ability of the system to meet current and projected demands. The City's existing model was expanded and updated to simulate both the existing and future water distribution system. Field calibration work confirmed that the model accurately simulates operation of the water distribution system. The model was used to evaluate the existing and future water distribution system under three conditions:

- Peak hour demand
- Maximum day demand plus fire flow
- Low demand during water treatment plant operation

System Evaluation and Capital Improvement Plan Recommendations

The Grants Pass water distribution system was analyzed to evaluate its performance and capacity under current and future demand conditions relative to critical service standards. This analysis identified system improvements necessary to maintain adequate performance through build-out of the UGB. These improvements were developed to either eliminate existing deficiencies in system performance or expand service to satisfy community growth. The elements of the water distribution system that were evaluated include water treatment plant capacity, treated water storage capacity, booster pumping capacity, and pipeline network performance.

Table 1-3 presents the specific costs for reservoir, pump station, and pipeline projects that are targeted for implementation through build-out of the UGB. These costs account for developer participation in the financing of some pipeline expansion projects. Costs shown are the City's estimated share of the pipeline extensions and do not include the cost which will need to be borne by developers. Figure 1-1 illustrates how the new pump stations and reservoirs will fit into the existing hydraulic profile of the water distribution system. Figure 1-2 illustrates the layout of the future system including new pipelines, pump stations, and reservoirs.

Timing of projects is based on the City's review of the expansion plans and reflects developer interest and submitted development plans. Some adjustment of timing and priorities should be expected.

Table 1-3. Estimated Capital Costs for CIP Project

Recommended Improvements	Capital Cost, $1,000
Period 2000 – 2005	
Pump Stations	805
Pipelines	7,109
Total	7,854
Period 2005 – 2010	
Pipelines	1,049
Pipeline Replacement	1,124
Total	2,173
Period 2010 - 2020	
Treated Water Storage	5,720
Pipelines	2,042
Pipeline Replacement	2,248
Total	10,010
Period Post 2020	
Treated Water Storage	1,560
Pump Stations	400
Pipelines	184
Total	2,144
Grand Total	22,181

CHAPTER 2

EXISTING WATER DISTRIBUTION SYSTEM

The Grants Pass water supply system currently distributes water to developed properties covering an area of more than 3,500 acres. The majority of the properties currently connected to the water distribution system are within the present city limits, although the City does provide service to some areas outside the city limits such as Harbeck-Fruitdale and the North Valley. The overall system is composed of a water treatment plant, eight booster pumping stations, eight reservoirs, two pressure reducing valves, and five altitude valves. Figure 2-1 illustrates the configuration of the Grants Pass water distribution system. The figure depicts all water distribution piping twelve inches in diameter and larger.

SOURCE OF SUPPLY

The source of supply for the City of Grants Pass is surface water from the Rogue River. The City draws water from the river with a pumping station located next to the water treatment plant. The treatment plant was originally constructed in 1930 and has undergone many renovations over the years. The most recent plant expansion was completed in 1983, bringing the total plant capacity to approximately 18 million gallons per day (mgd). Influent pumps deliver river water to the plant where the treatment process includes coagulation and sedimentation of suspended solids, filtration of the remaining particles, and chlorination for disinfection prior to pumping into the distribution system. The City has water rights for nearly 57 mgd.

DISTRIBUTION AND STORAGE SYSTEM

Pipeline Network

The Grants Pass water distribution pipeline network consists of approximately 130 miles of existing pipeline. Table 2-1 details the water distribution system according to pipeline length and diameter. These pipelines are made of cast iron or ductile iron and range in age up to approximately 80 years.

Service Pressures

The urban growth boundary for the City of Grants Pass encompasses lands of wide ranging elevations. As a result, the water distribution system service area contains eight separate service pressure zones. Table 2-2 summarizes the service elevations and static pressure range for each pressure zone. The lower end of the pressure range is based on reservoirs at 80 percent full and the upper end is based on full reservoirs. At this time, there are properties receiving City water service in each of the pressure zones except Zone 5.

According to the elevation ranges identified in Table 2-2, Figure 2-2 illustrates the extent of each pressure zone. In some areas, the pressure zone boundaries are modified slightly from these elevation ranges in order to accommodate special service pressure requirements. Pressure Zone 2A is a hybrid between Zones 1 and 2, as is the Rogue Community College's Zone 2B. The North Valley service area is actually a hybrid between Zones 1, 2, and 3, serving properties between the elevations of 995 feet and 1,165 feet. Due to the great range of elevations served in the North Valley, this pressure zone requires pressure reduction valves at service connections to maintain appropriate service pressures.

Table 2-1. Water Distribution System Pipeline Network

Pipe Size (inches)	Length (miles)
2	5.23
4	1.80
6	40.89
8	43.63
10	7.64
12	19.49
14	0.38
16	7.88
20	2.40
24	1.02
30	0.95
36	0.01
	131.32

Table 2-2. Pressure Zone Ranges

Zone	Elevation (feet)	Pressure (psi)
1	900 – 1,020	36 – 90
2	1,020 – 1,140	41 – 95
2A	960 – 1,035	61 – 94
2B	1,000 – 1,060	35 – 60[a]
3	1,140 – 1,280	36 – 100
4	1,280 – 1,420	42 – 104
5	1,420 – 1,560	41 – 104
NV	995 – 1,165	101 – 177

[a]This pressure range is approximate since construction details are not available for the RCC storage tanks.

Fire Protection

The Grants Pass Department of Public Safety provides fire protection for properties within the City. Since the water distribution system is an integral part of the City's fire protection system, the Department of Public Safety has adopted the Uniform Fire Code recommendations as the required fire flows for the various land use classifications within the City. These fire protection

requirements are discussed in detail in Chapter 4, "Water Distribution System Service Standards."

Booster Pumping Stations

The water distribution system includes the water treatment plant pumps and nine booster pumping stations that transfer water to the higher pressure zones. These pump stations either fill the reservoirs that serve these higher pressure zones or pump to maintain a minimum pressure in those areas that are not served by reservoirs. Table 2-3 details the technical information for each of the system's pumping stations.

Table 2-3. Existing Booster Pumping Stations

Pumping Station Name	Pressure Zone Served	Reservoirs Served	Number of Pumps	Pump Motor Size and Speed (hp/rpm)	Capacity of Each Pump (gpm)	Rated Discharge Head (feet)
Treatment Plant	1	No. 3	5	300/1,775	3,500	220
		No. 5		300/1,775	3,500	220
		No. 11		250/1,760	3,500	210
				250/1,760	3,500	210
				200/1,750	2,600	210
Lawnridge	2	No. 6	4	25/1,750	400	120
		No. 4		50/1,750	1,000	120
				50/1,750	1,000	120
				100/1,750	2,000	148
Madrone	2	No. 4	3	60/1,750	2,000	170
		No. 6		401,750	1,200	170
				301,750	900	170
Champion	3	No. 8	3	50/1,750	800	165
				150/1,750	2,300	165
				100/1,750	1,600	165
Starlite	3	--	4	15/3,500	60	185
				30/1,760	450	185
				60/1,760	1,050	185
				30/1,760	450	185
Hefley	4	No. 13	4	7.5/3,500	40	250
				15/3,500	120	250
				60/3,500	600	300
				60/3,500	600	300
North Valley	NV	No. 15	3	7.5/3,500	70	170
				30/3,500	500	174
				30/3,500	500	174
Hilltop	2	--	3	5/3,600	100	120
				7.5/3,600	150	120
				40/3,600	750	120
New Hope	2	--	4	30/3,600	350	212
				30/3,600	350	212
				30/3,600	350	212
				150/1,800	2,000	200
Harbeck Heights	2	--	3	5/3,600	90	100
				5/3,600	90	100
				50/3,600	1,200	125

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Reservoirs

There are eight treated water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999. Design information for these reservoirs is detailed in Table 2-4.

Table 2-4. Existing Reservoirs

Reservoir Location	Reservoir Number	Pressure Zone Served	Year Built	Construction Materials	Capacity (mg)	Bottom Elevation (ft)	Overflow Elevation (ft)
500 Block Woodson Dr.	3	1	1946	Concrete	3.5	1,089.5	1,108.5
1500 Block Ridge Rd.	4	2	1953	Concrete	0.75	1,216	1,240
1400 Block Sherman Ln.	5	1	1983	Concrete	3.5	1,079.5	1,108.5
2200 Block Crown St.	6	2	1982	Concrete	3.5	1,211	1,240
Heiglen Loop Rd.	8	3	1983	Concrete	2.0	1,341	1,370
1420 Denton Trail	11	1	1999	Concrete	4.5	1,080.1	1,108.5
1700 Block Sunset Ln.	13	4	1980	Concrete	0.08	1,510	1,520
3900 Block Highland Ave.	15	5	1985	Concrete	1.2	1,374	1,403

SYSTEM OPERATION

The general procedures for operation of the Grants Pass water distribution system are discussed in the following sections.

Grants Pass Water Treatment Plant

The water treatment plant operates as necessary to fill storage reservoirs in the distribution system on a daily basis. Therefore, the operating schedule varies with seasonal variations in water demand. During the winter months, the plant generally operates seven days per week for an eight hour period. Operational hours are extended during the high demand summer months, when the plant must operate up to twelve hours daily in order to keep the storage reservoirs full.

Booster Pumping Stations Serving Areas With Reservoirs

Those booster pumping stations that fill storage reservoirs are automatically controlled to maintain preset water levels. When sensors show that the water level in a reservoir has fallen below a preset threshold, the lead pump will activate and begin filling the reservoir to a high water level. If water demand on the reservoir is such that a single pump cannot maintain the water level, a lag pump (or pumps) will activate as necessary until the reservoir fills to a high water level.

Booster Pumping Stations Serving Areas Without Reservoirs

Those booster pumping stations that serve areas without storage reservoirs are automatically controlled to maintain a minimum discharge pressure at the pumping stations. When pressure sensors show that the discharge pressure has fallen below a preset threshold, the lead pump activates and pumps until the discharge pressure exceeds a high pressure level. If water demand in the pump station's service area is such that a single pump cannot maintain the pressure level, a lag pump (or pumps) will activate as necessary until the system pressure is restored.

Reservoirs

The reservoirs in the water distribution system are generally maintained between 80 and 100 percent full. This fluctuating volume represents the operating storage. The remaining storage is allocated to providing fire flow requirements and emergency reserves. In the case of Reservoir No. 15 in the North Valley, water levels are maintained at a much lower level due to limited demand in that portion of the distribution system.

Altitude valves control the flow into and out of Reservoirs No. 3, No. 4, No. 5, No. 6, and No. 11. These valves are designed to close when the reservoir is full and open when the system pressure drops. The other reservoirs in the distribution system float on the system.

Pressure Reducing Valves

There are two pressure reducing valve stations in the Grants Pass water distribution system. These stations control the flow of water from Pressure Zone 2 to Pressure Zone 2A. Pressure Zone 2A extends to slightly lower elevations than Pressure Zone 2 and thus requires some pressure reduction. Each station contains a single pressure reducing valve (one is a 10-inch valve and one is a 6-inch valve).

Supervisory Control and Data Acquisition (SCADA) System

The City upgraded the water distribution system SCADA system in 1999. The SCADA system monitors reservoir levels, pump operating status, and local pressures throughout the system. The central computer system for the man-machine interface is located at the water treatment plant.

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CHAPTER 3

WATER DEMAND ANALYSIS

This chapter presents the analysis of historic water production and water demand data for the City of Grants Pass. Analysis of these historical data allows for identification of the unique water use patterns that characterize the City and provides a basis for estimating future water demand in the community. Additional analysis relates the various measures of water demand (maximum monthly demand, maximum daily demand, and peak hour demand) to the average annual demand through the use of peaking factors.

The projection of future water demand is based on unit demand factors developed by land use type and corresponding customer classifications. These future demand projections provide the basis for assessing the adequacy of the existing water distribution system and planning for future improvements.

EXISTING WATER USE

There are several measures of water use that are important to analyze during the development of the water master plan. Following is a description of the influential water demand factors that will guide planning decisions with respect to the Grants Pass water distribution system:

- Annual average demand – A measure of the amount of water that must be obtained from the available sources of supply on an annual basis.
- Monthly average demand – Review of monthly average water demand illustrates seasonal variations in demand due to such factors as climate, irrigation, industrial production, and domestic use patterns.
- Maximum day demand – The maximum daily water demand is used to size booster pumping stations that serve areas with storage reservoirs. This measure of demand is also used along with fire demands to size storage reservoirs.
- Peak hour demand – The peak hour water demand is used to size pipelines and booster pumping stations that serve pressure zones without reservoirs.

Annual Average, Monthly Average, and Maximum Day Water Demand

The Grants Pass Water Treatment Plant operators record water production volumes for each day of operation. Analysis of this data allows for the identification of annual average, monthly average, and maximum day water demand. Tables 3-1 through 3-5 present historical water production data for the past five years, from 1995 to 1999.

Table 3-1. Grants Pass Water Use for 1995

Month	Average Daily Demand, mg	Peak Daily Demand, mg
January	2.23	3.84
February	2.30	3.52
March	2.38	3.00
April	2.59	3.61
May	3.74	5.99
June	5.03	8.20
July	5.55	6.73
August	6.48	8.32
September	5.36	6.83
October	3.52	4.94
November	2.96	3.71
December	2.63	3.64
Average	3.73	--
Peak Day	--	8.32

Table 3-2. Grants Pass Water Use for 1996

Month	Average Daily Demand, mg	Peak Daily Demand, mg
January	2.96	3.24
February	2.60	3.57
March	2.57	3.46
April	2.75	3.93
May	3.71	5.49
June	5.80	7.37
July	7.11	9.09
August	7.22	8.82
September	5.27	7.67
October	3.98	5.50
November	2.84	3.47
December	2.57	3.81
Average	4.11	--
Peak Day	--	9.09

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Table 3-3. Grants Pass Water Use for 1997

Month	Average Daily Demand, mg	Peak Daily Demand, mg
January	2.53	3.55
February	2.66	3.35
March	2.69	3.15
April	3.14	4.17
May	4.77	6.31
June	5.54	6.93
July	6.48	7.47
August	6.82	8.83
September	4.30	6.18
October	3.11	4.44
November	2.83	3.46
December	2.72	3.30
Average	3.97	--
Peak Day	--	8.83

Table 3-4. Grants Pass Water Use for 1998

Month	Average Daily Demand, mg	Peak Daily Demand, mg
January	2.50	3.21
February	2.62	3.47
March	2.76	3.11
April	3.02	4.90
May	3.33	4.34
June	5.73	7.52
July	7.08	9.15
August	7.62	9.47
September	6.31	8.63
October	3.64	5.66
November	2.65	3.94
December	2.78	4.06
Average	4.17	--
Peak Day	--	9.47

Table 3-5. Grants Pass Water Use for 1999

Month	Average Daily Demand, mg	Peak Daily Demand, mg
January	2.80	3.83
February	2.77	3.87
March	2.80	3.22
April	3.25	4.24
May	4.90	7.60
June	6.82	8.75
July	7.79	9.35
August	6.69	8.30
September	5.94	6.94
October	4.27	5.56
November	2.84	4.54
December	3.09	4.50
Average	4.50	--
Peak Day	--	9.35

From 1995 to 1999, the average annual demand increased from 3.73 mgd to 4.50 mgd. The highest peak daily demand was 9.47 mgd in August of 1998.

Per Capita Water Demand

Per capita water demand is a useful demand measure that is derived from this historical data. Table 3-6 presents the population for Grants Pass along with the average annual demand during the past five years which allows for calculation of the average demand in gallons per capita per day (gpcd). Ranging from 190 to 215, the average daily water demand is 202 gpcd. Note that this unit demand factor is based on water production and includes all uses: residential, commercial, industrial, public/institutional, and unaccounted. The variation in per capita demand for the different years reflects the regular variation in water use patterns caused by unsteady weather and end user demand characteristics.

Table 3-6. Grants Pass Water Use for 1995 to 1999, gpcd

Year	Population	Average Demand, mgd	Average Demand[a], gpcd
1995	19,660	3.73	190
1996	20,255	4.11	203
1997	20,535	3.97	193
1998	20,590	4.17	203
1999	20,935	4.50	215
Average	--	--	202

[a]Demands include all uses, including residential, commercial, industrial, and public/institutional.

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Unaccounted for Water

All water distribution systems experience losses of water during transmission from the treatment plant to the end user. These losses, known as unaccounted for water, result from many situations including unmetered customers, transmission system leaks, main breaks, faulty meters, fire fighting activities, system flushing, and other miscellaneous hydrant uses. Thus, the total volume of water metered for all end users is always somewhat less than the volume of water produced at the water treatment plant.

Since the City of Grants Pass meters water use for all customers, a comparison of water billing records and treatment plant production data provides a good estimate of the volume of unaccounted for water in the system. Table 3-7 shows the estimated volume of unaccounted for water in millions of gallons and also as a percentage of total production during the past two years. Since a water loss rate of 10 to 15 percent is considered good, the calculated unaccounted for water rate indicates that the distribution system is in good condition. The City is also conducting several programs that will reduce the unaccounted for water rate. These programs include residential meter replacements, commercial meter upgrades, and improved monitoring of hydrant use.

Table 3-7. Unaccounted for Water; 1998 and 1999

Year	Million Gallons	Percent of Total Water Production
1998	146	9.6%
1999	190	11.6%

Unit Demand Factors by Land Use Pattern

Water demand factors related to land use patterns are used to analyze water demand in the community. Based on historical billing data provided by the City's Utilities Department for 1998 and 1999, Tables 3-8 and 3-9 show water demand for each month within three land use pattern classifications: commercial, single family residential, and multi-family residential. The commercial classification includes general business, industrial, institutional and governmental-public land use categories. As indicated in the percentage summary of annual average demand by land use, the single family residential classification accounts for nearly half of the water used in Grants Pass.

Table 3-8. Grants Pass Water Use by Customer Class for 1998

Month	Demand (mgd)			
	Commercial	Multi-Family Residential	Single-Family Residential	Total
January	0.87	0.44	1.03	2.34
February	0.94	0.47	1.09	2.50
March	0.77	0.37	0.88	2.02
April	0.93	0.46	1.05	2.43
May	1.04	0.48	1.25	2.76
June	1.18	0.54	1.62	3.34
July	1.97	0.84	2.94	5.74
August	2.37	1.04	3.80	7.22
September	2.22	0.93	3.28	6.43
October	1.83	0.75	2.32	4.89
November	1.17	0.52	1.25	2.94
December	0.97	0.48	1.11	2.56
Annual average	1.36	0.61	1.80	3.77
Percent of total annual average demand	36.0%	16.2%	47.8%	100%

Table 3-9. Grants Pass Water Use by Customer Class for 1999

Month	Demand (mgd)			
	Commercial	Multi-Family Residential	Single-Family Residential	Total
January	0.90	0.47	1.09	2.47
February	0.93	0.49	1.01	2.43
March	0.81	0.39	0.90	2.10
April	0.97	0.48	1.12	2.57
May	1.09	0.51	1.31	2.90
June	1.73	0.81	2.60	5.13
July	2.19	0.96	3.50	6.64
August	2.18	0.97	3.48	6.64
September	2.19	0.96	3.08	6.23
October	1.66	0.74	2.35	4.75
November	1.25	0.74	1.40	3.39
December	0.92	0.49	1.05	2.47
Annual average	1.40	0.67	1.91	3.98
Percent of total annual average demand	35.2%	16.8%	48.0%	100%

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To develop a unit demand factor for the three different land use patterns, the water demand data presented in Tables 3-8 and 3-9 are combined with estimated areas for each of the land use classifications. Table 3-10 summarizes the acreages by land use classification and pressure zone for all areas receiving water service in Grants Pass. This summary was derived from analysis of a database containing the zoning designations for each parcel of property connected to the water distribution system for Pressure Zones 1 through 4. The acreage for Pressure Zone NV is based on an estimate of the properties connected to the system in the North Valley area. The quotient of water demand and acreage yields a unit demand factor for each land use classification in gallons per acre per day (gpad), as summarized in Table 3-11. Since the billing record information presented in Tables 3-8 and 3-9 does not include unaccounted for water, the calculation of these unit demand factors includes an allowance for unaccounted for water.

Table 3-10. Land Use by Pressure Zone in Acres

	Customer Class[a]	PZ 1	PZ 2	PZ 3	PZ 4	PZNV	Total	%
	Commercial	924	125	57	0	40	1,146	32%
	Residential	1,197	594	114	67	5	1,977	56%
	Multi-Family	358	42	35	0	0	435	12%
Total		2,479	761	206	67	45	3,558	100%
%		70%	21%	6%	2%	1%	100%	

[a]The Commercial customer class includes commercial, industrial, and public connections to the water distribution system.

Table 3-11. Unit Demand by Customer Class

Customer Class	1999 Average Demand[a] (mgd)	Land Use Area (Acres)	Average Unit Demand (gal/acre day)
Commercial/Industrial/Public	1.56	1,146	1,400
Multi-Family Residential	0.75	435	1,700
Single-Family Residential	2.13	1,977	1,100

[a]The 1999 average demand is based on billing records plus an additional 11.6 percent to reflect unaccounted for water

Peak Hour Demand

The peak hour demand on the water distribution system typically occurs during the hottest, driest period of the year when customers are heavily irrigating landscaped yards and parks. For the City of Grants Pass, the peak hour demand usually happens in the month of August during the peak day demand. In order to evaluate this peak hour demand, hourly water level data was collected from each of the reservoirs in the distribution system during the summer of 1999. These data were analyzed in combination with the water production rate for the water treatment plant to

identify the peak hour demand on each day for which data was available. Based on this analysis, the peak hour demand is estimated to be 4.5 times the average annual demand.

Historical Peaking Factors

The water demands observed over the past five years can also be expressed as a ratio to the annual average demand known as peaking factors. Although peaking factors vary significantly from user to user, these historical peaking factors are useful for comparing system-wide water use patterns in Grants Pass to other communities and for projecting future water use patterns. Table 3-12 identifies Grants Pass water demand patterns for the past five years and shows peaking factors based on ratios to annual average demand. The identified peak hour demands for the system are not actual measurements, but rather estimates derived from the reservoir level analysis conducted during the summer of 1999 as described above. Table 3-13 summarizes the average peaking factors for the system. These values are fairly typical for a Western Oregon community. In general, they are slightly higher than the peaking factors for Corvallis and slightly lower than values for Portland.

Table 3-12. Maximum Month, Peak Day, and Peak Hour Demand Ratio for Grants Pass 1995 to 1999

Year	Annual Average Demand, mgd	Maximum Month Demand, mgd	Peak Day Demand, mgd	Peak Hour Demand, mgd	Ratio of Maximum Month to Annual Average Demand	Ratio of Peak Day to Annual Average Demand	Ratio of Peak Hour to Annual Average Demand
1995	3.73	6.48	8.32	17.01	1.74	2.23	4.56
1996	4.11	7.22	9.09	18.60	1.76	2.21	4.52
1997	3.97	6.82	8.83	18.10	1.72	2.22	4.56
1998	4.17	7.62	9.47	19.40	1.83	2.27	4.66
1999	4.50	7.79	9.35	19.20	1.73	2.08	4.26
Average					1.76	2.20	4.51

Table 3-13. Peaking Factor Summary[a]

Description	Factor
Maximum month demand	1.8
Maximum daily demand Average for city	2.2
Peak hourly demand Average for city	4.5

[a]The average demand multiplied by the peaking factor yields the respective peak demand.

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FUTURE WATER DEMAND

The land use demand factors developed in the previous sections provide a basis for projecting future water demand in Grants Pass. The land use demand factors can be used in conjunction with land development projections to estimate water demand.

Although the timing of land use development within the UGB is unknown, information is available regarding the current zoning designation for all properties within the UGB. Table 3-14 summarizes the acreage of properties within the UGB according to land use, differentiating between properties that are currently receiving water service and those that will connect to the water distribution system in the future. Using the unit demand factors developed for these land use classifications, the table also projects average annual water demand at the UGB build-out condition. This analysis assumes the existing mix of residential/commercial properties will stay the same in the future.

Table 3-14. Land Use Based Water Demand Projections for UGB Build-Out

Land Use	Existing Acreage	Future Acreage	Total Acreage	Unit Demand, gallons/acre-day	Estimated Average Annual Demand, mgd
Commercial	1,146	598	1,744	1,400	2.4
Single-Family Residential	1,977	2,419	4,396	1,100	4.8
Multi-Family Residential	435	440	875	1,700	1.5
Total	3,558	3,457	7,015		8.7

Based on this estimate of the UGB build-out average annual demand, the future maximum month, maximum day, and peak hour demand can be estimated using historical peaking factors. Table 3-15 summarizes existing water demand and projections for the build-out condition.

Table 3-15. Water Demand Summary for Grants Pass

	Current Water Demand, mgd	Future Total Water Demand, mgd
Average Annual	4.5	9
Maximum Month	8	16
Maximum Day	10	20
Peak Hour	20	40

In addition to properties within the UGB, there are properties contiguous to the boundary that are likely candidates for receiving water service in the future. Based on information from City planning staff, a rough estimate of the area of these properties is 400 acres. Assuming that these properties largely fall within the single family residential and industrial land use classifications,

the additional acreage would increase the annual average demand on the system by approximately 0.5 mgd.

Another location with the potential for future water system expansion is the North Valley/Merlin area. The existing North Valley Reservoir and Pump Station have the capacity to accommodate additional demand in the area. Based on the existing pumping capacity at the North Valley station, this portion of the system can serve a maximum day demand of 570 gpm. Assuming similar development patterns to the rest of Grants Pass, this demand rate is equivalent to approximately 300 acres of service area. Currently the North Valley system serves approximately 45 acres.

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CHAPTER 4

WATER DISTRIBUTION SYSTEM SERVICE STANDARDS

As the provider of water service to the community, the City of Grants Pass needs to establish benchmarks for service quality that can be used to measure the performance of the water utility. With respect to water distribution system service, customers expect the City to provide water at an adequate pressure for a full range of demand conditions. Also, since the water distribution network is an integral part of the fire fighting system, customers expect the City to provide adequate fire flow supplies to protect buildings and other properties within the community. The service standards presented in this chapter are provided as a basis for evaluation of the City's existing treated water supply and distribution system, and to guide the planning and design of improvements to the system necessary to meet future demands.

The following discussion presents service standards for water quality, quantity, and pressure, as well as the minimum supply levels for fire protection. The standards set forth in this chapter are derived from regulations, rules, and recommendations established by a variety of sources including the Oregon State Health Division (OSHD), the Environmental Protection Agency (EPA), the American Water Works Association (AWWA), the Insurance Services Office, Inc. (ISO), and the Uniform Fire Code (UFC).

WATER SERVICE QUALITY STANDARDS

Water service quality standards largely pertain to protecting public health and consistently delivering a satisfactory product to the customer. Most of the water quality considerations are related to supply and treatment issues and are not the subject of this plan. In a water distribution network, attention to enhancing the reliability of the system under all conditions is an important part of maintaining high quality water service. Reliability is achieved through a number of system features including adequate storage; redundant pumping, transmission, and rechlorination where required; and alternate power supplies. Reliability and water quality are also improved by designing looped water distribution pipelines and avoiding dead-end distribution mains whenever possible. Proper valve placement is also necessary to maintain reliable system operation under normal and abnormal operating conditions.

DISTRIBUTION AND STORAGE SYSTEM STANDARDS

The water supply distribution and storage system must allow for effective service under normal operations as well as during times of system stress, such as maximum day, fire flows, and peak hour demands. The following operational performance criteria are used in this Master Plan to evaluate the adequacy of the City's existing treated water distribution system, including treated water storage facilities. These criteria also provide the basis for development of a program of

improvements that will provide effective service as the City grows in the future. These criteria reflect standard water system planning criteria and design guidelines which have been developed by various State agencies and other water utilities to evaluate water systems under both normal and stressed demand conditions, including maximum day, fire flows, and peak hour demand conditions. The criteria used in this Master Plan are recommended to the City as long term system performance criteria and are defined below for critical operational conditions impacting individual water system components.

Water Supply and Treatment Capacity

The following criteria should be used to assess the adequacy of the City's water supply and treatment capacity.

Average Annual Demand. The reliable yield of all sources of supply needs to exceed the projected annual demand on the system. The definition of reliable yield of water supplies is that which can be delivered to the City during the worst drought.

Maximum Day Demand. Total potable water production and supply delivery capacity shall be equal to or greater than the maximum day demand. It is recommended that the total maximum potable supply capacity be at least ten percent greater than the maximum day demand. The treatment plant capacity analysis should account for the planned hours of operation if the plant will not be operated full time.

Maximum Day Demand plus Fire Flow. The water distribution system shall have the capability to meet a system demand condition equal to the occurrence of a maximum day demand concurrent with a fire flow event. If the supply to an individual pressure zone is from a pumped source, the supply requirement shall be met with the largest pump out of service. Since the plant does not operate at all times, the fire flow shall be met from treated water storage whenever possible in those pressure zones with reservoirs. Fire flow requirements are based on the UFC and are shown in Appendix 1. The minimum fire flow in the UFC is 1,000 gallons per minute for single and two-family dwelling units with less than 3,600 square feet.

Peak Hour. Peak hour demand shall be met from supply sources and treated water storage reservoirs.

System Pressure Requirements

Under normal operating conditions, water pressure in the distribution systems shall range between 35 and 100 psi. An analysis of the existing water distribution system indicates that this pressure range is generally consistent with the storage facility designs and pressure zone designations. The reservoir overflow elevations and pressure zone elevation ranges are such that the maximum head (elevation difference between a full reservoir and the lowest service connection) does not exceed 240 feet or 104 psi. The use of pressure reducing valves between pressure zones is acceptable when approved by the City Engineer. The minimum head (elevation difference between the low reservoir operating level and the highest service connection) does not

fall below 83 feet or 36 psi. Consistent with Oregon Department of Health rules, the minimum system pressure under fire flow conditions shall be 20 psi at the property line.

Some customers are required to install reduced pressure principle backflow preventers to protect the water distribution system from potential hazards associated with the customers' water use activities. These backflow preventers cause an extra pressure drop prior to delivery of the water to the customer. Depending on the location of the customer within the pressure zone, this pressure drop may move the service pressure below the service standard minimum. Under such conditions, adjustments to the pressure zone boundaries may be necessary to provide adequate service pressures.

System Storage Requirements

Criteria have also been defined for determining treated water storage capacity needs within the distribution system and individual pressure zones to meet diurnal operational peaks and emergency conditions. Storage requirements can generally be categorized into the following three components:

- operational storage
- fire flow storage
- emergency storage

A discussion of the typical design characteristics for these three components follows.

Operational Storage. Over any 24-hour period, water demands will vary. Typically, water demands will be high in the morning when people are getting ready to go to work and school, then will decline to some nominal baseline level (corresponding to water use patterns of commercial/industrial areas). They will then begin to increase again in late afternoon, reaching a higher water demand in the early evening as people return home from work. In Grants Pass, the water production rate varies as well since the water plant does not operate 24 hours per day. The water treatment plant is operated during normal working hours and shuts down at night to reduce operator labor costs. The storage volume used during periods when demand exceeds the production rate is called operational storage. Analysis of water demand data from the summer of 1999 indicates that up to 45 percent of daily water demand take place when the plant is not in operation. As a result, the effective operating storage used in Grants Pass is up to 45 percent of the daily water supply. When a water plant is operating full time and producing water at a rate equal to the daily demand, the operating storage is typically equal to 25 percent of daily demand. Therefore, the operating storage requirements are generally set at 25 percent of the maximum day demand. Since the Grants Pass water plant will most likely extend its hours of operating as system demand increases, this lower percentage of operating storage will ultimately become the appropriate requirement in this community as well. Based on this information, the current operational storage requirement should be 45 percent of the total volume of water used on a maximum day in order to be consistent with the current operating strategy. For assessments of future storage requirements, the operational storage requirement should be based on 25 percent of maximum day demand to account for the prospective longer hours of plant operation.

Fire Storage. Discussions with Public Safety Department staff indicate that their fire flows requirements are based on the UFC. Minimum fire flows are to be met concurrently with a maximum day demand condition, while maintaining a minimum residual system pressure of 20 psi. Fire flows and the expected duration establish treated water fire storage requirements. The UFC fire flows and associated duration to be used in this master planning effort are shown in Appendix 1.

The storage requirements are based on flow [in gallons per minute (gpm)] requirements for the size of building (in square feet) and type of construction (wood frame, metal, masonry, installation of sprinklers, *etc.*). Once the fire flow requirement is established, it is multiplied by the required duration. This calculation will provide an estimate of the total volume needed for fire flow storage. The highest fire flow requirement for Grants Pass is for school facilities. These facilities typically have a UFC fire flow requirement of 4,000 gpm for a duration of four hours. The resulting volume needed for fire flow reserve is 960,000 gallons. A typical fire flow requirement for a commercial development is 2,000 gpm for a duration of two hours, resulting in a storage volume requirement of 240,000 gallons. Another important fire flow value is the requirement for residential areas. According to the UFC, typical one- and two-family residential areas are required to have two hour duration fire flows of 1,000 gpm for homes less than 3,600 square feet and 1,500 gpm for homes more than 3,600 square feet. This fire flow requirement equates to a storage volume requirement of 120,000 and 180,000 gallons respectively.

The largest fire flow requirements in Pressure Zones 1 and 2 are for school facilities. The largest fire flow requirement in Pressure Zone 3 and the North Valley are for commercial businesses. The fire flow requirement for all other pressure zones in the system should be based on a large home. Required fire flow volumes must be stored in reservoirs located within the pressure zone or readily available by gravity from storage in higher pressure zones.

Emergency Storage. A reserve of treated water is also required to meet demands during emergency outage periods, when normal supply is interrupted. Such conditions may arise due to loss of the water plant due to a power failure, loss of raw water supply, pumping equipment or pipeline failure, or the need to take facilities out of service for repair. Since the risk of an emergency situation is different for every town, the amount of reservoir volume allocated to emergency storage is different also. The required emergency storage volume is a function of several factors including the diversity of the sources of supply, redundancy and reliability of the production facilities, and the anticipated length of the emergency outage.

The Rogue River is the sole source of supply for the Grants Pass water system. Although the reliability and quality of the City's water supply has been excellent, it is vulnerable to temporary contamination by chemical spills into the Rogue River.

Consideration of a specific scenario is useful for preparing the City to manage emergency storage supplies during such an emergency event. Following are important issues that the City must be prepared to consider with respect to an emergency scenario:

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- If the Rogue River became contaminated, it is estimated that it would take up to three days to allow the contamination to pass by the water treatment plant or to modify the process to treat the contaminated water.
- Immediately following the water treatment plant shutdown, the public must be notified and advised to adopt water rationing measures to prolong the availability of emergency storage supplies.
- If the shutdown were to occur during a period of peak day demand, it would take up to 12 hours for water rationing measures to be adopted, after which the demand would likely drop to one-half the annual average day demand for the remainder of the shutdown period.
- Response to an emergency depends on the ability of the City to reach its citizens with the necessary information. An extensive emergency curtailment plan is essential to effectively reduce water demand during an emergency.

Since every city considers a different set of factors in their risk evaluation, the volume of storage designated for emergency situations varies significantly from community to community. Review of other water system planning criteria for communities with surface water supply situations shows that emergency storage volumes vary from 25 percent of maximum day demand to 150 percent of maximum day demand. A 1998 Grants Pass City Council resolution set the emergency storage requirement at 75 percent of maximum daily demand. This resolution is included as Appendix 2.

Reservoir storage volumes must also be carefully selected to preserve high quality water in the distribution system. Disinfectant residuals will dissipate over time, so if water is stored for too long, there is a risk of bacterial regrowth. The recommended storage capacities should provide adequate water reserves without requiring the addition of disinfectant at the reservoir. Proper reservoir management plays a large role in ensuring that the average residence time of water supplies does not exceed two to three days. When a reservoir does not accomplish adequate turnover, the current City policy is to install rechlorination facilities

Total Water Storage. The minimum treated water storage capacity in the system available by gravity flow to each pressure zone should equal the sum of the following:

- Operational. The storage allocated for meeting diurnal demand peaks should be equivalent to 45 percent of the maximum day demand. This storage volume should be located within the pressure zone or available by gravity to the pressure zone. The operating storage volume can be reduced to 25 percent of maximum day demand when the water plant eventually reaches full time operation.
- Fire Flow. The storage allocated to provide fire flows should be equivalent to the maximum fire flow in the pressure zone times the duration the flow rate must be maintained. This volume is estimated as 960,000 gallons for Pressure Zones 1 and 2, 240,000 gallons for Pressure Zone 3, and 180,000 gallons for all other pressure zones.

- Emergency. The minimum emergency storage volume allocated for providing water during periods when normal supply is interrupted should be equivalent to 75 percent of the City's maximum day demand.

A table comparing the existing storage volume in the system and the recommended minimum storage volume is provided in Chapter 6, "Water Distribution System Evaluation."

Pipeline Networks

The pipelines and transmission mains in the City's distribution system will generally be sized based on the criteria described below for normal, maximum day and peak hour demand conditions.

- Service pressures shall be maintained between a maximum of 100 psi and a minimum of 35 psi.
- New reservoirs shall be placed so the overflow elevation is 100 feet above the normal upper service elevation of the pressure zone it is serving. Most of the existing reservoirs satisfy this standard.
- Fire flows will be provided by storage unless a specific exception is approved by the City. Booster stations can be allowed for small areas under the condition that they provide adequate flows, pressures, and reliable operation. These small areas will be defined as isolated zones with less than approximately 50 connections and where elevated storage is not practical.
- The minimum allowable residual pressure at hydrants located in the immediate vicinity of a simulated fire shall be 20 psi or greater during a maximum day demand.
- The minimum residual pressure during a peak hour demand shall be 35 psi.

Pump Stations

As mentioned above, if pumping facilities are to be used to meet the demands of a pressure zone with storage, sufficient pumping capacity shall be provided so that the maximum day demand can be supplied with the largest pump out of service. The pumping facility shall also be equipped with an emergency generator of sufficient capacity to operate the pumping plant at its rated capacity. This minimum supply requirement sets the pumping capacity requirement, if the pressure zone includes adequate treated water storage at sufficient elevation to allow gravity flow to serve the zone. If the pressure zone does not have storage reservoirs, the pump station should supply peak hour demand with one pump out of service. The station should also have two additional pumps to meet fire flow requirements and should be equipped with hydropneumatic tanks.

Valves

Valve location and spacing are important considerations in the design of a water distribution system. Pipelines must include an adequate number of properly located valves to allow for

isolation of pipeline sections in the event of maintenance operations or new construction. ISO provides standards for valve spacing as identified in Table 4-1 for pipelines according to their function. A general guideline for locating valves in the distribution system is that smaller branch mains should be equipped with a valve so that any service problems on the branch pipeline does not require a shut off of the major transmission line. Within the distribution grid, placement of a valve on all but one leg of tees and crosses will minimize the extent of a service disruption during system work. For the same reason of localizing service disruptions, system design should always avoid direct service taps into transmission pipelines. Reservoirs shall be equipped with seismic valves that prevent drainage after a significant earthquake or a remote controlled shut-off valve.

Table 4-1. ISO Maximum Valve Spacing Standard

Pipeline Function	Maximum Spacing
Supply pipeline	1 mile
Transmission pipeline	¼ mile
Residential distribution	800 feet
Commercial distribution	500 feet

Hydrants

Fire hydrants are dispersed throughout the distribution system to provide the emergency flows required for fire protection. The requirements for spacing fire hydrants are defined in the Uniform Fire Code and have been modified by the City's development codes as shown in Table 4-2. Where required fire flows exceed 1,500 gallons per minute, the water supply must be provided by more than one hydrant. In addition to the maximum spacing requirements, any parts of a residence must be within 500 feet of a fire hydrant or any part of a business must be within 300 feet of a hydrant. Distances are measured along the route that the fire department will use to deploy the fire hose.

Table 4-2. Uniform Fire Code Hydrant Distribution Requirements

Land Use Category	Maximum Hydrant Spacing, feet
Residential	500
Commercial, general	300
Industrial	300
Schools	300
Offices	350

CHAPTER 5

DISTRIBUTION SYSTEM MODELING

A computer based hydraulic model of the Grants Pass water distribution system was developed as part of the master planning effort to analyze the capability of the system to meet current and projected demands. Specifically, the model was used for the following types of analysis:

1. Evaluation of existing water facilities and operating strategies
2. Identification of current deficiencies in the distribution system
3. Development of recommended capital improvements
4. Sizing of system extensions

In addition to the system analysis work conducted as part of this master plan, the model will also be useful to the City in the future for a variety of purposes. The model will be an excellent tool for evaluating system planning issues and the effects of different operating strategies. The model will also serve as a good predictor of impacts to the system from proposed development projects.

Computer Modeling Program

The computer program CYBERNET 3.1 by Haested Methods was used to model the distribution system. The program was specifically developed for conducting steady-state and extended period analyses of a pressure pipe network. The model provides output in the form of local pressures and flows.

The CYBERNET computer model blends the capabilities of several types of programs (including CADD, GIS, database, and a hydraulic analysis engine) to facilitate the analysis and design of a water distribution system. The model incorporates all of the important hydraulic features of the distribution system including pipelines, pump stations, storage reservoirs, and valves. In addition to these hydraulic features, the model also establishes junction nodes at all intersections between pipes as well as any other important locations in the distribution system. The model links a graphical representation of the water distribution system to a database that describes the physical characteristics of each hydraulic feature (i.e. length, diameter, and roughness coefficient for a pipeline) and the physical and demand conditions at junction nodes (i.e. ground elevation and water demand).

CYBERNET also includes a scenario manager feature that allows the modeler to quickly evaluate a variety of water demand and operating conditions when analyzing the system. Using this feature, the modeler can conduct an hydraulic analysis of different combinations of demand alternatives, system configuration alternatives, and operating strategy alternatives. For example, to evaluate available fire flows in the distribution system, the modeler can create a scenario that

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includes the maximum day demand, an 80 percent full storage reservoir configuration, and a night time operating strategy.

Grants Pass CYBERNET Model

The City of Grants Pass prepared an initial version of the CYBERNET model for the existing water distribution system based on their previous system model and updated system maps. The initial version of the model included all of the basic system data for pipelines and junction nodes. This version of the model was then delivered to West Yost & Associates (WYA) for additional refinement to ready the model for the master planning work. The key components of the existing system model included the following:

- The majority of system pipes 6-inches in diameter.
- All system pipes 8-inches and larger in diameter.
- All existing pumping stations.
- All existing storage reservoirs.
- All existing pressure reducing valves.

The City also provided a database of property parcels within the Grants Pass urban growth boundary that included information on acreage and zoning designation. Each parcel was associated with a CYBERNET junction node as a basis for allocating water demand across the water distribution system. These parcels were then categorized as either single family residential, multi-family residential, or commercial land use in order to be consistent with the demand analysis categories developed in Chapter 3. Using the acreage based demand factors developed in Chapter 3, each junction node was assigned an average water demand according to the acreage and land use designation of parcels drawing water from that node.

Each junction node in the system has a four number suffix label. The first number in the suffix label corresponds to the Pressure Zone in which the node is located. For example, J-1011 is in Pressure Zone 1, J-2216 is in Pressure Zone 2, and so on.

Model Calibration

The purpose of calibrating the hydraulic model is to confirm that the computer model accurately represents the operation of the water distribution system. Hydrant flow testing in the field is an important tool for developing measurements of actual system pressures and flows that can be compared to model predictions. These tests are useful in evaluating pipeline friction factors (specifically Hazen Williams equation coefficients or C-factors) and ensuring that the model closely represents actual observed conditions. The hydrant flow tests for this calibration process were performed by City water distribution personnel, the City Fire Department, and WYA staff on March 28 and 29, 2000.

The baseline C-factors used in the model were selected based on the material and age of the pipelines. The vast majority of pipelines in the distribution system are cast iron and ductile iron; minimal amounts of other pipe materials are present. City staff reviewed the available

construction drawings and categorized each pipeline in the model according to the decade in which it was installed. According to City records, pipelines installed prior to 1972 were predominantly of cast iron construction and pipelines installed after 1972 were predominantly of ductile iron construction. C-factors for these materials can range from a low of around 40 for unlined cast iron pipes in very poor condition to a high of 140 for newly installed, cement-lined pipe. This information was used along with pipeline condition information from the City's maintenance staff to assign an appropriate C-factor to each pipeline. Table 5-1 summarizes the C-factors selected for each decade of pipeline installation.

Table 5-1. Pipeline Age-Based C-Factor Summary

Decade of Pipeline Construction	Hazen Williams C-Factor
1920s	65
1930s	75
1940s	85
1950s	95
1960s	105
1970s	115
1980s	125
1990s	135
2000	140

After refining the City's hydraulic model, WYA worked with City maintenance staff to select 11 hydrant flow test sites. Selection of the hydrant test locations was based on pipeline size, age, location within the different pressure zones, and configuration of the surrounding pipeline system.

The general testing procedure for a hydrant flow test is outlined below:

- Measure the static pressures at the designated test hydrant and at each observation hydrant.
- Flow the designated test hydrant and measure the discharge flow and pressure.
- Measure the residual pressures at the designated test hydrant and at each observed hydrant while the test hydrant is flowing.

In addition to the field measurements taken at the flowing and observation hydrants, other important system data were recorded during the hydrant flow tests. Using the water treatment plant's SCADA system, plant staff monitored reservoir levels and pump operating status throughout the water distribution system during the test period. These data were critical for establishing the water system context within which the hydrant flow tests took place. A review of plant water production data and analysis of the changes in reservoir levels also allowed for calculation of system demand on the flow test days. Water demand during the hydrant flow tests

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was approximately 2.85 mgd, approximately two-thirds of the annual average demand in Grants Pass.

Calibration Results. Based on the field and SCADA system data collected on March 28th and 29th, each hydrant flow test was simulated using the CYBERNET model of the water system. CYBERNET predictions were then compared to the field measurements to evaluate the need for adjustments to pipeline friction coefficients, ground elevations, or other system variables. Overall, the required adjustments to the baseline input data were very minor and calibration of the CYBERNET model was readily successful.

The model's ability to accurately predict head losses in the system during high flow conditions is of primary importance. This ability is necessary for verifying the presence of adequate pressures in the system during critical fire flow conditions. During a hydrant flow test, these head losses in the system correspond to the difference between static and residual pressures at the observed hydrants. Tables 5-2 through 5-11 summarize the static, residual, and differential pressures for both the field test data and the calibrated model predictions. The goal of our calibration effort was to achieve no greater than a 3 to 4 psi difference between the field hydrant test differential and the calibrated model differential. This goal was achieved with an average difference in differentials of 1.5 psi.

The static pressures in the system are largely related to the elevation of the test hydrant, pump operating status, and reservoir levels. However, the exact nature of local background flows at the time of the flow test can also influence the static measurement since they induce a pressure drop in the system. The average difference between the measured and modeled static pressures is 2.25 psi and in general they are well within 5 psi of each other. Otherwise, differences in the modeled and actual local background flows or slight pressure gauge calibration problems are the most likely explanation for discrepancies in static measurements.

Table 5-2. Field Test #1
Ironwood and Webster Rd.
6", 8", and 12" lines in Pressure Zone 1 – 1990s Installation
Hydrant Test Flow Rate = 1,540 gpm

		Field Data			Modeled Data			Comparison
Hydrant	Node	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Of Differential Pressures (psi)
733 (F)	1411	91			89			
659	1239	87	60	27	90	62	28	1
732	1412	89	60	29	90	62	28	1

(F) Flowing hydrant

Table 5-3. Field Test # 2
Fairview Avenue between Agness and Foothill
8" line in Pressure Zone 1 – 1980s Installation
Hydrant Test Flow Rate = 1,530 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
506 (F)	1413	60			58			
505	1417	60	55	5	59	56	3	2
507	1418	57	50	7	56	52	4	3
508	1419	50	45	5	52	48	4	1

(F) Flowing hydrant

Table 5-4. Field Test #3
Evelyn Ave. and Hawthorne Ave.
8" and 16" lines in Pressure Zone 1 – 1930s Installation
Hydrant Test Flow Rate = 1,790 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
109 (F)	1420	64			65			
115	1374	65	59	6	62	58	4	2
116	1421	65	60	5	67	63	4	1
119	1078	55	51	4	57	53	4	0

(F) Flowing hydrant

Table 5-5. Field Test #4
Forestview and West Harbeck
8" line in Pressure Zone 1 – 1990s Installation
Hydrant Test Flow Rate = 1,070 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
791 (F)	1000	57			56			
790	1422	60	36	24	60	38	22	2
709	1423	65	42	23	65	46	19	4

(F) Flowing hydrant

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Table 5-6. Field Test #5
Southridge and Harbeck Rd.
8" line in Pressure Zone 1 – 1980s Installation
Hydrant Test Flow Rate = 1,410 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
610 (F)	1266	52			50			
609	1424	56	44	12	54	41	13	1
608	1425	60	54	6	59	52	7	1

(F) Flowing hydrant

Table 5-7. Field Test #6
Millbanks and M St.
12" lines in Pressure Zone 1 – 1950s Installation
Hydrant Test Flow Rate = 1,590 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
206 (F)	1361	75			74			
24	1426	70	65	5	75	72	3	2
722	1427	75	70	5	73	70	3	2
611	1428	72	67	5	74	71	3	2

(F) Flowing hydrant

Table 5-8. Field Test #7
Highland Ave. at Sandy
12" line in Pressure Zone 2 – 1980s Installation
Hydrant Test Flow Rate = 1,930 gpm

Hydrant	Node	Field Data			Modeled Data			Comparison Of Differential Pressures (psi)
		Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	
550 (F)	2159	89			89			
291	2007	84	73	11	87	76	11	0
548	2158	85	70	15	91	80	11	4

(F) Flowing hydrant

Table 5-9. Field Test #8
Scoville Rd and Ausland
12" line in Pressure Zone 3 – 1980s Installation
Hydrant Test Flow Rate = 820 gpm

		Field Data			Modeled Data			Comparison
Hydrant	Node	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Of Differential Pressures (psi)
535 (F)	3047	50			43			
536	3070	55	46	9	52	42	10	1
537	3071	66	59	7	64	55	9	2

(F) Flowing hydrant

Table 5-10. Field Test #9
Morgan Lane and Candler
8" lines in Pressure Zone 3 – 1980s and 1990s Installation
Hydrant Test Flow Rate = 1,960 gpm

		Field Data			Modeled Data			Comparison
Hydrant	Node	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Of Differential Pressures (psi)
416 (F)	3067	95			95			
765	3072	87	80	7	83	77	6	1
829	3026	87	80	7	89	81	8	1

(F) Flowing hydrant

Table 5-11. Field Test #10
Morgan Lane and Crown
12" line in Pressure Zone 3 – 1990s Installation
Hydrant Test Flow Rate = 1,910 gpm

		Field Data			Modeled Data			Comparison
Hydrant	Node	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Static Pressure (psi)	Residual Pressure (psi)	Differential Pressure (psi)	Of Differential Pressures (psi)
831 (F)	3027	72			70			
764	3073	82	79	3	75	72	3	0

(F) Flowing hydrant

Reservoir 11 Flow Monitoring. In addition to the hydrant flow tests that were conducted throughout the water distribution system, field measurements of the Reservoir 11 fill rate were also compared with CYBERNET predictions as part of an operational strategy assessment for the new reservoir. Water treatment plant staff recorded the Reservoir 11 fill rate along with water

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levels in all of the Pressure Zone 1 reservoirs during the week of May 1, 2000. WYA selected the field data from the morning of May 3, 2000, during the treatment plant start-up as the basis for comparison with the model. At that time, plant staff measured a Reservoir 11 fill rate of approximately 1,100 gpm with three pumps in operation. Using the monitored Pressure Zone 1 reservoir levels and an assumed system demand equal to the average day, the CYBERNET model predicted a Reservoir 11 fill rate of 1,060 gpm. This information is summarized in Table 5-12.

Table 5-12. Reservoir 11 Flow Monitoring

Location	Plant Measurement (gpm)	Model Prediction (gpm)	Comparison of Differential Flow Rate (gpm)
Reservoir 11	1,100	1,060	40

Extended Period Analysis - Memorial Day Weekend 2000. Finally, the model was used to simulate changes in storage reservoir water levels during a 38-hour period while the water treatment plant was not operating over Memorial Day Weekend 2000. To account for the dynamic conditions in the distribution system over this length of time, the extended period analysis required more detailed data inputs than the steady state analyses conducted for the hydrant flow tests. The additional information required for an extended analysis include a diurnal curve for water demand in the system and pump on/off timing for each distribution system booster pump, all of which was available from SCADA system records for this 38-hour period. Some redistribution of demand among the pressure zones was also necessary to account for increased demand in the residential areas and reduced demand in the commercial areas due to the holiday. With these adjustments in place, the model was able to predict the final water levels in the reservoirs to within an average of 1.1 feet of their measured level as shown in Table 5-13. The largest discrepancies between measured and predicted water levels were for Reservoirs No. 4 and No. 13. These two reservoirs are the smallest in the system, where the level differences represent relatively small volume differences. Given the generalized methodology used to distribute demand across the system, the predictions from the 38-hour extended period analysis are very accurate.

Calibration Conclusion. The comparative analysis of field measurements and model predictions confirms that the calibrated CYBERNET model is capable of simulating operation of the existing water distribution system with a high degree of accuracy. Therefore, the model provides an excellent basis for analyzing the adequacy of existing facilities, analyzing proposed plans for system expansion and modification projects, evaluating different operational strategies, and designing specific improvements to the system.

Table 5-13. Memorial Day Weekend Extended Period Analysis

Reservoir	Beginning Reservoir Level at Plant Shut Down *May 28, 16:00* (feet)	Ending Reservoir Level at Plant Start-Up *May 30, 06:00* (feet)	Ending Reservoir Level CYBERNET Prediction *May 30, 06:00* (feet)	Difference between CYBERNET And Measured Level (feet)
3	17.6	2.3	3.2	0.9
4	18.5	14.7	17.6	2.9
5	28.3	14.0	12.9	1.1
6	27.1	21.6	22.2	0.6
8	27.9	21.0	20.8	0.2
11	29.6	21.2	19.9	1.3
13	7.4	7.4	5.3	2.1
15	14.4	13.4	13.4	0
Average				1.1

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CHAPTER 6

WATER DISTRIBUTION SYSTEM EVALUATION

The Grants Pass water distribution system was analyzed to evaluate its performance and capacity under current and future demand conditions relative to the service standards defined in Chapter 4. This analysis identified system improvements necessary to maintain adequate performance through build-out of the urban growth boundary (UGB). These improvements were developed to either eliminate existing deficiencies in system performance or expand service to satisfy community growth. Each improvement project was assigned a general timing estimate in one of the following four future periods:

- Year 2000 to 2005
- Year 2005 to 2010
- Year 2010 to 2020
- Year Post 2020

In this manner, the identified improvements and estimated timings form the basis for the recommended capital improvement program presented in Chapter 7.

The elements of the water distribution system evaluated in this chapter include water treatment plant capacity, treated water storage capacity, booster pumping capacity, and pipeline network performance with respect to current and future demand conditions. These evaluations were based on the demand projections presented in Chapter 3 and the results of computer model hydraulic analyses.

WATER TREATMENT PLANT CAPACITY

The Grants Pass water treatment plant has a maximum treated water production capacity of 18 mgd. However, as noted later in the pumping capacity analysis, the firm pumping capacity of the plant is currently limited to approximately 16 mgd. As specified in Chapter 4, "Water Distribution System Service Standards", the available water treatment plant capacity should be sufficient to meet anticipated maximum day demand with at least an additional ten percent capacity available. The additional ten percent is necessary to allow for backwashing filters, meeting drinking water quality standards with difficult raw water, or repairing equipment failures. As the single source of supply for the system, the water treatment plant must provide the entire required capacity.

Table 6-1 summarizes the treatment plant capacity evaluation for current and build-out demand conditions. The existing water treatment plant has ample capacity relative to current demand. This situation allows the City to operate the water plant on a part-time basis even during the current water demand peaks in the summertime. Although the plant will eventually require a

capacity expansion of approximately 4 mgd prior to build-out, the existing plant capacity is anticipated to be sufficient for the near term future. An extension of the plant operating hours will forestall the need for treatment capacity expansions. The future expansion will bring the ultimate plant capacity to 22 mgd and will require full-time operation. With extended operating hours and a two percent annual water demand growth rate, this plant expansion will not become necessary until well after the year 2020.

Table 6-1. Water Treatment Plant Capacity Evaluation

Period	Maximum Day Demand, mgd	WTP Capacity Requirement, mgd	Existing WTP Capacity, mgd	Additional WTP Capacity Required, mgd
Current	10	11	18	None
UGB Build-Out	20	22	18	4

TREATED WATER STORAGE CAPACITY

The Grants Pass water distribution system includes eight treated water reservoirs serving five separate pressure zones. As defined in Chapter 4, the treated water storage reservoirs serve three principal purposes: operational storage to meet diurnal fluctuations, emergency storage, and fire flow storage. The required storage volume for these three purposes is determined individually and then combined to identify the total amount of storage volume required within a given pressure zone and for the overall system. For added reliability, the storage should be located to allow gravity flow into the pressure zone where it is required. This arrangement eliminates the need for pumping facilities that require a backup power system during power outages. Storage located in higher pressure zones also benefits lower zones by providing a potential source of gravity supply through the addition of pressure reducing stations to the system. The following treated water storage standards were established in Chapter 4 for evaluating system capacity:

1. Operational storage equal to 45 percent of maximum day demand for current demand and part-time plant operation. Operational storage equal to 25 percent of maximum day demand for future demand and full-time plant operation.
2. Emergency storage equal to 75 percent of maximum day demand.
3. Fire flow storage based on the largest fire flow requirement in the pressure zone.

The combination of these three storage components yields the total storage required, which is equal to 1.2 times maximum day demand plus fire flow for the current condition and simply maximum day demand plus fire flow for the future condition. The fire flow volume is based on a 4,000 gpm/4 hour fire in Pressure Zones 1 and 2, a 1,500 gpm/2 hour fire in Pressure Zone 4, and a 2,000 gpm/2 hour fire in the other pressure zones.

Table 6-2 summarizes the evaluation of treated water storage requirements for current demand conditions. The existing system contains an overall treated water storage capacity of 19 million

gallons which is ample for overall current levels of demand. The significant volume of storage that is available in the system relative to maximum day demand provides ample operating storage, thus allowing the water treatment plant to effectively operate on a part-time basis without compromising emergency storage or fire flow storage supplies. Existing storage volumes are also sufficient for current demand on a zone-by-zone basis except in Pressure Zone 4 where only 80,000 gallons of storage volume is available. The existing available storage in North Valley is somewhat overstated since only approximately one third of the 1.2 million gallon storage volume available in Reservoir 15 is actually utilized due to limited demand in that portion of the system.

Table 6-2. Current Treated Water Storage Evaluation

Pressure Zone	Current Max Day Demand, mgd	Required Operational Storage, mg	Required Emergency Storage, mg	Required Fire Flow Storage, mg	Required Total Storage, mg	Existing Available Storage, mg
PZ 1	7.0	3.2	5.3	0.96	9.4	11.5
PZ 2	2.1	0.9	1.6	0.96	3.5	4.3
PZ 3	0.6	0.3	0.5	0.24	1.0	2.0
PZ 4	0.2	0.1	0.2	0.18	0.5	0.1
PZ NV	0.1	0.1	0.1	0.24	0.4	1.2
Total	10.1	4.5	7.6	2.6	14.7	19.0

Table 6-3 summarizes the treated water storage evaluation for future demand conditions. For the build-out demand condition, the water distribution system will extend to two large areas with Pressure Zone 2 elevations and one large area with Pressure Zone 4 elevations. These areas (Williams Highway and Meadow Wood areas in southern Grants Pass and Laurelridge in northwestern Grants Pass) are isolated from existing portions of the system within the same pressure zone range. All three areas will thus require dedicated treated water storage. Table 6-3 refers to these pressure zones as PZ 2WH, PZ 2MW, and PZ 4LW. As indicated in the table, Pressure Zone 4 storage will remain deficient in the future. Otherwise, the only other areas requiring additional storage to satisfy build-out development are in Pressure Zone 1. Additional Pressure Zone 1 storage is necessary due to substantial expansion of the water distribution system on the south side of the Rogue River and would be best supplied from two reservoirs. These reservoirs would be located in the southeastern and southwestern extremities of the zone. Since some of the existing storage is located in higher pressure zones where it is unavailable to the lower zones, the new treated water storage volume required to satisfy build-out demand conditions is actually greater than the 4.5 million gallon difference between required and existing storage.

Table 6-4 summarizes the recommended location and volume of future storage tank construction. These new storage tanks are located and sized to satisfy system demand through build-out. Whenever possible, the reservoir numbers used in this table are consistent with the numbering convention used in the 1979 Master Plan. Two reservoirs that were included in the previous

master plan, Reservoir No. 7 and Reservoir No. 9, are no longer recommended for construction. Reservoir No. 7 was eliminated due to scaling back of the UGB in the northeast part of town. Reservoir No. 9 was eliminated due to the presence of adequate storage capacity in the other Pressure Zone 1 reservoirs. No reservoir construction is necessary in the near term future.

The proposed Reservoir No. 12 is located on the southwest periphery of Pressure Zone 1. Due to the great distance from the water treatment plant to this reservoir, the other relatively closer Pressure Zone 1 Reservoirs will fill faster. This fill rate imbalance can be remedied by designing the future RCC pump station with the capability to both serve Pressure Zone 2 and fill Reservoir No. 12.

Table 6-3. Build-Out Treated Water Storage Evaluation

Pressure Zone	Build-Out Max Day Demand, mgd	Required Operational Storage, mg	Required Emergency Storage, mg	Required Fire Flow Storage, mg	Required Total Storage, mg	Existing Available Storage, mg
PZ 1	14.2	3.5	10.6	0.96	15.2	11.5
PZ 2	2.8	0.7	2.1	0.96	3.8	4.3
PZ 2MW	0.4	0.1	0.3	0.18	0.6	0.0
PZ 2WH	1.1	0.3	0.8	0.18	1.3	0.0
PZ 3	1.2	0.3	0.9	0.24	1.4	2.0
PZ 4	0.3	0.1	0.2	0.18	0.5	0.1
PZ 4LW	0.2	0.1	0.2	0.18	0.4	0.0
PZ NV	0.1	0.0	0.1	0.24	0.4	1.2
Total	20.4	5.1	15.3	3.1	23.5	19.0

Table 6-4. Recommended Storage Tank Construction

Reservoir	Location	Volume, mg	Construction Timing
No. 10	Highway 99 East	1.7	Post 2020
No. 12	Rogue Community College	2.0	2010-20
No. 14	Valley View West	0.4	2010-20
No. 16	Meadow Wood	0.6	2010-20
No. 17	Cathedral Hills	1.3	2010-20
No. 13 (replacement)	Sunset Lane	0.4	2010-20
Total		6.4	

BOOSTER PUMPING CAPACITY

There are ten booster pumping stations in the Grants Pass water distribution system. The booster pumping facility criteria require that pumping stations are able to supply the maximum day

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demand within all dependent pressure zones over a 24 hour period. This criterion assumes that there is sufficient treated water storage within the pressure zone to meet the operational, emergency, and fire storage criteria. The pumping station should be equipped with a backup energy source of sufficient capacity to operate the pumping plant at its rated capacity. The rated capacity of a pumping station is based on the largest pump being out of service. This criteria does not apply to the small pressure zones which are served by booster pumps with hydropneumatic tanks. For these small pressure zones, the pump station should be sized such that it can supply peak hour demand with one pump out of service. These stations should also have a redundant pump to meet fire flow requirements.

Table 6-5 summarizes the booster pumping capacity evaluation for the water distribution system with current demand levels. Current pump station capacities are adequate for meeting the appropriate performance criteria of either maximum day demand or peak hour demand. However, the Hilltop and Harbeck Heights pump stations serve isolated areas without storage, so these stations should have a redundant fire flow pump. A pipeline linking the two service areas would also provide redundancy, but yields no cost savings. The water treatment plant pumping capacity is significantly higher than the system-wide maximum day demand, allowing the pumps to operate on a part-time basis. Table 6-5 also includes the New Hope Pump Station which was recently built to serve Zone 2 developments in the southwest. Although construction is complete, the existing connections to the system and associated demand are currently limited.

Table 6-5. Current Booster Pumping Capacity Evaluation

Pump Station	Current[a] Max Day Demand, mgd	Current[a] Max Day Demand, gpm	Existing Pumping Capacity, gpm
Plant[b]	10	6,940	11,100
Lawnridge & Madrone	3.1	2,150	4,500
Champion	1	690	2,400
North Valley	0.1	70	570
Hefley[c]	0.2	140	760
Starlite[d]	0.08	60	960
Hilltop[e]	0.04	30	250
Harbeck Heights[f]	0.04	30	180
New Hope[g]	NA	NA	1,050

[a]Peak hour demand is used for those stations that serve areas without reservoirs.
[b]Noted capacity is based on plant operator measurement of the firm capacity for the plant effluent pumps. The capacity of the plant influent pumps is 9,600 gpm with one pump out of service.
[c]The Hefley station has two fire flow pumps with a total capacity of 1,200 gpm.
[d]The Starlite station has a fire flow pump with a capacity of 1,050 gpm.
[e]The Hilltop station has a fire flow pump with a capacity of 750 gpm.
[f]The Harbeck station has a fire flow pump with a capacity of 1,200 gpm.
[g]The New Hope station has a fire flow pump with a capacity of 2,000 gpm.

Table 6-6 shows the booster pumping capacity evaluation for future demand conditions. Three new pump stations will be required for the build-out system in order to serve southeastern and southwestern Pressure Zone 2 and 3 and northwestern Pressure Zone 4. Two of these stations are already under design: Meadow Wood Pump Station in the southeast and Laurelridge Pump Station in the northwest. A third pump station is necessary to serve the Rogue Community College campus in the southwest. The Meadow Wood Pump Station is being designed to serve both Pressure Zone 2 and a small area of Pressure Zone 3. Since these three pump stations are planned to be built to serve areas without adequate reservoir storage for fire flows, the recommended pump station capacities allow for the inclusion of fire flow pumps. In addition to the new pump stations, booster pumping capacity expansions will be necessary at the water treatment plant for Pressure Zone 1. Also, the Hilltop and Harbeck Heights pump stations lack redundant fire flow pumps. The Ausland Pump Station, which was identified for future construction in the 1979 master plan, is not included in this master plan due to scaling back of the UGB in the northeast part of town. The City may still maintain possession of a site for this pump station since the UGB may expand to the northeast in the future.

Table 6-6. Build-Out Booster Pumping Capacity Evaluation

Pump Station	Build-out Max Day[a] Demand, mgd	Build-out Max Day[a] Demand, gpm	Existing Pumping Capacity, gpm	Needed Capacity, gpm
Plant	20.4	14,170	11,100	3,070
Lawnridge & Madrone	4.5	3,120	4,500	NA[b]
Champion & Starlite	1.7	1,180	3,360	NA
North Valley	0.1	70	570	NA
Hefley	0.3	210	760	NA
Hilltop[a]	0.04	30	250	NA
Harbeck[a]	0.04	30	180	NA
New Hope	1.1	760	1,050	NA
Meadow Wood	0.4	280	0	280
Laurelridge	0.2	140	0	140
Rogue Community College	0.1	110	0	110

[a]Peak hour demand is used for those stations that serve areas without reservoirs.
[b]NA = Not applicable.

Based on these capacity evaluations, Table 6-7 summarizes the recommended location, capacity, and timing of pump station construction or renovation projects through build-out. Two of these pump stations, Meadow Wood and Laurelridge, are already under design. In many cases, the capacity improvements can be phased to keep pace with demand.

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Table 6-7. Recommended Pump Station Capacity Improvements

Pump Station	Capacity, gpm	Construction Timing
Treatment Plant Effluent Pumps	3,100	Post 2020
Hilltop Fire Flow Pump	1,000	2000-05
Harbeck Heights Fire Flow Pump	1,000	2000-05
Meadow Wood	1,280	2000-05
Laurelridge	1,140	2000-05
Rogue Community College	1,110	2000-05

PIPELINE NETWORK PERFORMANCE EVALUATION

The CYBERNET computer model was used to evaluate performance of the water distribution system pipeline network. Separate models were developed to evaluate the system under existing and build-out conditions. The existing system model was developed to reflect the pipeline configuration as of the spring of 2000. The build-out system model includes all of the proposed pipelines necessary for solving existing system deficiencies as well as expansion of the water distribution system to serve all developable land within the urban growth boundary. A map showing all existing and proposed pipelines in the Grants Pass water distribution system is included in Appendix 3. The specific alignments of the proposed pipelines shown on this map are tentative and flexible. The actual alignments will conform to future land use, development pattern, easement acquisition, and topographic considerations identified during the design phase of project implementation.

The distribution system was modeled under three scenarios that simulated the critical conditions that are most demanding of pipeline network performance capabilities. The three scenarios that were modeled include:

- Peak hour demand
- Maximum day demand plus fire flow
- Low demand during water treatment plant operation

Peak Hour Demand Analysis

The peak hour demand scenario was modeled as taking place during the afternoon on a summer day when the irrigation demands are high. The water treatment plant and booster pump stations in the system were operating at their rated capacity during the peak hour and storage reservoir levels were set at 80 percent full.

The service standard for minimum system pressure was used as the criteria for identifying deficiencies for the peak hour demand scenario. Table 6-8 summarizes the nodes in the system with pressures less than 35 psi during peak hour demand conditions. In each case, the pressure deficiency is caused by excessive elevation of the service connection since the location of the node is beyond recommended range for the given pressure zone. The top of the elevation range

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for Pressure Zone 1 is 1,020 feet and the top of the elevation range for Pressure Zone 2 is 1,140 feet. Table 6-8 identifies the location, pressure estimate, and recommended corrective action for each of the deficient nodes. In addition to the identified low pressures, these nodes are also problematic with respect to available fire flows. The majority of the deficiencies can be corrected by reconfiguring the pipeline network to modify the local pressure zone boundary. In some areas, these modifications to zone boundaries may require the installation of individual pressure reduction valves on some service connections. The Woodson Drive pressure deficiency is caused by an isolated hilltop which does not easily allow for reconfiguration of the pressure zone boundaries. Auxiliary pumping will solve the peak hour low pressure problems on Woodson, but fire flow availability will remain a problem.

Table 6-8. Peak Hour Demand Deficiency Summary

CYBERNET Node Label	Location	Pressure Zone	Elevation (feet)	Pressure (psi)	Recommended Corrective Action	Project Timing
J-1102	Foothill Blvd.	PZ 1	1,036	27	Reconfigure PZ boundary	2000-05
J-1142	Foothill Blvd.	PZ 1	1,026	31	Reconfigure PZ boundary	2000-05
J-1356	Madrone & Beacon	PZ 1	1,037	28	Reconfigure PZ boundary	2000-05
J-1377	Grant and E	PZ 1	1,052	24	Reconfigure PZ boundary	2000-05
J-2067	Valley View Rd	PZ 2	1,164	28	Reconfigure PZ boundary	2000-05
J-2216	Woodson Dr.	PZ 2	1,162	21	Auxiliary pumping	2000-05

Maximum Day Demand Plus Fire Flow Analysis

The maximum day demand plus fire flow scenario was modeled as taking place during the night when storage reservoir levels were depleted to 80 percent of their full capacity. Since the water treatment plant is typically not operating at night, all of the required fire flow was derived from storage sources in those zones with reservoirs. This scenario was modeled to verify the availability of minimum fire flows for residential (1,000 pgm), commercial (2,000 gpm), multi-family (2,000 gpm), and schools (4,000 gpm) at those nodes servicing these land uses.

As indicated above, minimum fire flow requirements were used to identify system deficiencies for the maximum day demand plus fire flow scenario. The available fire flow at a node is defined as the maximum flow available from a given node without reducing pressure anywhere within the same zone below 20 psi. The fire flow analysis report from the CYBERNET model was reviewed to identify any deficiencies with respect to minimum fire flow requirements. In addition to verifying the system-wide minimum fire flow of 1,000 gpm, the available fire flows were also reviewed for any violations of the 2,000 gpm minimum fire flow for all nodes serving commercial and multi-family developments and the 4,000 gpm minimum fire flow requirement

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for all nodes serving school facilities. Due to the presence of several areas in the system with insufficient looping or extensive networks of 6-inch diameter pipe, there were many nodes that did not meet minimum fire flow requirements. Table 6-9 identifies the location, available fire flow estimate, required fire flow, and recommended corrective action for each of the deficient areas. Although Table 6-9 identifies only one node for each problem area, many of these problem areas are actually indicated by a cluster of deficient nodes. The deficiencies identified in Table 6-9 provided the basis for the development of capital improvement projects for the existing system.

In order to distinguish pipeline projects in the CIP, each individual project is assigned a numerical designation such as P-101, P-102, etc. Since correction of existing deficiencies is a high priority, these pipeline projects are targeted for implementation in 2000-05.

Low Demand with Booster Pump Operation

The low demand, booster pump operation scenario simulated a time when the pumps were filling reservoirs while demand on the system was only at 20 percent of average day. Once again, the water level in reservoirs was set at 80 percent of full capacity.

The service standard for maximum system pressure was used as the criteria for identifying deficiencies for the low demand scenario. Table 6-10 summarizes the nodes in the system with pressures over 100 psi. In each case, the excessive pressure was caused by nodes with elevations below the elevation of the service connection since the location of the node is beyond recommended range of service connections for the given pressure zone. In each case, the excessive pressure is caused by the low elevation of the service connection since the location of the node is beneath the recommended range for the given pressure zone. Although Table 6-10 identifies only one node for each problem area, many of these problem areas are actually indicated by a cluster of excessive pressures.

In general, the recommended corrective action for high pressure areas is the installation of individual pressure reduction valves on service connections. However, the very high pressures in the North Valley pressure zone warrant the installation of two pressure reduction stations. Site selection for these pressure reduction stations will require a review of the topography in the North Valley and consideration of any planned expansion to the service area.

Table 6-9. Maximum Day Demand Plus Fire Flow Deficiencies

Location and Minimum Node Label	Pressure Zone	Available Fire Flow, gpm	Required Fire Flow, gpm	Recommended Corrective Action	Pipeline Project Number
Residential Land Use Areas					
W. Harbeck Rd J-1261	PZ 1	833	1,000	Provide looping from W. Harbeck Rd. to Allen Creek Rd. (600 ft, 8-inch)	P-101
Ringuette St. J-1246	PZ 1	704	1,000	Provide looping from Ringuette St. to Union Ave. (1,110 ft, 12-inch)	P-102
B St. and Grant St. J-1076	PZ 1	470	1,000	Reconfigure Pressure Zone 2 boundary	NA*
Leonard St. J-1153	PZ 1	746	1,000	Provide looping from G St. (440 ft, 8-inch)	P-103
Lower River Rd. J-1463	PZ 1	923	1,000	Provide looping along Lower River Rd. (1,660 ft, 8-inch)	P-104
Prospect Ave Looping J-2500	PZ 2A	732	1,000	Provide looping to Gilbert Way (600 ft, 8-inch)	P-105
Crescent Dr. J-2030	PZ 2	459	1,000	Provide large diameter transmission line into Crescent Dr. from Hawthorne (6,000 ft, 12-inch)	P-106
10th and Primrose Pl. J-2051	PZ 2	904	1,000	Provide looping from 9th St. (1,350 ft, 8-inch)	P-107
Tokay Heights J-2212	PZ 2	695	1,000	Provide looping from Sherman Ln. (1,960 ft, 12-inch)	P-108
Starlite Pl. J-3060	PZ 3	877	1,000	Reconfigure Pressure Zone 4 boundary	NA*
Commercial and Multi-Family Land Use Areas					
Marion Ln. J-1241	PZ 1	1,374	2,000	Provide looping to Ringuette St. on Redwood Hwy. (1,280 ft, 8-inch)	P-109
C St. J-1133	PZ 1	1,281	2,000	Provide looping from D St. (550 ft, 8-inch)	P-110
Schools					
Fruitdale Elementary J-1334	PZ 1	3,859	4,000	Hamilton Lane looping (1,030 ft, 12-inch)	P-111
Riverside Elementary J-1366	PZ 1	2,467	4,000	Agness to Gladiola connection (2,420 ft, 12-inch)	P-112
Parkside Elementary J-1179	PZ 1	2,201	4,000	Lincoln Rd. looping and Bridge to Brownell Connector (4,430 ft, 12-inch)	P-114
Lincoln Elementary J-2034	PZ 2	2,927	4,000	10th and Savage St. tie-in (60 ft, 12-inch)	P-114

* NA = Not Applicable

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Table 6-10. Excessive Pressures under Low Demand with Booster Pump Operation

CYBERNET Node Label	Location	Pressure Zone	Elevation (feet)	Pressure (psi)	Recommended Corrective Action
J-2033	10th St. and Churchill St.	PZ 2	997	109	Individual pressure reduction valves
J-2129	A St. and Piedmont Ave.	PZ 2	998	119	Individual pressure reduction valves
J-3006	Buddy Lane and Hawthorne	PZ 3	1,081	126	Individual pressure reduction valves
J-3061	7th St and Morgan	PZ 3	1,115	116	Individual pressure reduction valves
J-3066	Cook Ave and Highland	PZ 3	1,105	112	Individual pressure reduction valves
J-4019	Beacon Dr. and Roseana Dr.	PZ 4	1,197	139	Individual pressure reduction valves
J-5018	Merlin	North Valley	995	174	Pressure reduction valves for the pressure zone

Pipeline Network Expansion

In addition to the improvement projects designed for correcting deficiencies in the existing system, the pipeline network will also require expansion into areas within the UGB that are not yet served by the water distribution system. In order to plan for these future expansions of the pipeline network, the CYBERNET model was used to size the main transmission lines that will be required to extend water service throughout the UGB. The majority of the pipeline network expansion projects are located to the south of the Rogue River, although there are also a few expansion projects in northern Grants Pass. In a few instances, the expansion projects required additional improvements within the existing system. Table 6-11 summarizes the location, size, and length of pipelines associated with each expansion project. As with the previous pipeline projects, these system expansions are assigned a numerical designation for the CIP such as P-201, P-202, etc.. This summary includes the main transmission pipelines only and not the smaller distribution pipelines. Many smaller distribution pipes were modeled, however, to ensure adequate service in peripheral areas.

Many of the pipeline extensions will allow the integration of existing private water systems into the City's water distribution network. In general, the incorporation of private systems does not present any problems. However, the Williams Crossing private system in southern Pressure Zone 2 includes some pipelines that extend beyond the upper elevation limit for that zone. House connections at these higher elevations will likely require auxiliary pumping.

Table 6-11. Summary of Pipeline Network Expansion Projects for UGB Build-Out

Expansion Project Location	Pipeline Project Number	Developer Participation	Pipe Size, inches	Pipe Length, feet	Project Timing
Redwood Avenue Extension	P-201	No	16	2,660	2000-05
Redwood Avenue Looping North	P-202	Yes	12	12,250	2000-05
Redwood Avenue Looping South	P-203	Yes	12 8	8,480 1,250	2000-05
Rogue Community College Extension	P-204	No	16 12	2,310 2,200	2000-05
Allen Creek Connector	P-205	No	20	5,300	2000-05
Reservoir No.12 Extension	P-206	No	20	2,920	2010-20
Williams Highway Extension	P-207	Yes	12	5,830	2000-05
Williams Highway Looping	P-208	Yes	12	11,270	2005-10
Reservoir No. 17 Extension	P-209	No	16	700	2010-20
West Harbeck Road Connector	P-210	No	12	2,750	2000-05
G.I. Lane Extension	P-211	Yes	8	1,540	2000-05
Pedestrian Bridge Connector	P-212	No	12	3,050	2000-05
Canal Lane Looping	P-213	Yes	8	880	2005-10
Rogue River Highway Extension:	P-214	No	12 8	4,690 1,240	2005-10
Fruitdale Drive Extension	P-215	No	12	5,770	2010-20
Cloverlawn Loop	P-216	Yes	12 8	2,130 5,510	2010-20
Reservoir No. 10 Extension	P-217	No	20	1,120	Post 2020
Cloverlawn to Crestview Loop Extension:	P-218	Yes	12 8	5,460 1,920	2010-20
Reservoir No. 16 Extension	P-219	No	16	1,300	2010-20
Southeast N St. Extension	P-220	Yes	12	4,000	2000-05
Shannon Ln Extension	P-221	Yes	12	1,840	2000-05
Lincoln Road Extension	P-222	Yes	12	5,190	2000-05
Ament Road Extension	P-223	No	12	5,690	2000-05
Starlite Connector	P-224	No	12	2,500	2000-05
Starlite Extension	P-225	Yes	12	3,830	2005-10
Greenfield Road Loop	P-226	Yes	12	9,170	2010-20
Valley View Rd	P-227	Yes	12	940	2000-05
Laurelridge Development	P-228	Yes	8	4,460	2000-05
Reservoir No. 14 Extension	P-229	No	16	490	2010-20

The extension of a pipeline from Valley View Dr. to Starlite (P-224) will complete connection of Pressure Zone 3 areas in northern Grants Pass. With this connection, the City may consider taking the Starlite pump station out of service and relying on the Champion pump station to serve all of Pressure Zone 3.

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Developers in Grants Pass are required to cover the basic costs for extending the water system to their property. Therefore, many of the system expansion projects will require developer participation. This participation typically involves financial contributions to cover the cost of an eight-inch distribution pipeline. In the event that a larger diameter pipeline is necessary for purposes of allowing water transmission to more distant parts of the system, the City will pay the difference between the larger diameter and an eight-inch line. For each pipeline network expansion project identified in Table 6-11, the participation of developers in pipeline financing is noted.

Although pipeline expansions into areas outside of the existing UGB are not formally considered in this plan, the model was used to confirm the potential of serving adjacent portions of the Redwood District, Allen Creek, and Spalding areas that are considered candidates for future UGB expansions. The pipeline network within the existing UGB is adequately sized to allow significant expansion into these adjacent areas.

Pipeline Replacement

The Grants Pass water distribution system contains approximately 28,000 feet of two-inch pipe and 10,000 feet of four-inch pipe. These small, old lines tend to create problems with service pressures and fire flows in some areas due to excessive head losses. Therefore, the City should maintain an ongoing program of annual pipeline replacement to improve service in these areas. The small diameter lines should be targeted for replacement within the 20 year planning period of this study. Since many small diameter pipelines will be replaced as part of projects designed to solve existing system deficiencies, the replacement program included in the CIP will not formally begin until the year 2005. At that time, the CIP will budget sufficient funds to replace 2,500 feet of small diameter pipeline per year.

CHAPTER 7

COST OF RECOMMENDED CAPITAL IMPROVEMENT PROGRAM

Based on the evaluation of existing system performance and future expansion requirements presented in Chapter 6, this chapter integrates the projects into a staged Capital Improvement Program (CIP). Specific improvements are identified for each of the first five years of the CIP and for the full build-out of the City's service area. For each of the recommended projects, this chapter also presents cost estimates based on the unit construction costs identified below.

UNIT CONSTRUCTION COSTS

This section presents the basis for cost estimates including the unit construction costs that are used to estimate the cost of recommended water system facility improvements. The unit costs are for construction only and do not include cost estimates for land acquisition, contingencies, engineering, legal costs, environmental, inspections and/or contract administration. These additional costs are added to the unit construction costs as a percentage mark-up to provide the City with a budgetary estimate of the total project capital cost for each proposed improvement.

These unit costs are representative of the construction cost of water system facilities under normal construction conditions. Estimation of construction costs for facilities to be constructed in areas requiring rock excavation, special foundation considerations or other special conditions should be developed based on specific unit cost data matching the expected conditions or requirements.

The unit costs were developed based on cost data supplied by manufacturers, published cost data and cost information from previous projects. All construction costs have been adjusted to reflect a year 2000 price level using the Construction Cost Index published by the Engineering News-Record (ENR). The estimated ENR index for September 2000 is 6,300. These cost data can be adjusted to reflect past or future construction costs by multiplying by the ratio of the past/future ENR to the base ENR of 6,300.

Construction cost data presented in this chapter are not intended to represent the lowest prices in the industry for each type of construction, but rather to be representative of average or typical construction costs. They are order-of-magnitude cost estimates with an expected accuracy of +50 percent to –30 percent to reflect the variability of costs according to the time of year that the work is bid, level of competition within the local economic environment, and future changes in the cost of labor and materials. As planning level unit costs, they are intended for guidance in evaluating various options and for budgetary purposes only within the context of this master planning effort.

Pipelines

Unit costs for cement-mortar lined ductile iron water mains 6- through 36-inches in diameter are provided in Table 7-1. These costs include pipe materials, trenching, placing and jointing pipe, valves, thrust blocks, fittings, hydrants, placing imported pipe bedding, and native backfill material. The table includes costs for both new construction and pipe installations requiring asphalt pavement replacement. Asphalt replacement will be included in all cost estimates unless the proposed pipeline alignment will clearly cross unpaved ground. The cost for new or reconnected service lines from the pipeline to the property line, for average suburban residential development densities, is included in the costs for pipelines up to 18-inches in diameter. Service connection costs are not included for larger diameter pipelines which serve primarily as transmission lines. Unit costs increase if rock excavation is necessary, typically adding 50 to 200 percent or more to the baseline installation cost. The cost for any proposed pipelines within known rocky areas will be increased by a factor of 2.

Table 7-1. Construction Costs for Water Mains

Pipeline Diameter, inches	Construction Cost[a,b], Dollars Per Lineal Foot New Construction	Construction Cost Including Pavement Replacement
8	49	62
10	56	69
12	69	80
14	72	89
16	76	96
18	80	104
20	89	113
24	106	126
30	138	173
36	177	217

[a]Costs based on the 20-city average ENR CCI of 6,300.
[b]Costs do not include engineering, overhead, and contingency.

Treated Water Storage Tanks

Unit costs were developed for the construction of water storage tanks in the size ranges of 0.5, 1, 2, and 5 million gallons (MG). Table 7-2 lists the estimated construction cost for a ground-level pre-stressed concrete storage tank. As previously stated, these costs represent construction under normal excavation and foundation conditions and would be significantly higher for special or difficult foundation requirements. In addition to these construction costs, an allowance of $100,000 is included for land acquisition at each tank site.

Table 7-2. Construction Costs for Storage Reservoirs

Tank Volume, million gallons[a]	Construction Cost[b] Concrete Reservoirs, $1,000[c,d]
0.5	525
1	719
2	1,080
5	2,048

[a]Costs for volumes not shown may be interpolated.
[b]Costs do not include engineering, overhead, and contingency.
[c]Costs based on the 20-city average ENR CCI of 6,300.
[d]Costs are based on prestressed concrete tank design.

Distribution Pumping Plants

Distribution pumping station costs vary considerably, depending on such factors as architectural design, pumping head, and station capacity. Estimated average construction costs for distribution pumping stations, as shown in Table 7-3, are based on enclosed stations with architectural and landscaping treatment suitable for residential areas. Pumping station cost estimates include allowances for a standby pumping unit and auxiliary engine-generator sets.

Table 7-3. Construction Costs for Distribution Pumping Stations

Pumping Station Capacity, mgd[a]	Construction Cost[b], $1,000[c]
1	150
2	181
5	263
10	368

[a]Costs for capacities not shown may be interpolated.
[b]Costs do not include engineering, overhead, and contingency.
[c]Costs based on the 20-city average ENR CCI of 6,300.

Engineering Markups and Contingencies

Engineering services associated with new facilities include preliminary investigations and reports, site and route surveys, foundation explorations, preparation of drawings and specifications, construction services, surveying and staking, sampling of testing material, and start-up services. For this study, engineering costs are assumed to be 10 percent of the construction cost estimates after construction contingencies have been applied.

There are also program implementation costs which cover such items as legal fees, financing expenses, administrative costs, and interest during construction. The cost of these items can also

vary, but for the purpose of this study, it is assumed that these charges will equal approximately 5 percent of the construction costs after construction contingencies have been applied.

Construction Management covers such items as contract management and inspection during construction. The cost of these items can also vary, but for the purpose of this study, it is assumed that construction management charges will equal approximately 10 percent of the construction costs after construction contingencies have been applied.

It is also appropriate to allow for uncertainties unavoidably associated with the preliminary layout of projects. Such factors as unexpected construction conditions, the need for unforeseen mechanical items, and variations in final quantities are a few of the items that can increase project costs for which it is wise to make allowances in preliminary estimates. An allowance of 20 percent of the base construction cost was included to cover such contingencies.

Based on these factors, the total cost of all necessary engineering services, construction management, contingencies and program implementation is 45 percent of the base construction costs for each project. This factor is applied to the construction cost to estimate the total capital cost.

COST OF PROPOSED IMPROVEMENTS

Based on the unit costs and methodology described above, this section summarizes the estimated costs associated with each CIP project. The projects are divided into four categories according to their anticipated timing:

- Year 2000 to 2005
- Year 2005 to 2010
- Year 2010 to 2020
- Post 2020

The post 2020 CIP projects represent the long term phase of projects that will take place between the year 2020 and full service area build-out. All of the CIP projects are displayed along with the existing water distribution system on the system map enclosed in Appendix 3.

Summary of CIP Projects

Tables 7-4 through 7-7 summarize the capital costs for each project in the set of recommended improvements according to their anticipated timing. These CIP projects are intended to satisfy the system improvement and expansion requirements through build-out of the City's urban growth boundary. After the first improvement period, an allowance is also included for the annual replacement of 2,500 feet of small pipelines. The reported capital costs account for developer participation in the financing of some system expansions. The estimated capital costs for pipelines are shown in greater detail in Appendix 4.

Table 7-4. Estimated Capital Costs for CIP Projects
Period 2000 – 2005

Recommended Improvements	Capital Cost, $1,000
Pump Stations	
Hilltop/Harbeck Heights Fire Pumps	60
Meadow Wood Pump Station	255
Laurelridge Pump Station	245
Rogue Community College Pump Sta.	245
Subtotal	805
Pipelines	
Pressure zone boundary modifications	100
Pressure reduction valves	130
P-101 West Harbeck to Allen Creek	54
P-102 Ringuette St	129
P-103 Leonard St Looping	40
P-104 Lower River Road	149
P-105 Prospect Ave Looping	54
P-106 Hawthorne to Crescent	748
P-107 9th to 10th at Midland	122
P-108 Sherman Ln to Tokay Heights	227
P-109 Marion Ln	115
P-110 C St to D St Loop	49
P-111 Hamilton Ln Looping	134
P-112 Agness to Gladiola	119
P-113 Bridge to Brownell	431
P-114 Lincoln Rd Looping	83
P-115 10th and Savage Tie-In	7
P-201 Redwood Ave Extension	370
P-202 Redwood Ave Looping North	319
P-203 Redwood Ave Looping South	221
P-204 Rogue Community College Ext	577
P-205 Allen Creek Connector	869
P-207 Williams Hwy Extension	152
P-210 West Harbeck Rd Connector	318
P-212 Pedestrian Bridge Connector	353
P-220 Southeast N St Extension	113
P-221 Shannon Ln Extension	48
P-222 Lincoln Rd Extension	103
P-223 Ament Rd Extension	660
P-224 Starlite Connector	290
P-227 Valley View Rd	25
Subtotal	7,109
Total	7,914

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Table 7-5. Estimated Capital Costs for CIP Projects
Period 2005 – 2010

Recommended Improvements	Capital Cost, $1,000
Pipelines	
P-208 Williams Hwy Looping	294
P-214 Rogue River Hwy Extension	655
P-225 Starlite Extension	100
Subtotal	1,049
Pipeline Replacement	
12,500 feet total replacement	1,124
Subtotal	1,124
Total	2,173

Table 7-6. Estimated Capital Costs for CIP Projects
Period 2010 - 2020

Recommended Improvements	Capital Cost, $1,000
Treated Water Storage	
Reservoir No. 12	1,710
Reservoir No. 14	850
Reservoir No. 16	960
Reservoir No. 17	1,350
Reservoir No. 13 (replacement)	850
Subtotal	5,720
Pipelines	
P-206 Reservoir No. 12 Extension	479
P-209 Reservoir No. 17 Extension	97
P-215 Fruitdale Dr Extension	780
P-216 Cloverlawn Loop	56
P-218 Cloverlawn to Crestview Loop	142
P-219 Reservoir No. 16 Extension	181
P-226 Greenfield Rd Loop	239
P-229 Reservoir No. 14 Extension	68
Subtotal	2,042
Pipeline Replacement	
25,000 feet total replacement	2,248
Subtotal	2,248
Total	10,010

Table 7-7. Estimated Capital Costs for CIP Projects
Period Post 2020

Recommended Improvements	Capital Cost, $1,000
Treated Water Storage	
Reservoir No. 10	1,560
Subtotal	1,560
Pump Stations	
Treatment Plant Pumps	400
Subtotal	400
Pipelines	
P-217 Reservoir No. 10 Extension	184
Subtotal	184
Total	2,144

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CITY OF GRANTS PASS

WATER MANAGEMENT PLAN

FINAL REPORT

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June 2002

EXHIBIT 5
to UAPC Staff Report 191

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CHAPTER 1

EXECUTIVE SUMMARY

The purpose of the Water Management Plan is to identify and analyze the water supply and demand issues facing the City of Grants Pass, develop a reasonable approach to resolving the issues, and serve as a guide for City water management policies. This plan was developed in accordance with Oregon Water Resources Department guidelines and contains a comprehensive discussion of the existing water distribution system, current and future development of water demand, existing and potential water conservation measures, water curtailment strategies, implementation schedules, and long range water supply issues.

EXISTING WATER SYSTEM

The City of Grants Pass gets its drinking water from the Rogue River. The Rogue River has historically provided a plentiful supply for the City and even at its lowest flows has sufficient flow for the City's current demand. Grants Pass holds four water rights on the river, totaling 87.5 cfs (56 mgd). One right for 12.5 cfs (8 mgd) is perfected.

The City's water system consists of a water treatment plant, eleven booster pumping stations, eight reservoirs, and an extensive water distribution system with over 130 miles of pipeline. Due to the extent of the distribution system and the highly varied local topography, the service area contains seven separate pressure zones. The overall water system is limited by the capacity of the water treatment plant, which is rated at 18 mgd.

WATER DEMAND

The water system serves the residents of Grants Pass with a current population of 23,170 and surrounding developing areas. Table 1-1 shows current water demand.

Table 1-1. Current Water Demand

Condition	Current Water Demand, mgd
Average Annual	4.5
Maximum Month	8
Maximum Day	10

A comparison of water sold to water produced shows that the system has an excellent delivery record. The unaccounted-for water rate was limited to 10.9 percent in calendar year 2000. A breakdown of water demand by customer categories is shown in Table 1-2.

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Table 1-2. Water Demand by Customer Category

Customer Category	Percentage
Commercial	36
Multi-Family	16
Single Family	48
Total	100

WATER CONSERVATION MEASURES

The State requires in a Water Management Plan that the City examine the feasibility of six types of water conservation measures (WCMs 1-6) and that the City provide an implementation schedule for an additional six types of measures (WCMs 7-12). Grants Pass has a number of conservation programs in place and an analysis of potential additional conservation programs was performed. A list of measures was screened based on a list of criteria including projected water savings, cost, political feasibility and legal constraints, consistency with community values, and environmental impacts. From this process, six favored programs emerged and a detailed cost-benefit analysis was performed. Of these programs, five were recommended for implementation. Table 1-3 summarizes existing and recommended conservation programs.

Table 1-3. Grants Pass Water Conservation Measures

WCM	Program Description	Status
1	Leak reporting program	Existing
2	Low water use demonstration garden	Recommended (2002)
3	SDCs based on meter size Separate indoor and outdoor metering	Existing Existing
4	Enforces state building code regulations Distribute plumbing kits	Existing Recommended (2003)
5	Inverted block water rates	Recommended (2001)
6	Non potable water used at WWTP	Existing
7	Annual system accounting	Existing
8	Fully metered system	Existing
9	Visual leak inspection Customer tracking to spot leaks	Existing Existing
10	Random meter testing Customer tracking to spot dead meters Commercial meter replacement program	Existing Existing Existing
11	Pamphlet distribution, city newsletter Low water use demonstration garden	Existing Recommended (2002)
12	Bulk water dispensing station and fire hydrant flow meters Additional bulk water dispensing stations Water waste prohibition	Existing Recommended (2001) Recommended (2001)

WATER CURTAILMENT MEASURES

The City has not experienced any supply deficiencies within the last 10 years, but the potential exists for service interruption in the event of a supply contamination, treatment plant difficulties, transmission or pumping problems or prolonged drought. The City has in place a water curtailment ordinance; however, in compliance with OAR 690-086, the ordinance will be repealed and replaced with an ordinance that clearly defines three levels of water shortage alerts and contains specific water curtailment measures to be implemented at each level of alert. The citizens of Grants Pass will be well served to have in place a curtailment plan that defines levels of water shortage severity and mechanisms for dealing with the situation. Table 1-4 lists water shortage alert levels, operational triggers, and curtailment measure implementation requirements. Chapter 5 outlines specific curtailment measures for each alert level.

Table 1-4. Water Shortage Alert Levels, Triggers, and Curtailment Measure Implementation Requirements

Alert Level	Description	Trigger	Curtailment Measure Implementation
One	Potential Water Supply Shortage	A serious drought condition is occurring or is likely to occur in the region or Rogue River flow rates are measured or projected to be below a 1-in-10 year low flow level, or the County or State has declared a drought condition.	Voluntary
Two	Water Supply Shortage	The City's ability to deliver water is not adequate to meet demand due to supply, treatment, storage, or pumping restrictions, or extended treatment plant operation is required and storage cannot be maintained.	Mandatory
Three	Critical Water Supply Shortage	Supply is interrupted	Mandatory

LONG RANGE SUPPLY

Future demand requirements were developed using land use demand factors and zoning information. The rate of development for the area was estimated to continue at 2.8 percent. Peaking factors were developed using historical water use data. Resulting future water demand requirements are shown in Table 1-5.

000196

Table 1-5. Future Water Demand Requirements

Year	Average Demand, mgd	Maximum Day Demand, mgd
2010	6.1	13
2020	8.1	18
Build-out	9.5	21

Generally, the Rogue River provides an ample and reliable water supply for future water needs as Grants Pass expands within the urban growth boundary. It is important to note, however, that there are special factors such as the listing of salmon as an endangered species and long-term climate change that may impact the future ability of the river to maintain its reliable yield. These factors bear watching but at this time are ill defined, so it is difficult to quantify their potential effect.

CHAPTER 2

EXISTING WATER SYSTEM

SOURCE OF WATER

Source Availability and Reliability

The source of supply for the City of Grants Pass is surface water from the Rogue River. The Rogue River drains a large watershed extending from the Pacific Ocean to the crest of the Cascade Mountains. Grants Pass is located at approximately River Mile 100 and there are approximately 2,460 square miles of watershed area upstream of the City. As a result of this extensive drainage area, the Rogue River is a plentiful and reliable source of drinking water for the community.

The U.S. Geological Survey (USGS) maintains a river gaging station near the Grants Pass water treatment plant that provides extensive historical data on the flow characteristics of the Rogue River. Since the Lost Creek Reservoir storage reservoir was constructed upstream of Grants Pass in 1977 to regulate flow, USGS statistical data for the river are typically based on records from 1978 to present. Based on USGS data for this station, Table 2-1 presents the average, maximum, and minimum monthly flow rates for the Rogue River. Since construction of Lost Creek Reservoir, the lowest daily average flow at Grants Pass was 744 cubic feet per second (cfs) on October 10, 1994, and the lowest seven-day average flow was 799 cfs during the week of September 22, 1994. In general, dry weather flows are maintained by the combination of snow melt from the Cascades in the early summer and the release of stored water from Lost Creek Reservoir in the late summer.

Table 2-1. Rogue River Average Monthly Flows at Grants Pass
USGS Data for the 20-Year Period 1978 to 1997

Month	Average Monthly Flow, cfs	Maximum Monthly Flow, cfs	Minimum Monthly Flow, cfs
January	4,684	16,600	1,575
February	4,556	10,960	1,641
March	4,034	8,119	1,099
April	4,002	6,843	1,211
May	3,607	5,910	1,857
June	2,709	4,572	1,549
July	2,146	3,127	1,059
August	2,164	3,080	1,620
September	1,840	2,642	1,333
October	1,499	2,282	1,008
November	2,670	7,669	1,160
December	5,251	17,620	1,557

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Long-Term Reliable Yield

Due to the nature of the City's surface water supply source, the long-term sustainability of drinking water supplies for Grants Pass is generally good. As noted earlier, the large size of the watershed drained by the Rogue River typically provides abundant water supplies throughout the year. Even during extreme dry weather periods when river flows are at their lowest, the reliable flow rate in the Rogue River is approximately 750 cfs or nearly fifty times larger than the highest drinking water demand ever experienced in Grants Pass. There are some special circumstances, which may affect the long-term reliable yield for the Rogue River. For example, the listing of the Coho Salmon as an endangered species in the Rogue River may influence operational procedures at the Lost Creek Reservoir, which in turn may affect dry weather flow levels. Another issue is related to climate change and snow pack levels in the Cascade Range. Any reduction in average precipitation or the average snow pack will tend to reduce dry weather flow rates in the Rogue River. Since these factors are complex in nature, it is difficult to quantify their potential effect on the river's reliable yield at this time.

Water Rights

The City of Grants Pass has water rights for the withdrawal of 87.5 cfs from the Rogue River. Table 2-2 summarizes the details related to these water rights. Documentation for the water rights is included in Appendix A.

Table 2-2. Grants Pass Water Rights

Permit Number	Priority Date	Permitted Use	Permitted Rate, cfs	Available Quantity, cfs	Source Availability Analysis		
					Reliability	Impact of ESA	Water Quality
D15839	1888	Municipal/ Irrigation	12.5		High	Undefined	Good
S26901	1960	Municipal	25.0	735[a]	High	Undefined	Good
S45827	1965	Municipal	25.0		High	Undefined	Good
S47346	1983	Municipal	25.0		High	Undefined	Good

[a]Restriction that water can be diverted only when flow at the mouth of the Rogue exceeds 735 cfs.

Intergovernmental Agreements

The City has no system interties that would provide additional water supply.

SYSTEM CAPACITY, LIMITATIONS, AND OPPORTUNITIES FOR EXPANSION UNDER EXISTING WATER RIGHTS

The Grants Pass water supply system currently distributes water to developed properties covering an area of more than 3,500 acres and serves a population of 23,170 within the City limits and several hundred outside the City limits in Harbeck-Fruitdale, Redwood, and North Valley. The overall system is composed of a water treatment plant, twelve booster pumping stations, eight reservoirs, two pressure reducing valves, and six altitude valves. Figure 2-1

illustrates the configuration of the Grants Pass water distribution system. The figure depicts all water distribution piping twelve inches in diameter and larger and shows future piping improvements recommended in the 2000 Water Master Plan including extension of service to the urban growth boundary and pipeline looping.

Water Treatment Plant

The City draws water from the Rogue River with a pumping station located next to the water treatment plant. The treatment plant was originally constructed in 1930 and has undergone many renovations over the years. The most recent plant expansion was completed in 1983, bringing the total rated plant capacity to 18 million gallons per day (mgd). Influent pumps deliver river water to the plant where the treatment process includes coagulation and sedimentation of suspended solids, filtration, and chlorination for disinfection prior to pumping into the distribution system.

Distribution Pipeline Network

The Grants Pass water distribution pipeline network consists of approximately 130 miles of existing pipeline. Table 2-3 details the water distribution system according to pipeline length and diameter. These pipelines are made of cast iron or ductile iron and range in age up to approximately 80 years.

Table 2-3. Water Distribution System Pipeline Network

Pipe Size, inches	Length, miles
2	5.23
4	1.80
6	40.89
8	43.63
10	7.64
12	19.49
14	0.38
16	7.88
20	2.40
24	1.02
30	0.95
36	0.01
Total	131.32

The urban growth boundary for the City of Grants Pass encompasses lands of wide ranging elevations. As a result, the water distribution pipeline network contains seven separate service pressure zones. Table 2-4 summarizes the service elevations and static pressure range for each pressure zone. The lower end of the pressure range is based on reservoirs at 80 percent full and the upper end is based on full reservoirs. At this time, there are properties receiving City water service in each of the pressure zones except Zone 5.

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Table 2-4. Pressure Zone Ranges

Zone	Elevation, feet	Pressure, psi
1	900 – 1,020	36 – 90
2	1,020 – 1,140	41 – 95
2A	960 – 1,035	61 – 94
3	1,140 – 1,280	36 – 100
4	1,280 – 1,420	42 – 104
5	1,420 – 1,560	41 – 104
NV	995 – 1,165	101 – 177

In some areas, the pressure zone boundaries are modified slightly from these elevation ranges in order to accommodate special service pressure requirements. Pressure Zone 2A is a hybrid between Zones 1 and 2. The North Valley service area is actually a hybrid between Zones 1, 2, and 3, serving properties between the elevations of 995 feet and 1,165 feet. Due to the great range of elevations served in the North Valley, this pressure zone requires pressure reduction valves at service connections to maintain appropriate service pressures.

Storage Reservoirs

There are eight treated water storage reservoirs within the Grants Pass water distribution system that provide a total of 19 million gallons of treated water storage. These reservoirs were constructed between the years 1946 and 1999. Design information for these reservoirs is detailed in Table 2-5.

Table 2-5. Storage Reservoirs

Reservoir Location	Reservoir Number	Pressure Zone Served	Year Built	Construction Materials	Capacity, Mg	Bottom Elevation, feet	Overflow Elevation, feet
500 Block Woodson Dr.	3	1	1946	Concrete	3.5	1,089.5	1,108.5
1500 Block Ridge Rd.	4	2	1953	Concrete	0.75	1,216	1,240
1400 Block Sherman Ln.	5	1	1983	Concrete	3.5	1,079.5	1,108.5
2200 Block Crown St.	6	2	1982	Concrete	3.5	1,211	1,240
Heiglen Loop Rd.	8	3	1983	Concrete	2.0	1,341	1,370
1420 Denton Trail	11	1	1999	Concrete	4.5	1,080.1	1,108.5
1700 Block Sunset Ln.	13	4	1980	Concrete	0.08	1,510	1,520
3900 Block Highland Ave.	15	5	1985	Concrete	1.2	1,374	1,403

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Booster Pumping Stations

The water distribution system includes the water treatment plant pumps and nine booster pumping stations that transfer water to the higher pressure zones. These pump stations either fill the reservoirs that serve these higher pressure zones or pump to maintain a minimum pressure in those areas that are not served by reservoirs. Table 2-6 details the technical information for each of the system's pumping stations.

WATER SYSTEM OPERATION

Water Treatment Plant

The water treatment plant operates as necessary to fill storage reservoirs in the distribution system on a daily basis. Therefore, the operating schedule varies with seasonal variations in water demand. During the winter months, the plant generally operates seven days per week for an eight hour period. Operational hours are extended during the high demand summer months, when the plant must operate up to twelve hours daily in order to keep the storage reservoirs full.

Booster Pumping Stations Serving Areas With Reservoirs

Those booster pumping stations that fill storage reservoirs are automatically controlled to maintain preset water levels. When sensors show that the water level in a reservoir has fallen below a preset threshold, the lead pump will activate and begin filling the reservoir to a high water level. If water demand on the reservoir is such that a single pump cannot maintain the water level, a lag pump (or pumps) will activate as necessary until the reservoir fills to a high water level.

Booster Pumping Stations Serving Areas Without Reservoirs

Booster pumping stations that serve areas without storage reservoirs are automatically controlled to maintain a minimum discharge pressure at the pumping stations. When pressure sensors show that the discharge pressure has fallen below a preset threshold, the lead pump activates and pumps until the discharge pressure exceeds a high pressure level. If water demand in the pump station's service area is such that a single pump cannot maintain the pressure level, a lag pump (or pumps) activates until the system pressure is restored.

Reservoirs

Reservoirs in the water distribution system are generally maintained between 80 and 100 percent full. This fluctuating volume represents the operating storage. The remaining storage is allocated to providing fire flow requirements and emergency reserves. In the case of Reservoir No. 15 in the North Valley, water levels are maintained at a much lower level due to limited demand in that portion of the distribution system.

Altitude valves control the flow into and out of Reservoirs Nos. 3, 4, 5, 6, 11, and 15. These valves are designed to close when the reservoir is full and open when the system pressure drops. The other reservoirs in the distribution system float on the system.

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Table 2-6. Existing Booster Pumping Stations

Pumping Station Name	Pressure Zone Served	Reservoirs Served	Number of Pumps	Pump Motor Size and Speed, hp/rpm	Capacity of Each Pump, gpm	Rated Discharge Head, feet
Treatment Plant	1	No. 3	5	300/1,775	3,500	220
		No. 5		300/1,775	3,500	220
		No. 11		250/1,760	3,500	210
				250/1,760	3,500	210
				200/1,750	2,600	210
Lawnridge	2	No. 6	4	25/1,750	400	120
		No. 4		50/1,750	1,000	120
				50/1,750	1,000	120
				100/1,750	2,000	148
Madrone	2	No. 4	3	60/1,750	2,000	170
		No. 6		401,750	1,200	170
				301,750	900	170
New Hope	2	--	4	30/3,600	350	212
				30/3,600	350	212
				30/3,600	350	212
				150/1,800	2,000	200
Meadow Wood	2	--	4	5/3,500	50	240
				15/3,600	150	155
				60/3,600	500	275
				60/3,600	500	275
Harbeck Heights	2	--	3	5/3,600	90	100
				5/3,600	90	100
				50/3,600	1,200	125
Hilltop	2	--	3	5/3,600	100	120
				7.5/3,600	150	120
				40/3,600	750	120
Champion	3	No. 8	3	50/1,750	800	165
				150/1,750	2,300	165
				100/1,750	1,600	165
Starlite	3	--	4	15/3,500	60	185
				30/1,760	450	185
				60/1,760	1,050	185
				30/1,760	450	185
Hefley	4	No. 13	4	7.5/3,500	40	250
				15/3,500	120	250
				60/3,500	600	300
				60/3,500	600	300
Laurelridge	4	--	3	15/3,500	300	150
				15/3,500	300	150
				75/3,500	1,000	162
North Valley	NV	No. 15	3	7.5/3,500	70	170
				30/3,500	500	174
				30/3,500	500	174

Pressure Reducing Valves

There are two pressure reducing valve stations in the Grants Pass water distribution system. These stations control the flow of water from Pressure Zone 2 to Pressure Zone 2A. Pressure Zone 2A extends to slightly lower elevations than Pressure Zone 2 and thus requires some pressure reduction. Each station contains a single 6-inch pressure reducing valve

Supervisory Control and Data Acquisition (SCADA) System

The City upgraded the water distribution system SCADA system in 1999. The SCADA system monitors reservoir levels, pump operating status, and local pressures throughout the system. The central computer system for the human-machine interface is located at the water treatment plant.

SYSTEM CAPACITY LIMITATIONS

The capacity of the Grants Pass water system is dependent on three components: the supply source, permitted water rights, and the water supply infrastructure. The limitations of each of these components are discussed in the following sections.

Rogue River Supply Capacity

As noted earlier, the large size of the watershed drained by the Rogue River typically provides abundant water supplies throughout the year. Even during extreme dry weather periods when river flows are at their lowest, the reliable flow rate in the Rogue River far exceeds present and projected future water demands. Identified, but undefined factors such as the listing of Coho Salmon under the Endangered Species Act may impact the amount of water available to the City in the future.

Permitted Water Rights

The water rights currently held by the City are sufficient for present and future water needs. The City holds a perfected water right of 12.5 cfs and three water right permits totaling 87.5 cfs.

Water Treatment Plant Capacity

The Grants Pass water treatment plant has a rated water treatment capacity of 18 mgd. However, the capacity of the plant is currently limited to 16 mgd by its firm pumping capacity. This limit is only relevant in the event one of the influent pumps is out of service. The available capacity is sufficient to meet current maximum day demand with at least an additional ten percent capacity available. The additional ten percent is necessary to allow for backwashing filters, meeting drinking water quality standards with difficult raw water, or repairing equipment failures.

The existing water treatment plant has ample capacity relative to current demand and will continue to be sufficient for the near term future. This situation allows the City to operate the water plant on a part-time basis even during the current water demand peaks in the summer. An extension of the plant operating hours will forestall the need for treatment capacity expansions. With extended operating hours and a two percent annual water demand growth rate, a plant

000204

expansion will not become necessary until well after the year 2020. Table 2-7 summarizes the treatment plant capacity evaluation for current and build-out demand conditions.

Table 2-7. Water Treatment Plant Capacity Evaluation

Period	Maximum Day Demand, mgd	WTP Capacity Requirement, mgd	Existing WTP Capacity, mgd	Additional WTP Capacity Required, mgd
Current	10	11	18	None
UGB Build-Out	20	22	18	4

Treated Water Storage Capacity

The Grants Pass water distribution system includes eight treated water reservoirs serving five separate pressure zones. The treated water storage reservoirs serve three principal purposes: operational storage to meet diurnal fluctuations, emergency storage, and fire flow storage. The required storage volume for these three purposes is determined individually and then combined to identify the total amount of storage volume required within a given pressure zone and for the overall system. For added reliability, storage is located to allow gravity flow into the pressure zone where it is required. This arrangement eliminates the need for pumping facilities that require a backup power system during power outages. Storage located in higher pressure zones also benefits lower zones by providing a potential source of gravity supply through the addition of pressure reducing stations to the system. The City of Grants Pass maintains the following treated water storage standards for evaluating system capacity:

1. Operational storage equal to 45 percent of maximum day demand for current demand and part-time plant operation. Operational storage equal to 25 percent of maximum day demand for future demand and full-time plant operation.

2. Emergency storage equal to 75 percent of maximum day demand.

3. Fire flow storage based on the largest fire flow requirement in the pressure zone.

Table 2-8 summarizes the evaluation of treated water storage requirements for current demand conditions. The existing system contains an overall treated water storage capacity of 19 million gallons which is ample for overall current levels of demand. The significant volume of storage that is available in the system relative to maximum day demand provides ample operating storage, thus allowing the water treatment plant to effectively operate on a part-time basis without compromising emergency storage or fire flow storage supplies. Existing storage volumes are also sufficient for current demand on a zone-by-zone basis except in Pressure Zone 4 where only 80,000 gallons of storage volume is available. The existing available storage in North Valley is somewhat overstated since only approximately one third of the 1.2 million gallon storage volume available in Reservoir 15 is actually utilized due to limited demand in that portion of the system. Otherwise, the quality of water will degrade due to long-term storage.

Table 2-8. Treated Water Storage Evaluation

Pressure Zone	Current Max Day Demand, mgd	Required Operational Storage, mg	Required Emergency Storage, mg	Required Fire Flow Storage, mg	Required Total Storage, mg	Existing Available Storage, mg
1	7.0	3.2	5.3	0.96	9.4	11.5
2	2.1	0.9	1.6	0.96	3.5	4.3
3	0.6	0.3	0.5	0.24	1.0	2.0
4	0.2	0.1	0.2	0.18	0.5	0.1
NV	0.1	0.1	0.1	0.24	0.4	1.2
Total	10.1	4.5	7.6	2.6	14.7	19.0

Booster Pumping Capacity

There are eleven booster pumping stations in the Grants Pass water distribution system. The booster pumping facility criteria require that pumping stations are able to supply the maximum day demand within all dependent pressure zones over a 24 hour period. This criteria assumes that there is sufficient treated water storage within the pressure zone to meet the operational, emergency, and fire storage criteria and that the pumping station is equipped with a backup energy source of sufficient capacity to operate the pumping plant at its rated capacity. The rated capacity of a pumping station is based on the largest pump being out of service. However, for small pressure zones, the pump station is sized so that it can supply peak hour demand with one pump out of service and includes a redundant pump to meet fire flow requirements.

Based on these criteria, Table 2-9 summarizes the booster pumping capacity evaluation for the water distribution system with current demand levels. The water treatment plant pumping capacity is significantly higher than the system-wide maximum day demand, allowing the pumps to operate on a part-time basis. Table 2-9 includes the New Hope Pump Station, which was recently built to serve Zone 2 developments in the southwest. Although construction is complete, the existing connections to the system and associated demand are currently limited.

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Table 2-9. Booster Pumping Capacity Evaluation

Pump Station	Current[a] Max Day Demand, mgd	Current[a] Max Day Demand, gpm	Existing Pumping Capacity, gpm
Plant[b]	10	6,940	11,100
Lawnridge & Madrone	3.1	2,150	4,500
Champion	1	690	2,400
North Valley	0.1	70	570
Hefley[c]	0.2	140	760
Starlite[d]	0.08	60	960
Hilltop[e]	0.04	30	250
Harbeck Heights[f]	0.04	30	180
New Hope[g]	NA	NA	1,050

[a]Peak hour demand is used for those stations that serve areas without reservoirs.
[b]Noted capacity is based on plant operator measurement of the firm capacity for the plant effluent pumps. The capacity of the plant influent pumps is 9,600 gpm with one pump out of service.
[c]The Hefley station has two fire flow pumps with a total capacity of 1,200 gpm.
[d]The Starlite station has a fire flow pump with a capacity of 1,050 gpm.
[e]The Hilltop station has a fire flow pump with a capacity of 750 gpm.
[f]The Harbeck station has a fire flow pump with a capacity of 1,200 gpm.
[g]The New Hope station has a fire flow pump with a capacity of 2,000 gpm.

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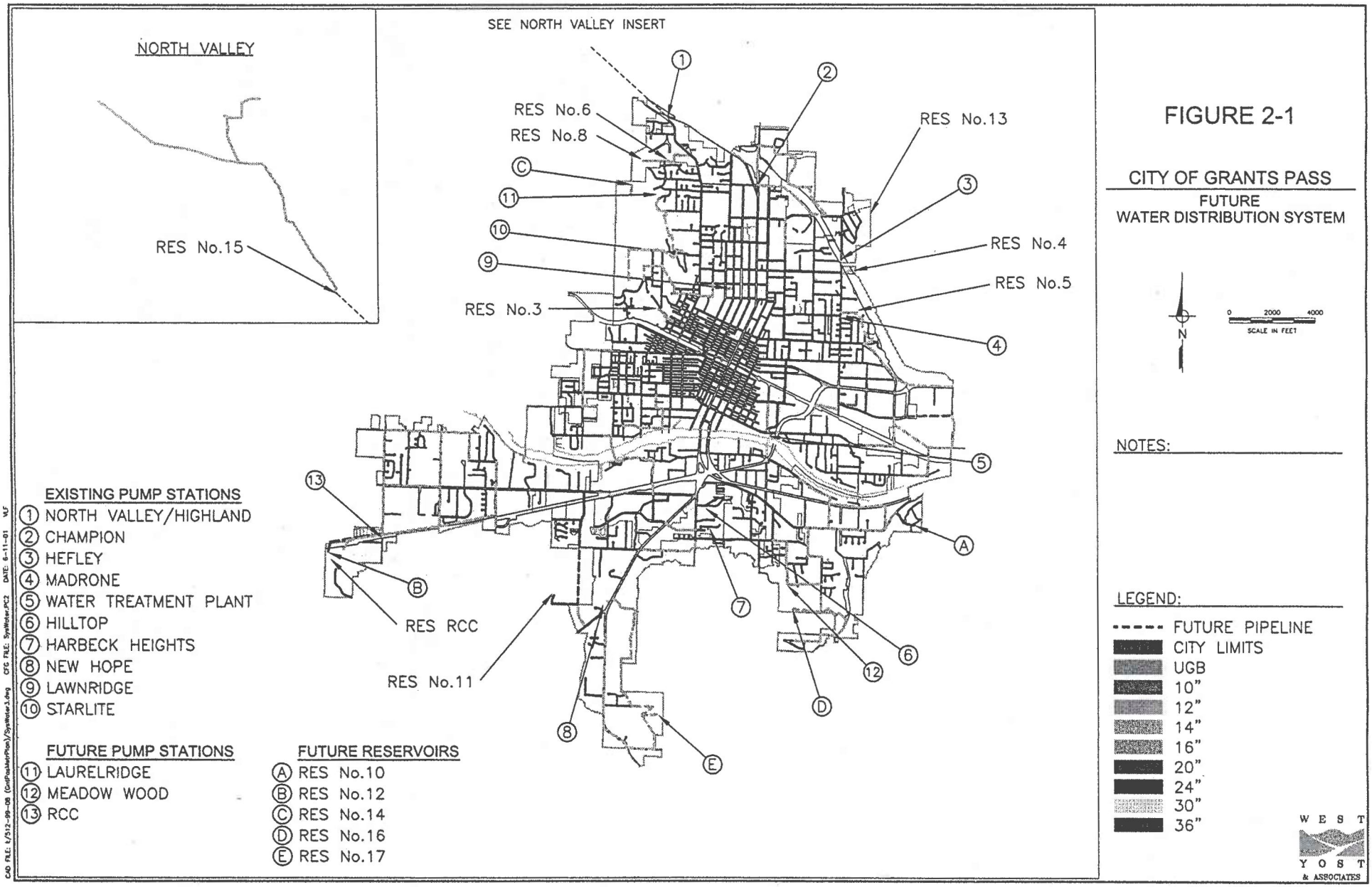

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CHAPTER 3

WATER DEMAND

This chapter presents historic water production and water demand data for the City of Grants Pass and provides a basis for estimating future water demand in the community. Additional analysis relates the various measures of water demand (maximum monthly demand, maximum daily demand, and peak hour demand) to the average annual demand through the use of peaking factors.

The projection of future water demand is based on unit demand factors developed by land use type and corresponding customer classifications. These future demand projections provide the basis for assessing the adequacy of the existing water distribution system and planning for future improvements.

POPULATION

The most recent population estimate for the City of Grants Pass is 23,170 according to the Year 2000 National Census report. The 1990 population was 17,503 indicating an annual growth rate of 2.8 percent during the 1990s. Table 3-1 presents the Grants Pass population for 1990 and each of the past six years. This significant increase between 1999 and 2000 is due to the more comprehensive counting techniques used for the 2000 census relative to the estimates prepared by Portland State University in intermediate years.

Table 3-1. Grants Pass Population

Year	Population
1990	17,503
1995	19,660
1996	20,255
1997	20,535
1998	20,590
1999	20,935
2000	23,170

The Population Research Center at Portland State University has observed that the counties of Southern Oregon in general have been experiencing substantial growth during the 1990s. Many communities along the Interstate 5 corridor, including Grants Pass, have experienced steady in-migration. This trend is expected to continue for Southern Oregon in the future.

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EXISTING WATER USE

Average, Maximum Month and Maximum Day Use

The Grants Pass Water Treatment Plant operators record water production volumes for each day of operation. Analysis of this data allows for the identification of annual average, maximum month, and maximum day water demand. Table 3-2 presents water production data for the past six years.

Table 3-2. Average, Maximum Month, Maximum Day Water Use

Year	Average Day, mgd	Maximum Month, mgd	Maximum Day, mgd	Peaking Factors	
				Maximum Month	Maximum Day
1995	3.73	6.48	8.32	1.74	2.23
1996	4.11	7.22	9.09	1.76	2.21
1997	3.97	6.20	8.83	1.72	2.22
1998	4.17	7.62	9.47	1.83	2.27
1999	4.50	7.79	9.35	1.73	2.08
2000	4.45	7.82	9.73	1.76	2.18
Average	--	--	--	1.76	2.20

Per Capita Water Demand

Per capita demand is a useful measure of household consumption of water. Table 3-3 presents the population for Grants Pass along with the average annual demand during the past six years, which allows for calculation of the average demand in gallons per capita per day (gpcd). Ranging from 190 to 215, the average daily water demand is 200 gpcd. This figure does not take into account water users outside the city limits, for which data is not readily available. Inclusion of these numbers would slightly decrease the per capita usage rate.

Table 3-3. Grants Pass Water Use for 1995 to 2000, gpcd

Year	Population[a]	Average Demand, mgd	Average Demand[b], gpcd
1995	19,660	3.73	190
1996	20,255	4.11	203
1997	20,535	3.97	193
1998	20,590	4.17	203
1999	20,935	4.50	215
2000	23,170	4.45	191
Average	--	--	200

[a]Includes City of Grants Pass population only.
[b]Demands include all uses, including residential, commercial, industrial, public/institutional, and unaccounted for water.

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Unaccounted for Water

All water distribution systems experience losses of water during transmission from the treatment plant to the end user. These losses, known as unaccounted for water, result from many situations including unmetered customers, transmission system leaks, main breaks, faulty meters, fire fighting activities, system flushing, and other miscellaneous hydrant uses. Thus, the total volume of water metered for all end users is always somewhat less than the volume of water produced at the water treatment plant.

Since the City of Grants Pass meters water use for all customers, a comparison of water billing records and water treatment plant production data provides a good estimate of the volume of unaccounted for water in the system. Table 3-4 shows the estimated volume of unaccounted for water in millions of gallons and also as a percentage of total production during the past three years. Based on industry standards, a water loss rate of 10 to 15 percent is considered good and the City's loss rate indicates that the distribution system is in good condition.

Table 3-4. Unaccounted for Water; 1998 -2000

Year	Million Gallons	Percent of Total Water Production
1998	146	9.6
1999	190	11.6
2000	177	10.9

DESCRIPTION OF CUSTOMERS SERVED

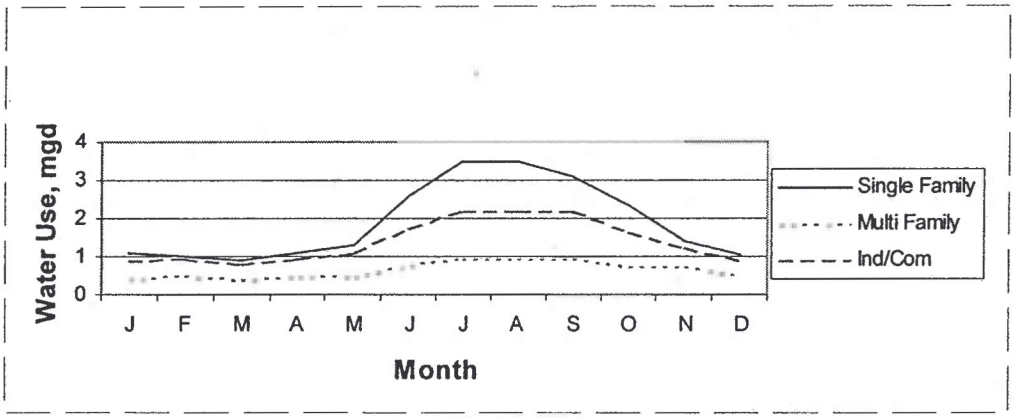
Water Demand by Customer Classification

Water demand related to customer class provides information as to the characteristics of water demand in the community and help to determine where conservation efforts would be most effective. Based on historical billing data provided by the City's Utilities Department for 1998 and 1999, Table 3-5 shows average water demand within three customer classifications: commercial, single family residential, and multi-family residential. There are 1000 commercial accounts, 3175 multi-family accounts and 6027 residential accounts. The commercial classification includes general business, industrial, institutional, and governmental-public land use categories. Single- and multi-family residential users consume 64 percent of the water in Grants Pass. The City serves a few industrial users each with consumptive water use comparable to commercial users. Seasonal demand for each of these three classifications is shown in Figure 3-1.

Table 3-5. Water Use by Customer Classification

	Demand (mgd)			
	Commercial	Multi-Family Residential	Single-Family Residential	Total
1998 Annual Average	1.36	0.61	1.80	3.77
1999 Annual Average	1.40	0.67	1.91	3.98
Average	1.38	0.64	1.86	3.88
Percent of Total Demand	36	16	48	100

Figure 3-1. Seasonal Water Use by Customer Classification*



* Based on 1999 records

Water Demand by Land Use

To develop a unit demand factor for the three different land use patterns, in Grants Pass, the water demand data presented in Table 3-5 is combined with estimated areas for each land use classification. The resulting demand by land use is shown in Table 3-6 and provides the basis for projecting future water demand.

Table 3-6. Unit Demand by Land Use

Land Use	1999 Average Demand[a], mgd	Land Use Area, Acres	Average Unit Demand, gal/acre day
Commercial/Industrial/Public	1.56	1,146	1,400
Multi-Family Residential	0.75	435	1,700
Single Family Residential	2.13	1,977	1,100

[a]The 1999 average demand is based on billing records plus an additional 11.6 percent to reflect unaccounted for water.

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SYSTEM INTERCONNECTIONS

The City supplies water to two areas outside City boundary through agreements with Josephine County. Through the first agreement, the County constructed a 16-inch water line and reservoir (Reservoir 15) to serve North Valley. The City has exclusive operating rights to the system and will eventually assume ownership of the infrastructure. A copy of the agreement is included in Appendix B.

The City also supplies water to an urbanizing area known as Redwood, outside of the southwest City boundary. Under an agreement with the County, the City administers the planning process in the area. As the area develops, areas that are currently served by wells will be connected to the City water system. A copy of this agreement is included in Appendix C.

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CHAPTER 4

WATER CONSERVATION MEASURES

Conservation of resources, including water, is a value that is traditionally held by Oregonians. It is consistent with our respect for natural resources and our shared sense of environmental stewardship. In keeping with this philosophy and in accordance with OAR 690-086-0140, the City is considering feasibility and applicability of the following water conservation measures:

Table 4-1. Water Conservation Measures (WCM) To Be Considered

WCM	Description
1	A system-wide leak repair program or line replacement to reduce system leakage to 10 percent (since system leakage is less than 15 percent).
2	Programs to encourage low water use landscaping.
3	Incentive programs to encourage conservation.
4	Retrofitting or replacement of existing inefficient water-using fixtures.
5	Adoption of rate structures that support and encourage water conservation.
6	Water reuse opportunities.

Further, the City is required to develop an implementation schedule for the following conservation measures:

Table 4-2. Water Conservation Measures To Be Implemented

WCM	Description
7	An annual audit of all water supplied.
8	If the system is not fully metered, a program to install meters on all unmetered water service connections. The program must begin immediately after the plan is approved and must identify the number of meters to be installed each year with full metering compliance within five years.
9	A regularly scheduled program for leak detection for the transmission and distribution system.
10	A meter testing and maintenance program.
11	A public education program on efficient water use.
12	Any other conservation measures that would improve water use efficiency.

EXISTING CITY MEASURES

While the City has not previously submitted a Water Management Plan, the City has implemented several water conservation measures. Many of the existing programs are consistent with the water conservation measures listed in Tables 4-1 and 4-2. Table 4-3 lists the City's

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existing water conservation programs and shows how they conform to the state's list of recommended and required water conservation measures.

Table 4-3. Grants Pass Existing Water Conservation Programs

WCM	Existing Programs
1	The City has a 24-hour emergency number where residents can call to report a water leak during non-business hours.
3	The City has a fully metered system so that all users pay for their water consumption. System development charges (SDCs) are based on meter size. In addition, separate metering for indoor and outdoor use is available for commercial and industrial customers.
4	The City enforces compliance with state regulations.
6	Treated effluent (non-potable) water, rather than potable city water, is used at the wastewater treatment plant where feasible on all new fixtures.
7	The City currently provides an annual accounting of water in compliance with the measurement standards in OAR 690, Division 85. The City meters water as it is diverted and as it leaves the water treatment plant and compares that with metered uses. The City tracks usage by category such as residential, industrial, commercial, and multi-family.
8	All service connections are metered.
9	Staff currently walks the lower elevation portions of the water system annually during flushing activities. Staff and citizens report approximately six leaks per year. Leaks are repaired immediately upon notification. For customer leaks, the City's finance system automatically generates work orders so that customers whose usage is substantially higher than the previous billing period are notified of the possibility of a leak. As an incentive for timely repair, the City forgives 50 percent of the bill over and above normal usage.
10	The City has standardized on Neptune and Badger meters with 15-year warranties. Staff has recently completed replacing older Neptune and Badger residential meters with the new meters. As the meters approach their warranty life, staff will randomly test hundreds of meters. This practice, when conducted in the past, showed that less than 1 percent of the meters are faulty; replacement of the faulty meters is a key component to the City's low unaccounted for water rate. Should a meter fail, the City's finance system automatically generates work orders for meter checks where usage is substantially lower than the previous billing period so that faulty meters can be replaced quickly. The City is currently replacing existing compound commercial meters with Metron single jet meters, which staff has found to be more dependable.
11	The City currently distributes water conservation pamphlets in water bills annually. In addition, conservation information is distributed in the annual Consumer Confidence Report and in the spring City newsletter. Examples are included in Appendix D.
12	To provide accountability for bulk water users, the City has a bulk water dispensing station at the Hillcrest Fire Station. In addition, the City has six 3-inch hydrant flow meters for use by area contractors.

EVALUATION OF ADDITIONAL CONSERVATION MEASURES

The water conservation screening process began with a review of the available literature and Water Management Plans from other Oregon communities. A complete bibliography is given in Chapter 8. From this review, a list of water conservation measures was developed. The list included the following:

- Offer rebates for costs of replacement of lawns with alternative landscaping
- Offer rebates for installation of drip irrigation systems for shrub or tree areas and automatic timers or controllers for turf areas
- Offer rebates for water efficient appliances
- Require water efficient landscaping in the plan approval process
- Construct a water-efficient demonstration garden
- Institute a residential audit program beginning with those single- and multi-family users with the highest consumption
- Distribute free Plumbing Fixture Check-up Kits
- Distribute free low flow shower heads and faucet aerators
- Offer rebates for ultra-low flow toilets
- Retrofit public facilities
- Adopt rates that support and encourage water conservation
- Backwash water treatment filters with untreated water
- Use treated wastewater effluent for irrigation of public areas
- Install additional bulk water dispensing stations to reduce unauthorized users
- Hire a firm to electronically detect system leaks
- Prohibit the wasting of water

These measures were qualitatively evaluated based on the following criteria:

- **Projected Water Savings** – Water savings depend on the applicability of the program to the water market. For Grants Pass, residential is the single highest water use category so programs that focus on residential conservation will likely be the most effective. Savings also depend on user participation and the volume of water saved per participant. Savings are also defined as the relative water savings compared to the estimated cost of implementation.
- **Supplier Cost** – The cost of the measure includes the City's cost to start up, operate, and maintain the conservation program. It may also include additional cost to the consumer in higher rates or reduced cost in water savings and rebates. Costs were estimated based on information from manufacturers, estimated staffing cost, and other communities' experiences.

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- **Political Feasibility and Legal Constraints** – The compatibility of the measure with the local political situation is necessary to consider in the evaluation of a measure. Legal constraints may include conflicts with existing City or State regulations.
- **Consistency with Community Values** – For a measure to be successful, it must be compatible with community-held values. For example, users may participate willingly in a measure to do their part for the community and environment or they may see it as an inconvenience or reduction in service. Consistency with community values was determined by discussions with City leaders and staff.
- **Environmental Impacts** – Measures may have direct or indirect environmental impacts, such as energy conservation, associated with them.

Two measures were dropped from consideration before the initial screening because they were not feasible. The measure that would require using untreated water to backwash filters was eliminated because it will threaten public health given the design of the water treatment process. The measure that would require using treated wastewater to irrigate public areas is not feasible because the City's wastewater treatment plant does not produce effluent of a quality that meets the standards for reuse of wastewater, OAR 340-55.

Table 4-4 shows the result of the preliminary screening of the remaining water conservation alternatives.

Of the thirteen measures that were screened, five resulted in an unfavorable rating. Two of the unfavorable measures involved sizable rebates. These measures were judged as too costly relative to the water savings.

The measure that would require water-efficient landscapes through the planning approval process is not feasible immediately. The City recently completed the lengthy process of updating their code; the update did not include conservation landscaping. However, the code is updated periodically and landscaping conservation requirements will be included in a future update.

The fourth measure receiving an unfavorable rating involved hiring a consultant to electronically detect leaks in the transmission system. The City's unaccounted for water rate for 2000 was 10.6 percent. Since this includes system flushing and fire fighting as well as leakage, it is certain that the leakage rate is below the 10 percent goal established by the State. Further, the City's capital improvement program already includes replacement of the City's oldest pipes, which are the most likely to leak. Therefore, the benefit from this measure was seen as minimal and it was dropped from further consideration.

Last, since public facilities are small, retrofitting them was determined to be too costly for the actual water that would be saved. They will be retrofitted per the state regulations as fixtures are replaced during regular maintenance and any new construction will comply with state regulations for low flow fixtures.

The two measures receiving neutral ratings were rebates for efficient irrigation systems and residential audits. These measures can be considered in more detail in the future if the top-rated measures do not provide expected results or if it is determined that additional measures are necessary.

Six measures received a favorable rating and were subjected to a detailed cost/benefit analysis. These measures are described in the following section.

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Table 4-4. Initial Water Conservation Program Screening

Program	Water Savings	Supplier Cost	Political/Legal Feasibility	Community Values	Environmental Impacts	Summary
Rebate for lawn replacement	●	○	○	○	⊖	○
Rebate for efficient irrigation systems	●	○	○	●	●	⊖
Require water efficient landscapes	●	⊖	○	○	○	○
Construct demonstration garden	⊖	●	●	●	⊖	●
Conduct residential audits	○	⊖	⊖	○	⊖	⊖
Distribute plumbing fix-up kits	⊖	⊖	●	●	●	●
Rebates for shower heads and faucet aerators	●	⊖	⊖	●	●	●
Rebates for ultra-low flow toilets	⊖	○	○	○	●	○
Retrofit public facilities	○	○	●	●	○	○
Adopt rates to encourage conservation	●	●	⊖	⊖	●	●
Install additional bulk water dispensers	●	⊖	●	●	●	●
Hire a firm to electronically detect leaks	○	○	⊖	○	⊖	○
Implement a water waste prohibition	⊖	●	⊖	●	●	●

● = favorable; ⊖ = neutral; ○ = unfavorable

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Description of Measures

Additional Bulk Water Dispensing Stations. Although this measure is not strictly a conservation measure in that it does not reduce water use, it is important in reducing the quantity of unaccounted-for water in the system. Installing bulk water dispensers where water use would be metered and sold for a nominal fee would reduce unauthorized use of water. The City proposes installing up to three stations. For the analysis, it was assumed that water would be sold for an amount that would cover the operation of the stations. The cost of producing the water, $0.04/100 gallons according to City staff, was income previously lost and would be recouped to pay back the capital cost of the stations.

Inverted Block Water Rates. Currently the City has a uniform water rate. Customers are charged a flat monthly rate for the first 500 cubic feet of water and a flat commodity charge by customer class for water over and above 500 cubic feet. A surcharge is added for higher pressure zones. The City's Bill Equalizer Payment Plan allows residents to pay an average monthly rate year round. While this allows residents to more easily budget for their utility bills, it does not encourage conservation, particularly in the high-use summer months. An inverted block rate would establish increasing prices for successive consumption blocks. This rate structure will effectively reduce summer usage and if the rate is structured correctly, maintain current levels of water department funding.

Low Water Use Demonstration Garden. The City proposes constructing a demonstration garden at the water treatment plant. The garden would showcase drought tolerant plants and efficient irrigation practices and equipment. With the garden, literature would be made available to further educate citizens on the advantages of low water use landscaping.

Distribute Plumbing Kits. Plumbing kits are available for distribution to single- and multi-family residential customers. For this analysis, it was assumed that over a period of two years, the City would distribute up to 1,500 plumbing kits to residents who requested them. The plumbing kits would include a low flow showerhead, a faucet aerator, and dye tablets to detect leaks. According to literature, the kits will reduce indoor water use up to 12.5 gpcd in the households where they are installed.

Fixture Rebates. As a follow-up program to the plumbing kit distribution, the City could offer rebates for low flow showerheads and faucet aerators. The City could initiate the program by itself, or in partnership with the power utility. It was assumed for this analysis that the program would be operated by the City and that 100 of the showerheads and aerators would be sold. Pacific Power and Light recently offered rebates on plumbing fixtures along with high-efficiency light bulbs.

Water Waste Prohibition. This measure has no capital cost associated with it; nor does it have a definable water savings associated with implementation. However, implementation of this measure is important because it defines the City's values as to the use of water as a limited resource. It is therefore included as a proposed water conservation measure. The measure would consist of passing an ordinance with a specific enforcement component.

Basis of Analysis

For each measure listed in this section, an estimate was made of implementation costs and the amount of water that would be conserved. The estimated amount of water conserved was based on the experiences of similar programs cited in the available literature. The financial benefit of implementing each measure could be calculated as the reduced operation cost to produce the water and delayed or eliminated capital projects due to lower water use. However, the reduced operation cost is offset by revenue lost by not selling the water (with the exception of the bulk water dispensing station). In addition, the capital projects listed in the Grants Pass Distribution System Water Master Plan are related to growth and system performance and would not be affected by reduced water consumption. Therefore, the programs were evaluated strictly on their cost per unit of water saved.

Analysis Results

Table 4-5 summarizes the estimated cost for each 100 gallons of water saved for the proposed conservation measures.

Table 4-5. Conservation Measure Analysis Summary

Measure	$/100 gallons saved
Bulk Water Dispensing Stations	0.16
Inverted Block Water Rates	0
Low Water Use Demonstration Garden	0.10
Distribute Plumbing Kits	0.10
Showerhead and Faucet Aerator Rebates	0.29
Water Waste Prohibition	0

Table 4-5 shows that implementing new water rates to discourage high water use is the most cost-effective conservation measure. The demonstration garden, plumbing kit distribution, and bulk water dispensing stations are moderately affordable and the rebate for showerheads and faucet aerators is the most expensive conservation measure considered.

RECOMMENDED CONSERVATION MEASURES

As a result of the evaluation described above, five conservation measures are recommended for implementation. Table 4-6 presents the results of the evaluation along with the State's WCM category for each measure:

Table 4-6. Water Conservation Program Recommendations

WCM Category	Program Description	Recommendation
12	Additional Bulk Water Dispensing Stations	Recommended
5	Inverted Block Water Rates	Recommended
2, 11	Low Water Use Demonstration Garden	Recommended
4	Distribute Plumbing Kits	Recommended
4	Showerhead and Faucet Aerator Rebates	Not Recommended
12	Water Waste Prohibition	Recommended

IMPLEMENTATION SCHEDULE

Table 4-7 shows an implementation schedule for the recommended conservation programs. The schedule shows that three measures (a new water rate structure, a water waste prohibition, and bulk water dispensing stations) be implemented immediately and that the remaining programs be implemented in following years.

Table 4-7. Recommended Conservation Program Implementation Schedule

Program Description	Implementation Schedule
Additional Bulk Water Dispensing Stations	2001
Inverted Block Water Rates	2001
Low Water Use Demonstration Garden	2002
Distribute Plumbing Kits	2003
Water Waste Prohibition	2001

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CHAPTER 5

WATER CURTAILMENT PLAN

OAR 690-086 requires that the Water Management Plan include a description of water supply deficiencies that have occurred within the last 10 years and a discussion of the ability of the water supplier to maintain delivery during long-term drought or short-term shortages. Also required is a water curtailment plan that includes a list of three or more stages of alert for potential shortage or water service difficulties, a description of predetermined levels of severity of shortage or water service difficulties that would trigger curtailment actions under each stage of alert, and a list of specific standby water use curtailment actions for each stage of alert.

The City has an existing water curtailment ordinance, Section 8.04.070, Emergency Water Conservation Procedures. The ordinance authorizes the Mayor to declare a water conservation emergency and gives the City Manager or his designee the authority to implement a water conservation program. The ordinance outlines conditions under which an emergency could be declared and lists measures that could be implemented to reduce water consumption. To comply with OAR 690-086, the existing ordinance will be repealed and a new ordinance will be implemented to conform to the stages of alert and definitions of triggers outlined in the plan. A copy of the new ordinance is included as Appendix E.

FREQUENCY AND MAGNITUDE OF SUPPLY DEFICIENCIES

There have been no supply deficiencies over the last 10 years. The City's source, the Rogue River, provides ample supply even in the driest conditions according to historical flow data. Water production is limited to 18 mgd by the capacity of the water treatment plant, which is much higher than current levels of demand. Peak day use in 2000 was 9.7 mgd, just 54 percent of the plant capacity. Available storage in the existing distribution system is summarized in Table 5-1.

Table 5-1. Available Storage

Type of Storage	Available Storage, mg
Operational	6.5
Emergency	9.9
Fire Flow	2.6
Total	19.0

Since Grants Pass relies solely on the Rogue River for its supply, the City is vulnerable to contamination of the river. It is estimated that it would take three days for a contaminant plume to pass the City's intake or for the treatment process be modified to neutralize the contaminant. Emergency storage will provide three days of storage if demand is reduced to about 70 percent of

the annual average demand, which is achievable with restrictions. The storage would last even longer if use were restricted to essential needs only.

CURTAILMENT TRIGGERS

Although the City has been fortunate not to have experienced a water shortage, the potential exists for a situation where the water supply cannot meet demand for a time. The shortage could be City-wide due to source contamination, treatment difficulties, prolonged drought, pumping, or transmission problems. The shortage could be localized to a pressure zone due to distribution, pumping, or storage problems. Whatever the situation, the City will be well served to have in place a curtailment plan that defines levels of water shortage severity and mechanisms for dealing with the situation.

The City will institute three levels of water supply alert. The alert levels and their operational triggers are summarized in Table 5-2.

Table 5-2. Water Shortage Alert Levels and Triggers

Alert Level	Description	Trigger
One	Potential Water Supply Shortage	A serious drought condition is occurring or is likely to occur in the region or Rogue River flow rates are measured or projected to be below a 1-in-10 year low flow level, or the County or State has declared a drought condition.
Two	Water Supply Shortage	The City's ability to deliver water is not adequate to meet demand due to supply, treatment, or pumping restrictions, or extended treatment plant operation is required and storage cannot be maintained.
Three	Critical Water Supply Shortage	The supply is interrupted.

CURTAILMENT ACTIONS

For each level of alert, actions appropriate to the situation will be implemented to curtail water consumption. The following sections identify curtailment actions for the different alert levels.

Level One Alert – Potential Water Supply Shortage

The City Manager has the authority to activate some or all of the following voluntary curtailment actions listed below until the reasons for a Level One Alert have passed:

1) Restrict watering based on odd/even address numbers for residential and business customers, and governmental agencies. No watering will be allowed on Wednesdays. The schedule will apply to all lawn watering and all nonessential

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water uses with exceptions as specified by the Manager. Watering hours will be restricted to before 6 a.m. and after 9 p.m.

2) Distribute brochures regarding conservation measures.
3) Implement a media outreach program.
4) Notify major water users asking for reductions in use or moving nonessential use to off-peak hours.
5) Cease operation of non-recirculating fountains.
6) Restrict hydrant and water line flushing.

Level Two Alert – Water Supply Shortage

The City Manager has the authority to mandate any or all of the following actions until the reasons for the Level Two Alert have passed:

1) Any Level One Alert actions.
2) No watering or lawn irrigation will occur unless the following specific uses are approved by the City Manager:
   a) New lawn, grass, or turf that has been seeded or sodded after March 1 of the current calendar year
   b) Athletic fields frequently used for organized play
   c) Golf course tees and greens
   d) Park and recreation areas of particular value to the community
3) City water will not be used to clean, fill, or maintain levels in decorative fountains.
4) City water will not be used to clean sidewalks, walkways, streets, driveways, parking lots, or other hard surfaces except where necessary for public health and safety.
5) City water will not be used to wash vehicles including automobiles, trucks, trailers, trailer houses, motorcycles, boats, or other type of mobile equipment.
6) Limitations may be placed on industrial and commercial water consumption.
7) Hydrant and water main flushing will be done in emergencies only.

Level Three Alert – Critical Water Supply Shortage

The City Manager has the authority to mandate any or all of the following actions until the reasons for the Level Three Alert have passed.

1) Any Level One Alert actions.
2) Any Level Two Alert actions.

3) No City water will be used for watering of landscaping or irrigating of lawns, grass, turf, athletic fields, golf course tees and greens, or parks and recreation areas.

4) No City water will be used to fill or maintain levels in scenic or recreational ponds and lakes, or other structures making similar use of water.

5) No City water will be used to fill, refill, or add to any swimming pools.

6) No City water will be used to wash the outside of buildings.

7) No City water will be used on construction projects.

8) No City water will be served to restaurant customers unless requested.

9) Residential use will be limited to health and safety uses only.

10) Limitations will be placed on industrial and commercial users up to and including interruption of supply with the exception of health and safety uses only.

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CHAPTER 6

LONG RANGE WATER SUPPLY

FUTURE WATER NEEDS

Land use demand factors for the City of Grants Pass were developed using zoning information from the local comprehensive plan and historical water use data. The land use demand factors are shown in Table 3-6. Although the timing of land use development within the UGB is unknown, information is available regarding the current zoning designation for all properties within the UGB. Table 6-1 summarizes the acreage of properties within the UGB according to land use, differentiating between properties that are currently receiving water service and those that will connect to the water distribution system in the future. Using the unit demand factors developed for these land use classifications, the table also projects average annual water demand at the UGB build-out condition. This analysis assumes the existing mix of residential and commercial properties will stay the same in the future.

Table 6-1. Land Use Based Water Demand Projections for UGB Build-Out

Land Use	Existing Acreage	Future Acreage	Total Acreage	Unit Demand, gallons/acre-day	Estimated Average Day Demand, mgd
Commercial	1,146	598	1,744	1,400	2.4
Single-Family Residential	1,977	2,419	4,396	1,100	4.8
Multi-Family Residential	435	440	875	1,700	1.5
Redwood and North Valley[a]					0.8
Total	3,558	3,457	7,015		9.5

[a]North Valley and the Redwood district, portions of which are adjacent to but not currently within the UGB, are expected to continue using Grants Pass water, adding 0.3 and 0.5 mgd, respectively, to the build-out demand.

Grants Pass experienced an annual growth rate of 2.8 percent from 1990 to 2000. The Grants Pass Distribution System Water Master Plan predicted a build-out population of 45,000. Assuming a current population of 23,170 and a conservative consumption rate of 200 gpcd, a straight line projection allows for the interpolation of water requirements for intermediate years 2010 and 2020. Future maximum day demands can be developed using the peaking factor developed in Table 3-2. Future water requirements are summarized in Table 6-2.

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Table 6-2. Estimated Future Water Demand

Year	Average Day Demand, mgd	Maximum Day Demand, mgd
2010	6.1	13
2020	8.1	18
Build-out	9.5	21

AVAILABLE SOURCES OF WATER

As discussed in Chapter 2, the Rogue River provides an ample and reliable water supply for the City of Grants Pass as it expands to the limits of its urban growth boundary. The City holds four water rights totaling 87.5 cfs (56.4 mgd). The listing of salmon as an endangered species and long-term climate change potentially may have an effect on the reliable yield of the river in the future. These issues warrant attention, but are ill defined at this time. It can be concluded that the City's supply can meet near and long-term future water needs. Historically, development surrounding the City has relied on groundwater supplies. Wells in the region either have not been reliable or productive or have had groundwater quality issues associated with them which has led to the extension of service by the City. The North Valley system is an example where alternative sources of water were not able to meet development demands.

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CHAPTER 7

PLAN UPDATE SCHEDULE

The City will monitor water production and metered sales and report its findings to the Water Resources Department as a part of the annual water use reporting program under OAR 690-85. This report will include a summary of the results of the water conservation measures implemented in the past year and note any significant changes in growth projections, per-capita water use, or community water needs.

The City will update this Water Management Plan five (5) years after it is approved by the Water Resources Department. The update will include recent water consumption data, system improvements, and an evaluation of the effectiveness of the conservation measures implemented as a result of this plan.

CHAPTER 8

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CITY OF GRANTS PASS, OREGON

WATER TREATMENT PLANT
FACILITY PLAN
Final Report

MAY 2004

MWH
MONTGOMERY WATSON HARZA

EXHIBIT 6
to UAPC Staff Report 3-31

CITY OF GRANTS PASS, OREGON

WATER TREATMENT PLANT
FACILITY PLAN



MAY, 2004

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ABREVIATIONS AND ACRONYMS

In order to conserve space and improve readability, the following abbreviations and acronyms have been used throughout this report:

°C	degrees centigrade
µg/L	micrograms per liter
µm	micrometers
ADA	Americans with Disabilities Act
BAT	Best Available Technology
BW	backwash
CI	cast iron (pipe)
CIP	cast iron pipe
CMU	concrete masonry unit
CT	product of concentration (C) and contact time (T)
D/DBPR	Disinfectants/Disinfection By-Products Rule
DBP	disinfection by-product
DHS	Oregon Department of Human Services, Drinking Water Program
DI	ductile iron (pipe)
EPA	United States Environmental Protection Agency
ESWTR	Enhanced Surface Water Treatment Rule
fps	feet per second
ft	foot
FTW	filter-to-waste
gal/sf	gallons per square feet
gph	gallons per hour
gpm	gallons per minute
gpm/sf	gallons per minute/square feet
HAA	haloacetic acid
HAA_5	sum of 5 HAA compound concentrations
HDPE	high density polyethylene
hp	horsepower
I&C	instrumentation and control
ICR	Information Collection Rule
IDSE	Initial Distribution System Evaluation
IESWTR	Interim Enhanced Surface Water Treatment Rule
IOC	inorganic contaminants
LCR	Lead and Copper Rule
LRAA	Locational Running Annual Average
MCC	motor control center
MCL	maximum contaminant level

MCLG	maximum contaminant level goal
MG	million gallons
mgd	million gallons per day
mg/L	milligrams per liter
mm	millimeter
NPDES	National Pollutant Discharge Elimination System
NPDWR	National Primary Drinking Water Regulations
NTU	Nephelometric Turbidity Unit
O&M	operations and maintenance
PE	plant effluent
PGE	Portland General Electric
PLC	programmable logic controller
ppd	pounds per day
psi	pounds per square inch
RCP	Reinforced concrete pipe
SCADA	Supervisory Control and Data Acquisition
scfm	standard cubic feet per minute
SDWA	Safe Drinking Water Act
sf	square foot
SOC	synthetic organic chemicals
SOW	Scope of Work
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TDH	total dynamic head
THM	trihalomethane
THMR	Trihalomethane Rule
TOC	total organic carbon
TSS	total suspended solids
TTHM	total trihalomethanes
UBC	Uniform Building Code
UBWV	unit backwash volume
UCMR	Unregulated Contaminant Monitoring Regulation
UFC	Uniform Fire Code
UFRV	unit filter run volume
UPS	uninterruptible power supply
UV	ultraviolet
VFD	variable frequency drive
VOC	volatile organic chemicals
WTP	water treatment plant
WRP	water restoration plant
WWW	waste washwater

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ES EXECUTIVE SUMMARY

The City of Grants Pass Water Treatment Plant (WTP) has successfully met the City's drinking water needs for over 70 years. The Rogue River supply is typical of many Pacific Northwest surface waters with low mineral content, low pathogen concentrations, and normally low turbidity, but with seasonal increases in turbidity due to precipitation and runoff. The Rogue River quality and flow is also influenced by the operation of upstream reservoirs including Lost Creek Reservoir and Savage Rapids Dam. Peak withdrawals by the WTP to meet demands in the summer months coincide with minimum river flows and low turbidities.

The WTP's main purposes include removal of suspended particulates, removal and inactivation of pathogens, and production of non-corrosive, palatable water according to Federal and State drinking water regulations. The plant has historically met all regulations and the few customer complaints are limited to occasional chlorinous tastes and odors. The plant appears well-positioned to continue to meet current and future drinking water regulations.

The plant's production has steadily increased over the last decade in response to increasing water demands within the City's service area. The City's service area has been expanding as areas previously served by small groundwater systems have been incorporated into the City's water system. Significant investments have been made to upgrade the distribution and storage systems over the past few years. Water production at the plant has increased by approximately 20% since 1995. In 2003, peak day water production from the WTP was 10.3 mgd, peak week production was 9.6 mgd, peak month production was 9.2 mgd, and the average annual production was 5.1 mgd.

The plant has a rated maximum capacity of 20 mgd with all raw water and finished water pumps operating. The reliable plant capacity is approximately 15 mgd with one of the largest pumps out of service. The plant is operated in a start/stop mode each day, with the hours of production varying between 8 to 15 hours per day depending on demands

and raw water quality. The plant normally operates at the peak production rate of 20 mgd (14,000 gpm) part of each day during the high demand season. As demands have increased each year, the daily plant operating duration has also increased. Eventually, the plant will have to increase its operating staff to allow 24 hour per day production during the peak demand season.

The Water Treatment Plant Facilities Plan (WTPFP) provides guidance for improving this major element of the City's water system and recommends a capital improvement program (CIP) that will meet the City's water treatment needs for the next 20 to 25 years.

Initial efforts for the WTPFP included the following elements that represent a "situation audit" according to planning guidelines for water treatment plants:

- Review of current and future water demands;
- Review of historical water quality and WTP performance;
- Review of current and future drinking water regulations and compliance;
- Review of hydraulic and process capacity;
- Detailed investigation of the filter media and alternative coagulation schemes; and
- Review of plant facilities and systems, for performance and code compliance

At the current rate of growth, it is expected that the plant will continue to be able to meet the City's water needs for at least the next 20 years, with some modifications and improvements. A major plant expansion is not envisioned until the middle to end of decade 2020. Although the existing plant site is extremely confined, the plant is capable of being expanded to approximately 30 mgd with major modifications. The existing plant structures appear to have significant remaining useful life. However, the older plant structures are vulnerable to damage during a severe seismic event.

While the plant has been able to successfully meet the City's water demands and also produce good water quality, this facilities planning effort determined that some

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challenges exist which have regulatory compliance implications and which create production inefficiencies including:

- The existing Rogue River intake does not comply with current Endangered Species Act (ESA) regulations to protect juvenile fish including salmonids, due to high approach velocities and screen deficiencies;
- The backwash/sludge holding pond is completely full of solids and immediate action is required, including development of a long-term solids management plan, to ensure continued compliance with the City's NPDES permit for discharge to Skunk Creek;
- The filter media is in a degraded condition and the plant (specifically the filters and sedimentation basins) is operating inefficiently, thereby requiring frequent backwashing and excessive raw water pumping, resulting in higher operating costs and longer operating durations; and
- Proposed drinking water regulations, including the Disinfectants/Disinfection By-Products (D/DBP) Rule and the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR), have the potential to require significant plant modifications depending on the outcome of current monitoring programs.

These challenges require the City to implement near-term improvements to the plant. The plant also requires a longer-term capital improvement program (CIP) to ensure reliability and redundancy of major equipment, including adding new equipment, replacement/repair of major equipment as they age and become less reliable, and to prepare for a major plant expansion.

Based on a prioritization and budgetary constraint assessment, ***Table ES-1*** presents the recommended near-term CIP for the WTP with estimated costs in year 2003 dollars:

Table ES-2 lists lower priority improvements to be completed starting in the fiscal year 2008/2009. Some of these projects might be completed earlier and/or broken into smaller elements as the plant's operating budget allows.

TABLE ES-1: NEAR-TERM IMPLEMENTATION PLAN FOR WTP IMPROVEMENTS

Fiscal Year	Improvements	ESTIMATED PROJECT COSTS[1]
Current	1. Solids Handling Improvements	$175,000
2004/2005	1. Intake Modifications (Engr. and Permitting)	$400,000
2005/2006	1. Intake Modifications (Engr. and Construction) 2. Filter Upgrades (Engr. and Construction) 3. Basin Modifications (Engr. and Construction)	$500,000 $200,000 $200,000
2006/2007	1. Intake Modifications (Construction) 2. Filter Upgrades (Construction) 3. Basin Modifications (Construction)	$700,000 $400,000 $400,000

[1] All costs presented in Year 2003 dollars. Costs should be escalated at an appropriate rate to determine cost for future years.

TABLE ES-2: IMPLEMENTATION PLAN FOR LOWER-PRIORITY WTP IMPROVEMENTS

Fiscal Year	Improvements	ESTIMATED PROJECT COSTS[1]
2008/2009	1. Filter Gallery Upgrades (Engr. and Construction)	$200,000
2009/2010	1. Filter Gallery Upgrades (Construction) 2. Chemical System Upgrades (Engr. and Const.)	$480,000 $ 50,000
2010/2011	1. Chemical System Upgrades (Construction) 2. Sludge Removal Systems (Engr. and Const.) 3. New Storage Building (Engr. and Construction)	$130,000 $ 75,000 $ 25,000
2011/2012	1. Sludge Removal Systems (Construction) 2. New Storage Building (Construction)	$225,000 $ 50,000
2012/2013	1. Emergency Generator for 5 mgd (Engr. and Const.)	$300,000

[1] All costs presented in Year 2003 dollars. Costs should be escalated at an appropriate rate to determine cost for future years.

In addition to the capital improvements presented above, the City should also implement the following efforts for the WTP over the next few years:

- Continue to explore alternative coagulation options to reduce solids production, improve plant performance and reduce operating costs;
- Continue collecting *Cryptosporidium* samples from the Rogue River to determine "bin classification" according to the LT2ESTR;

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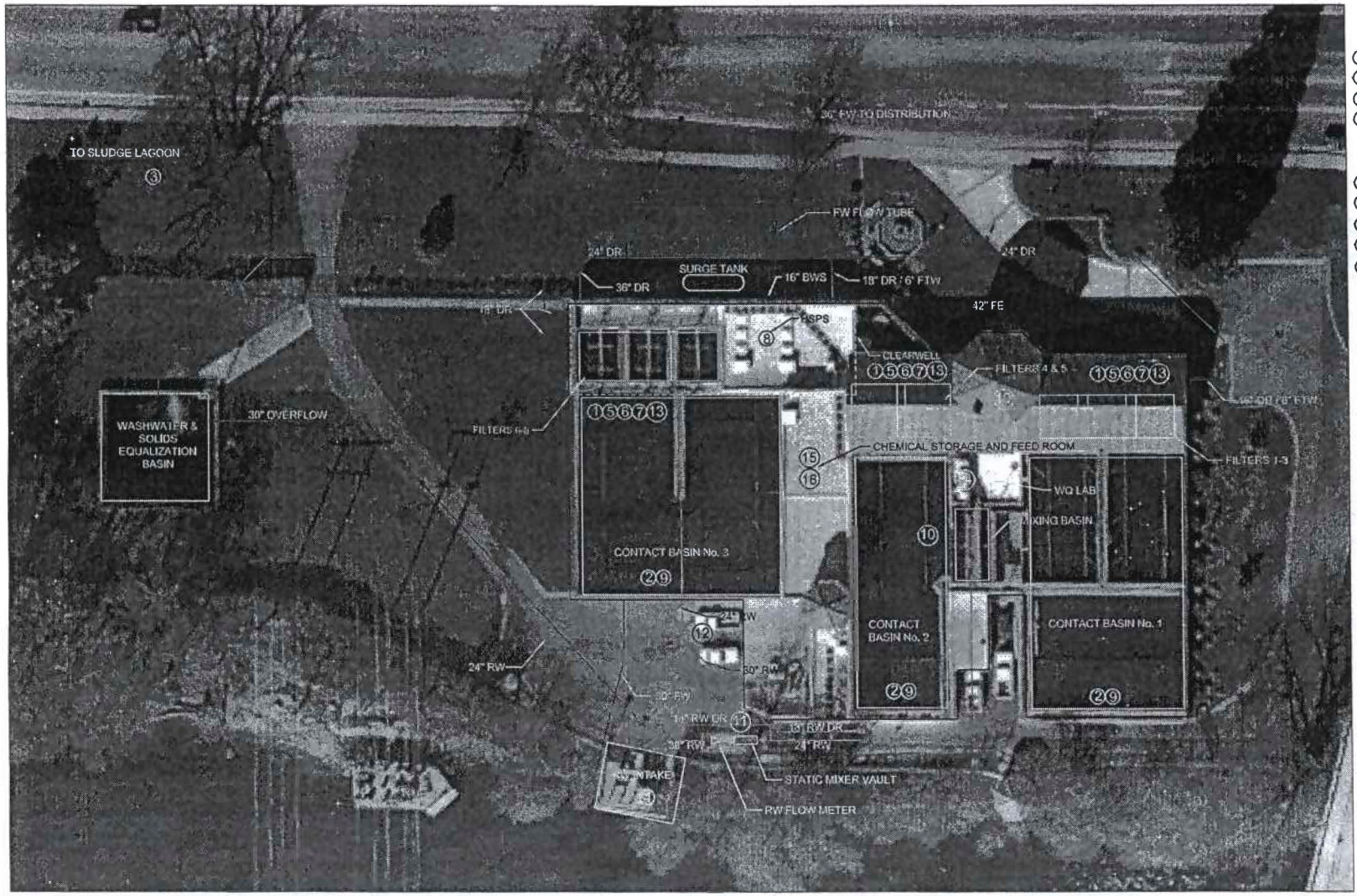
- Develop a DBP sampling program based on the proposed regulations, in conjunction with State of Oregon DHS, to monitor for trihalomethanes (THMs) and haloacetic acids (HAAs), to verify compliance with the proposed Stage 2 D/DBP Rule;
- Complete a Seismic Vulnerability Study; and
- Assess the viability and costs of the sludge handling and disposal program currently being implemented.

In the next 5 to 10 years, the City will need to verify that it can meet the LT2ESWTR and the D/DBP Rule with the existing plant. Current limited monitoring data suggests that compliance with both rules is likely. If compliance is ultimately determined to be unlikely, then the City may have to implement an alternative disinfection scheme at the WTP. The lowest cost approaches include UV irradiation and/or chloramines.

The City should periodically monitor plant performance and water demands over the next 10 years as it makes capital improvements and to verify that planned improvements are still required. An update of the WTP Facilities Plan should be completed in 5 to 10 years depending on water demands and regulations, including a review of plant expansion requirements.

As mentioned above, the plant is capable of being expanded to approximately 30 mgd with major modifications. Based on current growth estimates, the plant expansion will not be required until the middle to end of decade 2020. The estimated project cost for a plant expansion to 30 mgd is $7.5 million, in 2003 dollars, which minimizes the use of additional footprint on the existing site. It is recommended that the City assess available property for a future new plant to expand/partially replace the existing plant within the next 50 years.

Figure ES-1 presents a site plan of proposed plant improvements and upgrades for the next 10 years. ***Figure ES-2*** presents a site plan indicating improvements to expand the plant to 30 mgd.



Tier One Improvements

1. Re-build Filters
2. Add Floc and Baffling
3. Solids Handling and Disposal
4. Intake Modifications (30 MGD)

Tier Two Improvements

5. Replace Filter Valves/Actuators
6. Re-build Filter Gallery Piping
7. Replace/Relocate Filter Instrumentation
8. Spare Backwash Pump
9. Install Automated Sludge Removal
10. Relocate Lime Addition
11. New Coagulant Feed and Injection
12. New Flowmeter in Basin #3
13. Filter Effluent Particle Counters
14. Spectrophotometer for UV 254
15. Containment for Alum Tanks
16. Storage and Maintenance Area
17. OSHA Upgrades (Not Shown)
18. Seismic Vulnerability Study (Not Shown)
19. HVAC Upgrades
20. Emergency Generator (Not Shown)

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Figure ES-1
City of Grants Pass
Water Treatment Plant Facility Plan
Improvements to Maintain Capacity

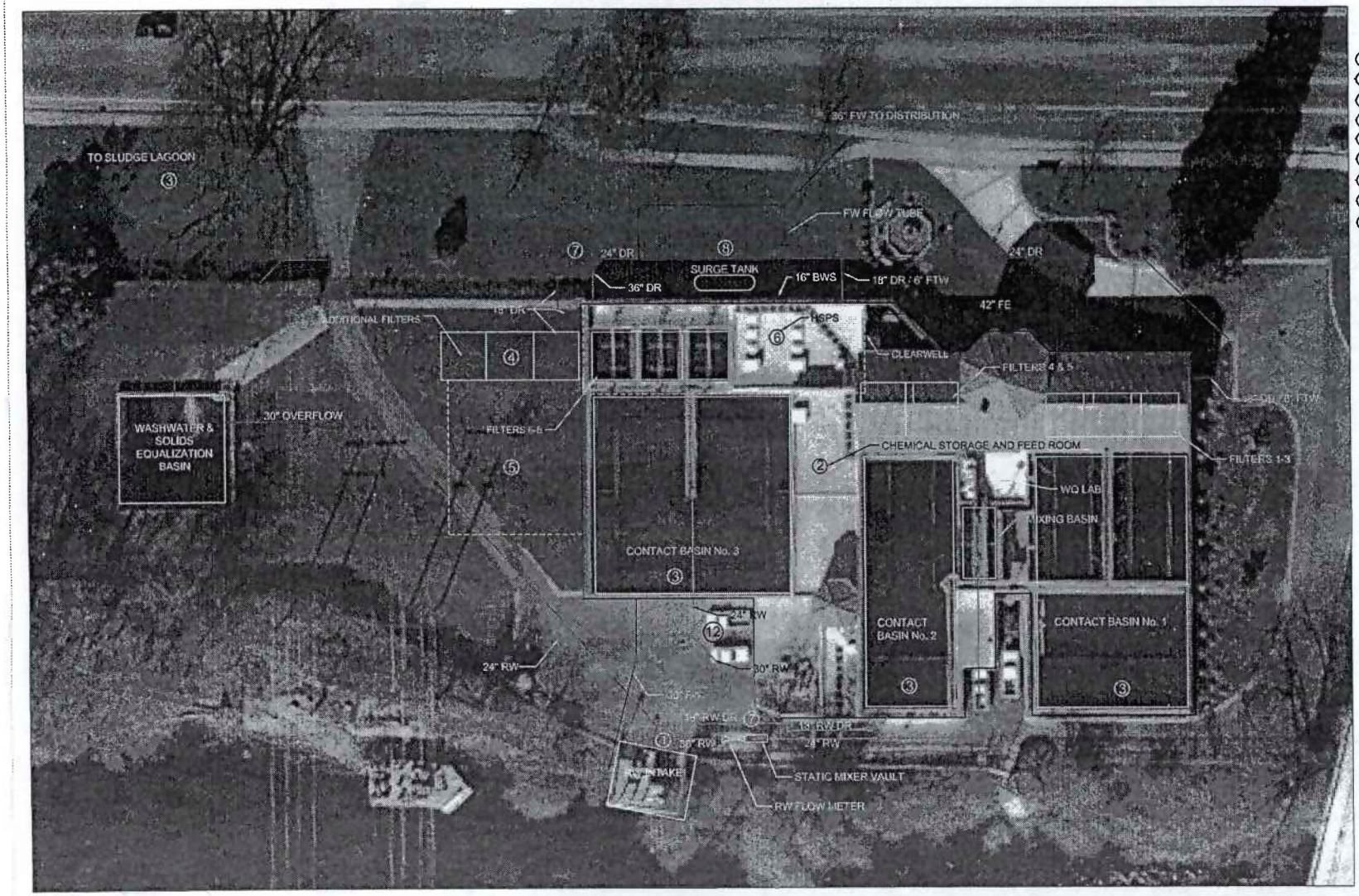

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Figure ES-2
City of Grants Pass
Water Treatment Plant Facility Plan
Improvements to Expand to 30 MGD

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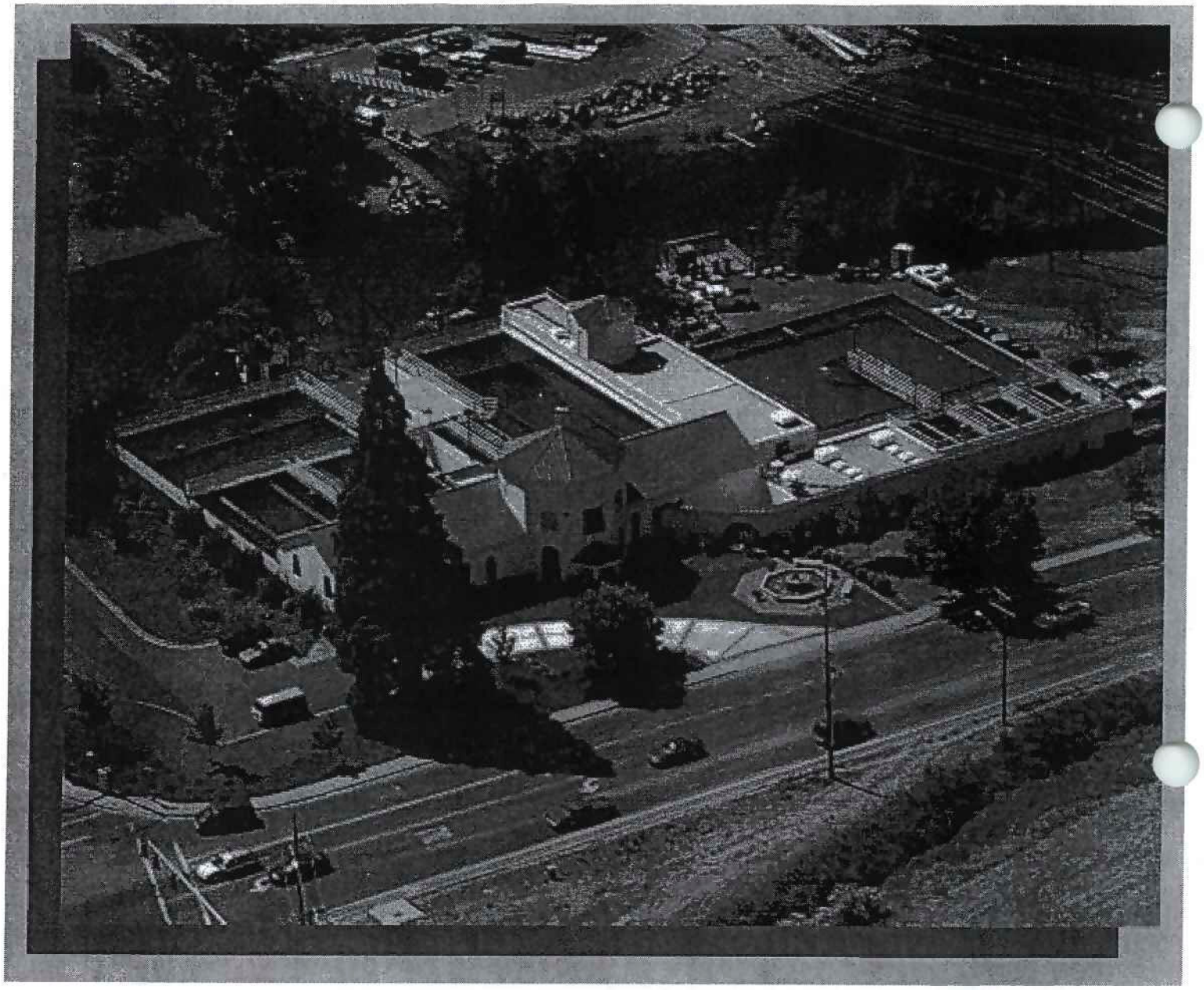

Introduction and Background

1 INTRODUCTION AND BACKGROUND

The purpose of the Grants Pass Water Treatment Plant Facility Plan (WTPFP) is threefold:

1) Define the ability of the existing plant to reliably continue serving the City's water needs,
2) Develop a list of prioritized Capital Improvements to upgrade the plant to improve operations, to meet increasing demands, and to meet existing and future drinking water regulations, and
3) Prepare a plan for water treatment needs within the 20 year planning horizon.

The WTPFP summarizes current and historic performance and design features of the Grants Pass Water Treatment Plant (WTP), provides guidance for improving this major element of the City's water system, and recommends a capital improvement program (CIP) that should meet the City's water treatment needs for the next 20 to 25 years. The report includes basic information and supporting materials to allow preliminary engineering analyses for upgrade and improvement options.

The work effort for the plant evaluation includes a Performance Evaluation, Regulatory Review, Capacity Review and Facilities Review. Each review is summarized in separate sections of this report. The reviews and analyses offer insights into possible improvements which may be required for a number of reasons including: maintaining existing capacity, increasing capacity, optimizing performance, meeting future drinking water regulations, ensuring a long remaining useful life, safety, and operational efficiency.

Following the plant evaluation, improvement alternatives are compared. Recommended improvements are presented, along with planning level cost estimates, according to priority. Sections 6 and 7 of this report present costs and a recommended schedule of improvements over the 20-year planning period.

1.1 WTP AND ROGUE RIVER SUPPLY BACKGROUND

The City has been experiencing steady growth over the past decade and has also assumed the water supply needs of neighboring communities. This has resulted in increasingly higher water demands for both baseflow demands in the non-peak season (November through May), as well as higher demands during peak season (June through October). In 1995, the City's peak month and peak day production were 6.5 mgd and 8.3 mgd, respectively. The peak month and peak day demand in 2003 was 9.2 mgd and 10.3 mgd, respectively. Hence, peak demands in the City have increased 2.5% to 3% per year, or over 20% during the past 8 years. Due to these higher demands, the plant has been experiencing some operational challenges which historically have not been an issue, including low production efficiency, increased sludge management problems and higher operating costs. The City recently completed a Water Distribution System Master Plan (West Yost & Associates, January 2001) to address impacts of the growing demands on the distribution system.

The Grants Pass WTP, located at 821 SE "M" Street, was originally built in 1931 with a single basin and three filters for a designed capacity of approximately 3.5 mgd. The plant has undergone several upgrades and expansions through the years to incrementally adjust to a growing population and more stringent treatment standards, including:

- 1950 – Capacity increased to 9 mgd through the addition of second basin and two additional filters.
- 1961 – Minor improvements to treatment process.
- 1983 – Capacity increased to 20 mgd through addition of third basin and three additional filters, construction of a new raw water intake and new chemical feed systems.
- 1995 to 2001 – Filter media and gravel support replaced due to suspected gravel/underdrain upset caused by excessive air in the backwash line.
- 1997 – Filter-to-waste (FTW) added for improved CT-removal credit.
- 1998 –SCADA upgrade; VFD included on BW pump.
- 1999-2000 – Improvements to the Equalization basin pumping station

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- 2001 – Liquid sodium hypochlorite system installed to replace gas system.
- 2001 – Riverbank stabilization adjacent to the intake structure, in cooperation with US Army Corps of Engineers
- 2002 – New PLC-based SCADA system and new monitoring devices were installed in the plant to replace outdated analog transmitters and to allow for more accurate and complete process performance monitoring and automated process control. The PLC replaced obsolete analogue loop-controllers and chemical feed controllers

The 1983 expansion required extensive internal remodeling of the original building as well as bank stabilization around the new intake structure. However, the original structure has been preserved and is currently listed on the American Water Works Association's (AWWA) National Historic Water Landmarks.

The plant draws water from an adjacent intake on the Rogue River. The City has been drawing water from the Rogue since 1888, and currently has a total water right of 82 cfs (53 mgd). The river is prone to turbidity events and yearly fluctuations in temperature and pH which create seasonal challenges to plant operations. The river flow and quality are also influenced by upstream dam operations, most notably the Lost Creek Reservoir and Savage Rapids Dam. The WTP is operated as a conventional filtration plant although it lacks formal flocculation prior to sedimentation in its basins. Solids from the basins, as well as backwash and filter-to-waste water, are transferred to a settling lagoon which overflows to Skunk Creek. Following cleaning in 2000, the lagoon is now full; a long-term solids management plan needs to be developed.

Figure 1-1 is an photographic overview of the City's Water Treatment System; ***Figure 1-2*** provides a plan-view layout of the WTP in its current configuration. ***Figure 1-3*** is a Process Flow Schematic of the plant indicating key processes, chemical addition points and sample locations.

Major facilities and structures at the Grants Pass WTP include:

- Raw water intake and screening facility, including a dual compartment intake structure complete with two stationary bar screens and one traveling screen.
- Raw water pumping station (4 pumps total, all with 75 Hp motors), flowmeter, and 36" static mixer
- One mixing basin (not currently in use) servicing Basins 1 and 2.
- Three sedimentation basins with total surface area of 18,800 square feet and total volume of 1,835,300 gallons.
- Eight mixed media gravity filters (18-22 inches media depth, not including support gravel) for a total of 2,493 square feet of surface area.
- A 433,000 gallon baffled clearwell.
- One 200 Hp backwash pump with VFD, 16" backwash pipeline and flow meter.
- A high service pumping station (5 pumps total, 2 constant speed pumps with 300 Hp motors, one constant speed pump with 250 Hp motor, two VFD pumps with 250 Hp motors).
- One 36-inch finished water transmission pipeline with flowmeter.
- One hydropneumatic surge tank (volume = 11,300 gallon) located on the finished water line.
- Chemical storage, metering and rapid mixing systems for liquid alum (50%), liquid sodium hypochlorite (12.5%), hydrated lime, dry (filter aid) polymer, dry potassium permanganate ($KMnO_4$), and powdered activated carbon (not currently in operation). Alum is used as the primary coagulant, filter aid polymer is added to the basin effluent to improve filter performance. Disinfection is achieved through both pre- and post-chlorination by sodium hypochlorite. Potassium permanganate is used to control taste and odor in the finished water. Lime is used to increase pH which reduces internal pipe corrosion within the distribution system.
- One 116,000 gallon equalization basin for backwash wastewater, filter-to-waste and sedimentation basin wastewater.
- Equalization basin pumping station (3 pumps total, two smaller pumps (30 hp each) with a combined capacity of 2,100 gpm at TDH = 42-feet, and one larger pump (60 hp), rated at 1750 gpm at TDH = 60-feet).

- One sludge lagoon (Medco Mill Pond) which discharges decant/overflow into Skunk Creek and eventually into the Rogue River.

Included in the operations building is a water quality laboratory for treatment process monitoring and control, the plant's electrical distribution equipment, main control board and other instrumentation/control equipment. Also included are office and administrative spaces, a lunchroom, workshop and meeting area.

The plant and raw and finished water pumping stations typically operate between 8 and 15 hours per day depending on system demands. During the peak demand months of July and August, the plant is operated up to 15 hours per day to meet peak day demands. The plant is staffed at all times when operating and employs two and one-half ($2^1/_2$) full-time employees (FTE) and one and two-fifths ($1^2/_5$) maintenance personnel; operators are rotated between the water and wastewater treatment plant, except for the plant supervisor.

This Facility Plan was completed for a number of reasons including:

- Document the existing plant capacity and project the expected remaining useful life,
- Determine required improvements, if any, to meet current and possible future drinking water regulations,
- Determine required improvements, if any, to meet other current or planned future regulations for public facilities,
- Determine improvements to replace or improve existing plant equipment and systems to keep pace with current technology where there is a need,
- Evaluate options to improve the plant's overall production efficiency to help minimize required production time and reduce/optimize operations costs,
- Evaluate options to minimize solids production, improve handling capacity and develop a long-term plan for solids handling,
- Recommend alternatives to increase the plant's capacity in preparation for future water treatment needs.

A list of improvements, categorized and prioritized (according to purpose and relative importance) along with estimated costs, was developed as part of this planning effort. This list of recommended improvements will assist the City in identifying its short-term and mid-term water treatment improvements, allowing the City to prepare for the next 10 years of operation, as well as longer-term improvements that will better prepare the City for 20+ years.

1.2 KEY ISSUES

Key issues to be addressed in the City's WTP Facility Plan are summarized below:

- Regulatory Compliance, including existing and pending water quality regulations, Endangered Species Act (ESA) requirements and National Pollutant Discharge Elimination System (NPDES) permit compliance.
- Treatment Optimization to ensure optimal plant performance, improve overall plant efficiency and to minimize operating costs associated with pumping and chemical usage, as well as sludge production.
- Reliability/Redundancy for primary and subordinate treatment facilities and associated ancillary equipment to ensure reliable plant production.
- Equipment Replacement/Repair for operational and maintenance purposes.
- Possible Capacity Expansion to meet future water treatment needs.

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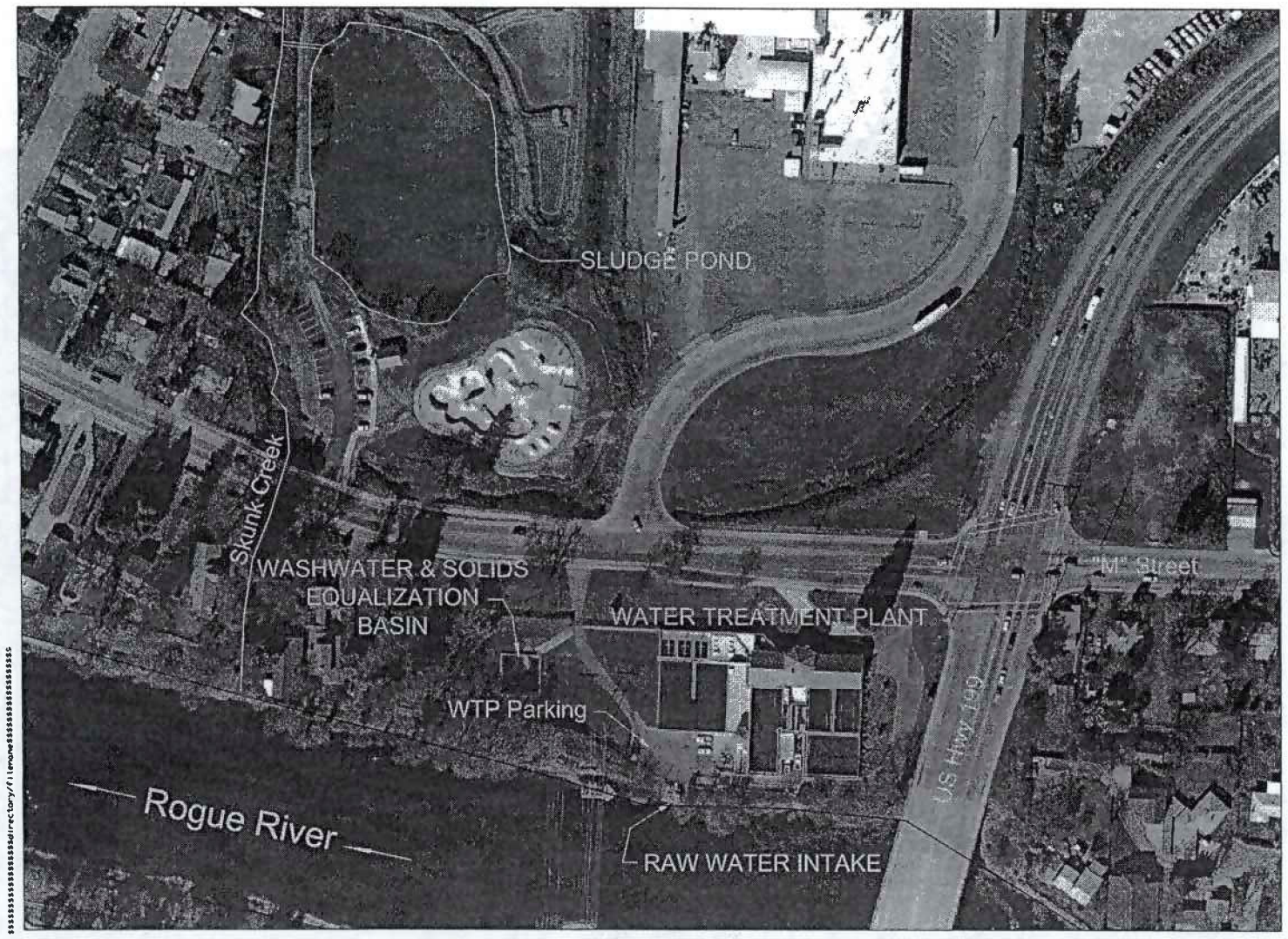

SCALE: 1" = 125' IF
LINE MEASURES 1"

MWH

Figure 1-1
City of Grants Pass
Water Treatment Plant Facility Plan
System Overview

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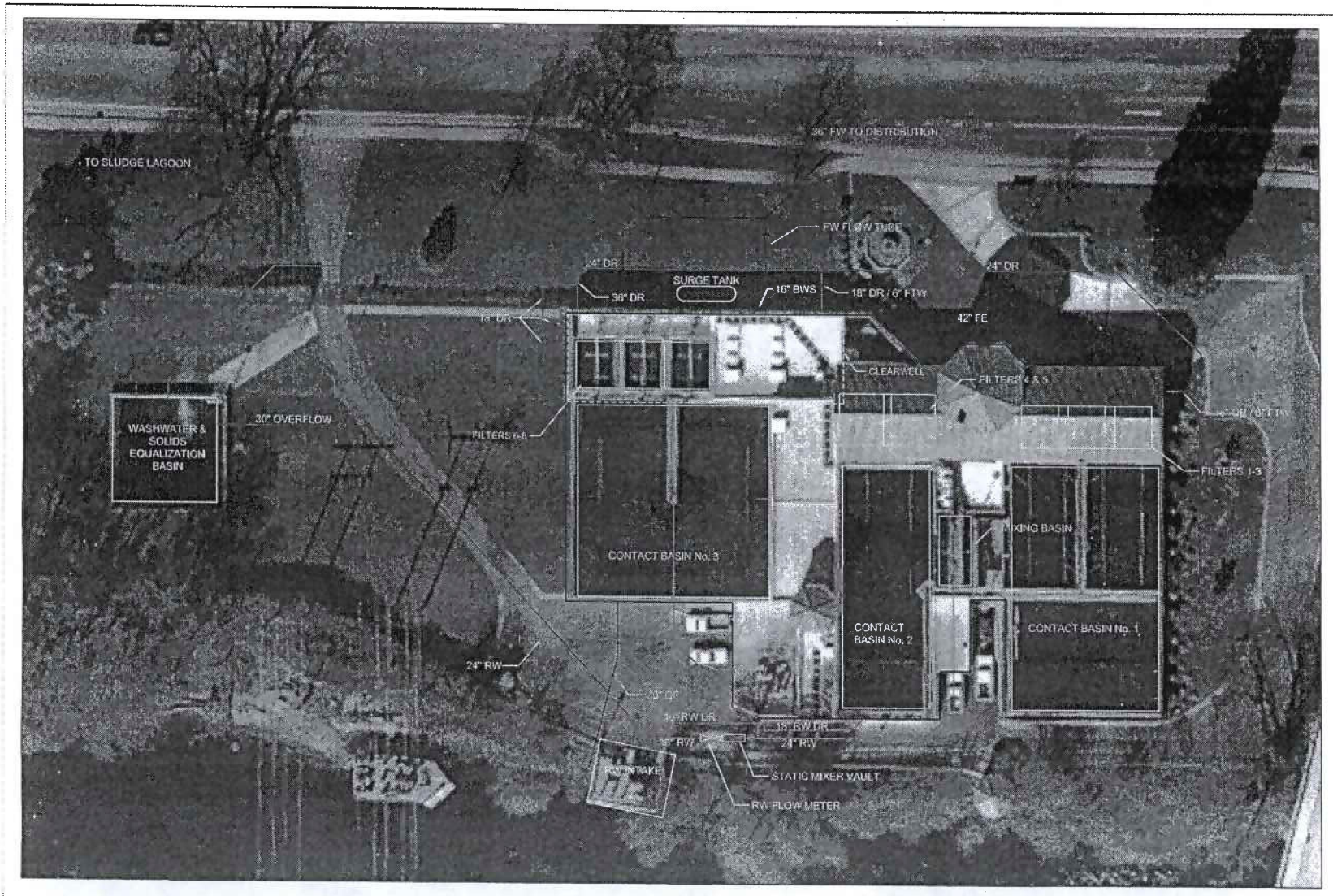

LEGEND

BUILDING OUTLINE ——

MAJOR PROCESS COMPONENTS

FW PIPING ——

RW PIPING ·····

WASTE/OF PIPING ——

SCALE: 1" = 40' IF LINE MEASURES 1"

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Figure 1-2
City of Grants Pass
Water Treatment Plant Facility Plan
Plan-view Layout

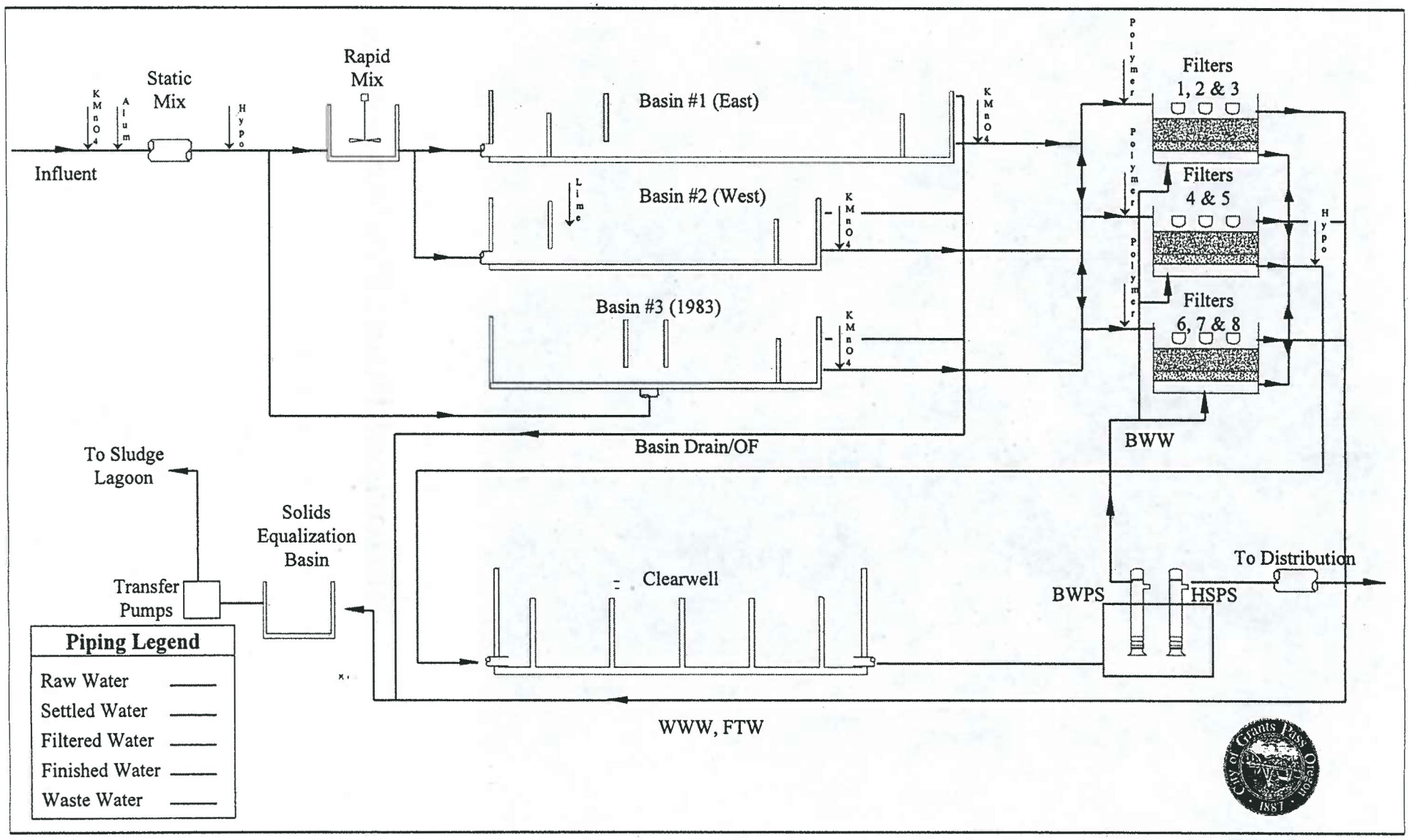

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Figure 1-3
City of Grants Pass
Water Treatment Plant Facility Plan
Process Flow Schematic

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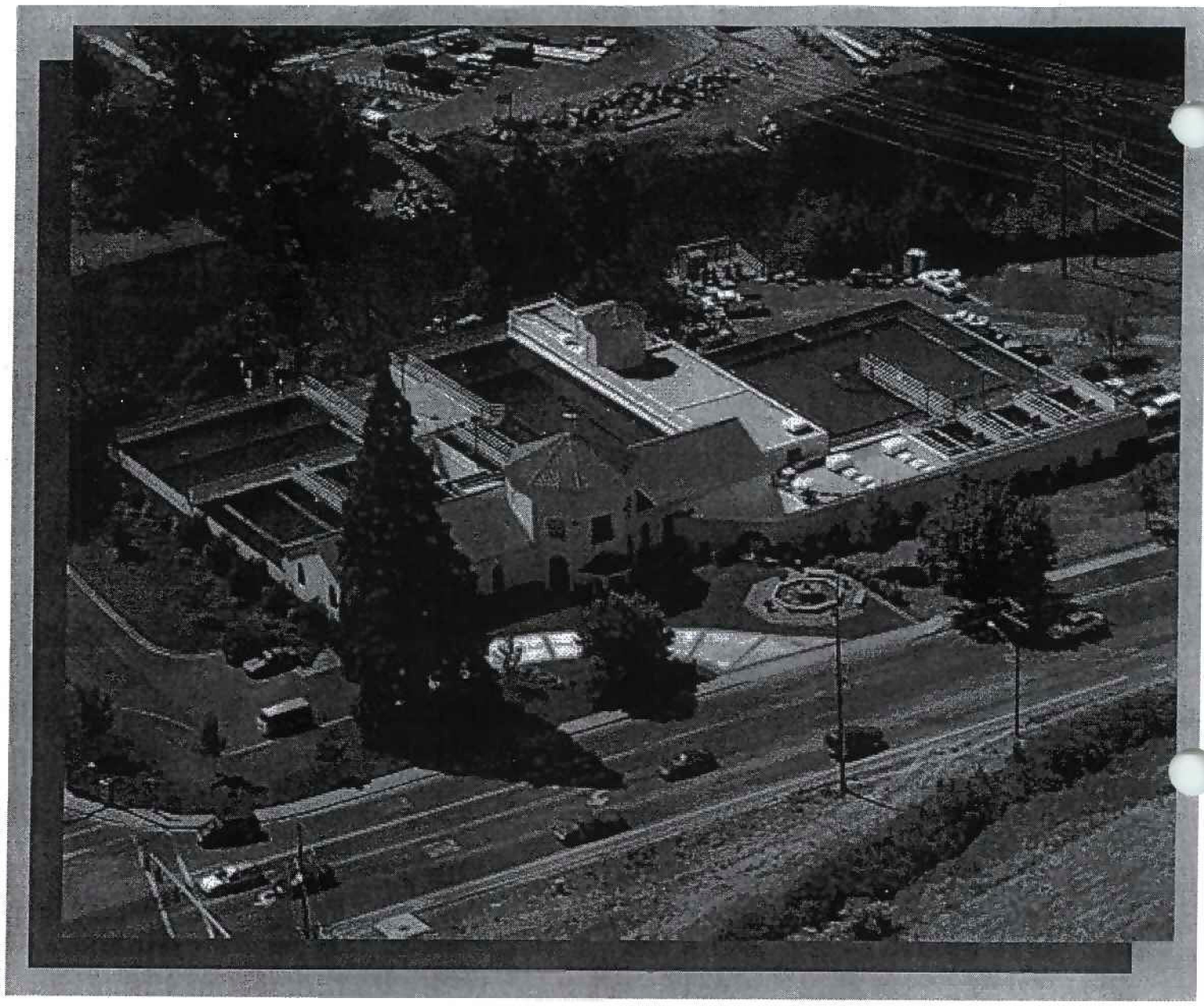

Historical Plant Performance

2 HISTORICAL PLANT PERFORMANCE

Historic operating data for the Grants Pass WTP are reviewed and analyzed in this section of the report. The purpose of this data review is to assist in determining the performance of the existing WTP processes for operational efficiency and regulatory compliance.

All available information relevant to the plant's current condition and performance was reviewed for this evaluation. Plant performance data dating back to January 1995 was provided by the City, however, this performance review focuses on more recent data. Four and one-half years of data and information, from January 1999 to July 2003 were reviewed, including plant flow information, selected raw, finished and distribution system water quality parameters, basin performance, chemical usage data, and overall filter performance indicators. Discussions with plant operators were used to supplement and verify this information.

2.1 PLANT FLOW

The Grants Pass WTP measures and records raw and finished water flows through the plant on a daily basis. Raw water flow is measured using a differential pressure type flowmeter located on the influent line prior to chemical addition. Finished water flow is measured using another differential pressure type flowmeter located on the WTP effluent line just downstream of the HSPS. Backwash flowrate is measured in the backwash supply line, but backwash flows were not recorded consistently prior to the SCADA improvements in 2002. Filter-to-waste (FTW) flows are discharged upstream of the filter effluent flow meters, and therefore have not been historically measured or recorded since the installation of FTW in 1997.

There was a significant increase (approx. 3%) in recorded values for flowrates (both raw and finished water) between 2001 and 2002 coinciding with, and possibly resulting from, the installation of the new SCADA system. As part of the SCADA upgrade, the analog

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system was rerouted, eliminating several analog signal converters on the influent and effluent flow meters. It is the opinion of the plant staff that the old signal converters may have inadvertently dampened the flow signal, reducing the measured value through each of the flowmeters by as much as 10% (compared to current SCADA readings), though validation of this theory is no longer feasible. Fortunately, this possible discrepancy does not impact the analysis of historical production efficiencies, as both the raw and finished water flowmeters were likely impacted proportionally. However, historical calculations of chemical dosage and system demands, which are dependent on raw water and finished water flow measurements, respectively, may be slightly inaccurate; estimates of chemical usage prior to installation of the SCADA system in 2002 may be 10% higher than actual reported dosages.

2.1.1 Plant Production

Figure 2-1 presents the historic average daily raw water flows and finished water flows from January 1999 to December 2003. ***Table 2-1*** presents a summary of this data, including annual average flow, average peak and off-season flows, minimum and maximum monthly average flows and maximum weekly and daily flows. The City has been experiencing increasing water demands over the past decade. Average day production has increased approximately 2 percent per year since 1999 (from 4.5 mgd in 1999 to 4.9 mgd in 2003). This increase may result from differences in measured flows through the plant before and after the SCADA improvements in 2002. A maximum peak day flow from the Grants Pass WTP of 10.5 mgd was observed on July 1, 2002. The highest average maximum monthly flow of 9.2 mgd was observed in July 2003. Increasing demands can be attributed to steady growth in the area, in addition to the City's recent incorporation of the urban growth boundary previously not served. As previously mentioned, the transition from the old analog transmitters to the new SCADA system may be partially responsible for the apparent increases in demand.

The flow data presented in ***Table 2-1*** was used to develop peaking factors that are useful in water supply planning efforts. The primary peaking factor is the ratio of peak day flow

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TABLE 2-1: SUMMARY OF WTP PRODUCTION

WTP PRODUCTION (MGD)											
Year	Annual Average	Peak Season[1] Average	Off Season[2] Average	Minimum Monthly Average		Maximum Monthly Average		Maximum Weekly Average		Maximum Daily	
				Month	Value	Month	Value	Dates	Value	Date	Value
1999	4.5	6.3	3.3	FEB	2.8	JUL	7.8	7/28-7/3	8.3	7/28	9.4
2000	4.5	6.4	3.2	JAN	2.8	AUG	7.8	8/2-8/8	8.5	7/31	9.7
2001	5.0[3]	6.6	3.6[3]	DEC	2.7	AUG	7.7	8/5-8/11	8.3	8/10	9.2
2002	4.9	7.3	3.3	JAN	2.7	JUL	8.9	7/10-7/16	9.4	7/11	10.5
2003	5.1	7.3	3.1	JAN	2.7	JUL	9.2	7/24-7/30	9.6	7/28	10.3

[1]Peak Season is June through October

[2]Off Season is November through May

[3]No data available for January 2001; calculated based on February through December data, and likely overestimates average values

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to annual average flow; this value ranged between 2.09 in 1999 to 2.14 in 2002. Another important peaking factor is the ratio of peak month flow to annual average flow. For the City, this value ranged from 1.73 in 1999 to 1.75 in 2002. These values are consistent with those used for demand forecasting in the City's Water Distribution System Master Plant (West Yost, 2001), where peaking factors of 2.2 and 1.8 were used for peak day and peak month, respectively. Additionally, based on recent studies, maximum day peaking factors for systems in the Pacific Northwest typically vary from approximately 2.0 to 2.5. The peaking factors for the City system are consistent with these regional numbers.

2.2 Raw Water Quality

Four raw water quality parameters were analyzed: turbidity, temperature, pH, alkalinity and organic content. These parameters are typically of most importance when evaluating a treatment plant's overall performance.

2.2.1 Turbidity

Raw water turbidity is probably the single most important water quality parameter when evaluating plant performance and alternative process design criteria. Turbidity is a measure of light penetration through a water sample and is indicative of the relative amount of particulate matter in the sample. Water with lower turbidity is typically easier to treat and usually requires lower chemical doses for optimum coagulation and filtration. High turbidity levels can reduce the effectiveness of disinfection treatment processes and can provide a medium for the growth of microorganisms.

The raw water turbidity from the Rogue River has historically been low and moderately variable during the majority of the year. High rainfall events generally correspond to an increase in River turbidity. Additionally, dam operations also affect turbidity in the River. ***Figure 2-2*** presents the average daily raw water flow rates, turbidity, as well as the observed daily precipitation between January 1999 and July 2003. The lowest turbidity periods occur during the warmer, drier months and the highest turbidity periods occur during the wet weather months.

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Average turbidities were generally less than 5 NTU from May to October; minimum turbidities were as low as 1.0 NTU during these months. Between September and April, average turbidities were typically 8 NTU, with average maximums approaching 200 NTU. The highest average day raw water turbidity was reached in December 2001 when average daily turbidities of 176 NTU were observed in the raw water. Raw water turbidities approaching 1,000 NTU were recorded during the winters of 1995 through 1997 according to plant staff.

2.2.2 Temperature

Temperature plays an important role in water treatment because it affects the rate of chemical reactions (including disinfection), floc settling and filter performance. Higher temperature water typically requires lower chemical doses and offers better floc formation, settling, filtration and disinfection characteristics. An increase in optimal filter backwash rates also results from an increase in water temperature due to the decreased viscosity of the warmer water.

The temperature of the raw water entering the WTP varies by season, as shown in ***Figure 2-3***. During the 4½ year period of record considered for this evaluation, wintertime low average temperatures were approximately 45°F (7°C) and summertime high average temperatures were approximately 61°F (16°C). The lowest observed temperature was 40°F (4.4°C) in February 2002. The highest observed temperature was 74°F (23°C), measured in July 2001.

2.2.3 pH

pH is a measure of the acidic or basic nature of a water sample and can also be indicative of whether or not a water is corrosive. A pH of 7.0 represents neutral conditions, and pH values in excess of this are considered acceptable for corrosion control. pH values less than 7.0 usually indicate corrosivity, which can lead to leaching of toxic metals into the water system and degradation of conveyance facilities. pH is also important in water treatment because of its impacts on coagulation performance and chemical disinfection. A pH in the range of 6.5 to 7.0 is considered optimum for alum coagulation and for chemical disinfection. In plants lacking ability to adjust pH at several points throughout

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the treatment process, corrosion control typically governs the pH, with some sacrifice in coagulation and disinfection performance.

Figure 2-4 presents the historical raw water pH values between January 1999 and July 2003; trendlines have been included to help highlight seasonal variability in pH. As shown in the figure, the pH of the raw water from the River typically varies between 7.3 and 8.0 throughout the year, with average values between 7.5 and 7.9. Historically, pH peaks twice each calendar year with the most pronounced peak occurring in the mid-spring and a secondary peak occurring in the early fall, corresponding to algal activity in the river. Historic minimums occur in the winter months, presumably due to heavy rainfall events. The lowest observed raw water pH was 7.30 in June 2000. The highest observed pH was 8.50 in March 2001. pH is also affected by algae throughout the summer, with diurnal swings that can vary between 7.5 to 8.5.

2.2.4 Alkalinity

Alkalinity is important in water treatment because of its impact on coagulation performance as well as its impact on corrosivity and pH stability. Alkalinity above 20 mg/L as $CaCO_3$ is generally considered adequate for alum coagulation and improved pH stability in the distribution system. Alkalinity can also impact TOC removal requirements, depending on raw water organic concentrations.

Alkalinity is not measured regularly at the Grants Pass WTP; however, some data was collected from 1999 to 2003. Raw water alkalinity typically ranges from 30 to 45 mg/L as $CaCO_3$. The highest observed alkalinity was 49.3 mg/L as $CaCO_3$ in April 2001. Raw water alkalinity has not been measured with enough frequency to establish seasonal alkalinity trends, however, it is expected that alkalinity would decrease in the winter (corresponding to the rainy season) and increase in the summer.

2.2.5 Organic Content

The natural level of organic matter in the raw water can affect its treatability as well as other parameters, including chlorine demand and disinfection by-product (DBP) formation and taste and odor. Organic content can be derived from the natural decay of

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plant life, as in humic and fulvic acids, or the presence of algae. As the concentration of organic matter in the water increases, the requirement for chemicals that react with the organic matter (alum and chlorine, for example) also typically increases. Since DBPs result from chlorine's reaction with organic matter, higher concentrations of organic matter in raw water usually result in higher levels of DBPs in the distribution system. Elevated algae concentrations can sometimes create difficult treatment conditions such as interference with coagulation, filter clogging and nuisance tastes and odors, depending upon the type and concentration of the algae.

Total Organic Carbon (TOC) is a general measure of the natural organic matter (NOM) present in the raw water. This parameter is sometimes used as an indicator of DBP formation potential. TOC is also important as existing regulations intended to minimize DBP formation require the removal of a fraction of the overall raw water TOC through the treatment process, depending on the raw water TOC concentration and alkalinity.

The Grants Pass WTP staff recently began a monitoring program for to determine TOC concentrations in the raw and finished water. Quarterly TOC sampling was performed throughout 2001; monthly sampling was performed throughout 2002. Results from this sampling effort are presented in ***Figure 2-5***. The data suggest that the TOC concentrations in the raw water are comparable to other U.S. surface water supplies, typically ranging between 0.5 to 5 mg/L, and slightly higher than other similar Pacific Northwest surface water supplies, which range between 0.5 to 3.0 mg/L. Five samples taken between November 2001 and March 2002, measured concentrations of TOC above 2.0 mg/L, the current "trigger" concentration for TOC removal requirements under existing regulations. Further discussion of required TOC removal efficiencies and other regulatory issues associated with TOC are discussed in ***Section 3-Regulatory Review***. More data is required to better understand the seasonal variability of TOC in the raw water. Grants Pass should continue to monitor raw TOC on a monthly basis. Settled and/or finished water TOC should also be monitored to demonstrate TOC removal through the basins and through the plant.

Because TOC analysis is expensive and labor intensive, the City should consider purchasing a bench-top ultraviolet (UV) spectrophotometer, and incorporating daily UV absorbance monitoring at the WTP as a surrogate for TOC. Dissolved and soluble organic carbon absorbs UV light at a wavelength of 254 nm; a spectrophotometer measures the percentage of UV absorbance, a value directly proportional to TOC. Once calibrated, UV_{254} readings can be correlated to TOC concentrations. UV_{254} sampling will a relatively inexpensive, simple and accurate alternative to lab analyses of TOC.

2.2.5.1 Taste and Odor

According to plant staff, the Rogue River experiences occasional seasonal taste and odor events during the warmer summer months (August and/or September). Rigorous monitoring of these events has identified the source as geosmin, a naturally occurring organic compound resulting from algae metabolism. Geosmin is capable of imparting an objectionable odor at very low concentrations (0.010 ug/L); geosmin levels below 0.008 ug/L are considered acceptable.

Figure 2-6 presents results of geosmin sampling along the Rogue River, downstream of the Lost Creek Reservoir, performed by the Medford Water Commission. As shown in the figure, concentrations of the compound decrease downstream of the reservoir, likely resulting from tributary dilution. The Medford Water Commission recently installed pre-ozonation to address seasonal taste and odor events. Though concentrations in Grants Pass may be considerably lower than those measured upstream, treatment provisions for taste and odor causing compounds may still be warranted at the WTP. Plant staff have received several customer complaints during "heavy" taste and odor events in the river, but most taste and odor complaints are usually due to chlorine.

2.3 CHEMICAL USAGE

Chemical usage at the Grants Pass WTP was analyzed to determine any seasonal trends that may offer insight into the overall treatment process performance. The five major chemicals currently used at the plant are aluminum sulfate (alum), filter aid polymer, hydrated lime, liquid sodium hypochlorite, and dry potassium permanganate. Liquid

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alum is used as the primary coagulant. The polymer is used to condition the water entering the filters for improved filter performance. Lime slurry is currently added to the settled water leaving Basin #2 to increase the pH for corrosion control. Sodium hypochlorite is added to the raw water and finished water as a disinfectant, and potassium permanganate is added to the raw water and to two of the three sedimentation basins to control taste and odor.

2.3.1 Alum

Liquid alum is stored as a 50 percent solution (by weight) and fed via metering pump to the raw water pipeline upstream of the static mixer, prior to the flow split to the basins. The addition of alum to the raw water destabilizes (neutralizes) negatively charged suspended particles, thereby allowing the formation of insoluble floc particles via coagulation and flocculation, and their subsequent removal via sedimentation and filtration. The alum feed is continuous using carrier water; the carrier water flow rate is estimated at 15 gpm. Alum dose is manually adjusted based on raw water turbidities, pilot filter turbidities, previous experience and results from jar tests. On average, alum is diluted approximately 40:1 with carrier water, resulting in an alum concentration of approximately 1.25% in the chemical injection stream. Mixing occurs through an in-line, 36-inch diameter static mixer, downstream of the chemical addition vault.

Figure 2-7 shows the annual trends in alum usage between January 1999 and July 2003. The required alum dose varies throughout the year; typical fall and winter alum doses average 25 mg/L (as dry alum) while spring and summer alum doses average 17 to 18 mg/L (as dry alum). The highest alum doses are typically above 50 mg/L (as dry alum) in the fall and winter because of high turbidity events. The minimum daily alum dose varies slightly throughout the years, ranging from 13 mg/L to 20 mg/L (as dry alum) between June and October.

These alum doses are considered relatively high, especially when the river turbidity is very low (1 to 2 NTU) during most of the summer. Alum is known to produce floc which is less resistant to shear and retention within filter media, and does not settle as

well as other coagulant flocs. Also, alum does not perform as well during colder water conditions as the floc takes longer to form. Alum sludge does not dewater as easily as other chemical sludge. While the use of alum as the primary coagulant has historically been effective in producing good-quality water, there are concerns that continued use may not be able to meet performance expectations (i.e. low sludge production, long filter run lengths) as the plant production demands increase. Higher alum doses also increase solids production, exacerbating solids management issues at the plant.

2.3.2 Polymer (Filter Aid)

The Grants Pass WTP currently uses a nonionic polymer (Magnifloc 990N) as a filter aid. The dry polymer is mixed and aged with water, then fed via metering pump and carrier water to the filter influent; flows are split 8-ways to each filter using rotameters. Filter aid polymer is used continuously throughout the year and total daily usage is monitored and recorded. The polymer's role in improving overall turbidity removal at the Grants Pass WTP is important. When introduced to the settled water, the polymer helps make the alum floc that carries out of the sedimentation basins "stickier". This property helps the filters retain the floc better and minimizes turbidity "breakthrough". If the filter aid were not added, the filtered water turbidity would be higher, and filter run lengths significantly shorter due to premature breakthrough (i.e. the filters would have to be backwashed more frequently).

As previously discussed, alum floc is known to be fairly weak in terms of its resistance to the shear forces typically found within a filter. A weak floc will not be retained well within filter media, resulting in turbidity "leakage" and premature turbidity breakthrough. Its shear resistance also decreases with lower water temperatures. Consequently, the need for filter aid polymer would be expected to increase in the winter and decrease in the summer, typical of many plants using alum as a primary coagulant.

Figure 2-7 presents the historic average daily filter aid polymer dosages from January 1999 through July 2003. Filter aid polymer dosages tend to increase in the winter when water temperatures are low and decrease in the summer and early fall when the water is

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warmer. The average daily polymer dose was 0.025 mg/L during the summer, increasing to approximately 0.050 mg/L in the winter and as high as 0.20 mg/L during winter's most challenging raw water conditions.

2.3.3 Lime

Lime is used to raise the pH by restoring alkalinity consumed through the coagulation process; plant staff maintains a target finished water pH of 7.2 for corrosion control. Hydrated lime is stored as a dry powder, and fed through a hopper to a chemical mixing tank; lime slurry is then fed to the settled water in Sedimentation Basin No. 2 prior to filtration. Increases in turbidity require an increased alum dose, resulting in a more acidic treated water. Lime can restore the alkalinity consumed during these events and maintain treated water pH in a range optimum for corrosion control. However, depending on the point of addition, lime can negatively impact treatment plant performance. Both coagulation and disinfection performance improves in lower pH ranges; adding lime prior into the sedimentation basin effluent may increase settled water turbidities and decrease disinfection of microbes.

Figure 2-8 shows average daily lime usage for pH adjustment from January 1999 through July 2003. As with other chemical additions, there is a noticeable seasonal trend in lime dose. Lower lime dosage are generally required in the summer months; no lime was used at the plant during the summers of 1999, 2000 and 2001; lower "baseline" doses of approximately 2.5 mg/L (as $Ca(OH)_2$) were maintained during the summers of 2002 and 2003. Lime addition throughout the winter months typically range between 2.5 to 10 mg/L, with maximums in excess of 20 mg/L. Higher doses are typically required in the winter months due to increased alum doses and decreased alkalinity in the raw water.

During the plant tour conducted on July 28th, 2003, all lime required for pH adjustment was being added near the effluent of Sedimentation Basin #2. Local pH in this region exceeded 9.0. Impacts of this chemical dosing strategy on finished water quality are discussed later in this section.

2.3.4 Sodium Hypochlorite

Liquid sodium hypochlorite (12.5% solution) is stored in three 2,120-gallon fiberglass tanks located on site. The hypochlorite system was installed in 2001 to replace the original gas chlorine injection system. Hypochlorite is added to the raw water ("pre-chlorination") to assist in coagulation, control biological growth through the sedimentation basins, and for disinfection purposes. Chlorine addition to the finished water ("post-chlorination") is intended for disinfection purposes and is added to maintain a chlorine residual in the distribution system. Chlorine is "boosted" throughout the distribution system (up to three times for some parts of the system) for residual maintenance. The operator-adjustable target chlorine residual entering the sedimentation basin was increased in February 2003 (from 0.4 mg/L to 1.0 mg/L free chlorine) to ensure a 0.5 mg/L residual is maintained throughout the basins. Prior to February 2003, a target dose of 0.5 mg/L was typically used, though this target had slight seasonal variations to account for changes in raw water quality and system demands (i.e. detention times). Chlorine residual at the effluent of the sedimentation basins was not measured prior to February 2003.

Figure 2-9 shows the free-chlorine residual in the treated raw water following chemical addition and rapid mixing by the 36-inch static mixer (pre-chlorine dose), as well as the free-chlorine residual in the finished water effluent following post-chlorination. Pre-chlorination dose has typically ranged from 0.2 mg/L to 1.4 mg/L, although this range represents changes in operational strategy as well as fluctuations caused by normal operation. Through recent sampling, plant operators observed that the chlorine residual entering the filters was often very low or undetectable. This observation has led the plant to increase pre-chlorination doses to improve disinfection through the plant; this recent increase is evident in ***Figure 2-9***. Finished-water chlorine residuals are generally maintained between 0.9 mg/L and 1.4 mg/L with an average of approximately 1.1 mg/L.

2.3.5 Additional Chemicals

In addition to the primary treatment chemicals used daily at the Grants Pass WTP, the plant also has the capability to dose potassium permanganate ($KMnO_4$) for taste and odor

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control. Though extensive research suggests that oxidation of severe taste and odor compounds (i.e. MIB and geosmin) with potassium permanganate is relatively ineffective, there is some anecdotal evidence that chemical oxidation may be effective on a case-by-case basis (*Identification and Treatment of Tastes and Odors in Drinking Water*, AWWA,1987). Permanganate has proven to be effective in oxidizing "minor" taste and odor compounds, depending on the species.

Low, variable doses of permanganate were used consistently from January 1999 through July 2003. Permanganate is dosed via metering pump to two addition points, one located in the static mixing vault prior to the flow split to Basin #3, the second in the mixing basin upstream of Basin #1 and #2, thereby limiting the concentration of permanganate in Basin #3. This chemical dosing strategy was developed in response to short-circuiting leading to permanganate carryover in Basin #3. The permanganate dose is adjusted on a visual basis to maintain a pink hue through the first baffle of the mixing basin. The average daily permanganate dosages for this period are shown in ***Figure 2-8***; actual doses in Basin #1 and #2 will be slightly higher, and in Basin #3 slightly lower than the averages presented in the Figure. The dosage of permanganate peaks in the winter months with increasing turbidity. Typical permanganate doses ranged from 0.3 mg/L to 0.5 mg/L (as $KMnO_4$). These doses are considered high for control of taste and odors, and may lead to manganese oxide deposits in the filter media and distribution pipelines. Based on preliminary recommendations of this plan, the permanganate dose was lowered in June 2003 to approximately 0.06 to 0.10 mg/L.

Originally, the plant was designed to dose powdered activated carbon (PAC) for an additional taste and odor control process. However, this system has been disconnected and is no longer used.

2.4 PLANT PERFORMANCE DATA

The WTP staff keeps daily records of plant performance data that were used to assist in the evaluation of overall plant performance. This section summarizes the historic operating performance of the treatment processes including the sedimentation basins, and

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filters. It is important to remember that the coagulation, flocculation/sedimentation and filtration processes are not independent of each other, but rather they are dependent on each other in terms of evaluating overall plant performance.

2.4.1 Coagulation Performance

The Rogue River water quality presents some treatment challenges at the WTP, resulting from wide swings in pH (seasonal as well as diurnal), seasonally variable turbidity, temperature, and color, as well as occasional taste and odor events. Excepting taste and odor, this variable raw water quality can significantly impact coagulation performance at the plant. Historically, these challenges have been met using a relatively high dosage of alum. This strategy has resulted in perhaps unnecessarily high solids production (putting a "stress" on the existing solids handling facilities), depressed pH (corresponding to an increase in pH adjustment chemical usage/costs), and decreased overall plant efficiencies; each of these issues is discussed in detail later in this report. Improvements to the filters and/or basins may serve to improve overall plant efficiencies. However, without these improvements, continued use of alum as the sole, primary coagulant may not be sufficient to meet performance expectations (i.e. minimal solids production, long filter run lengths) as the plant production demands increase. Alternative coagulation strategies for the City's WTP are discussed in **Section 4**.

2.4.2 Sedimentation Basin Performance

The City's WTP relies on three Sedimentation Basins for flocculation and some sedimentation, prior to filtration; no formal flocculation (mixing) is provided in the basins. Basin #1 was constructed as part of the original plant; Basin #2 and #3 were incorporated into the plant during the various plant expansions. Therefore, the design (and effluent water quality) differs between basins. Raw water flow is split into two pipes downstream of the static mixer; the first pipe leads to a slow mix basin for Basins #1 and #2, the second leads to Basin #3. Each pipe has a butterfly valve for flow control. However, the flowmeter installed in the pipeline during the plant expansion prior to the Basin #3 inlet is not currently in operation and is in need of repair. A gate valve located at the influent to the slow mix basin is also used to control flow. The pipes/valves were designed to split the plant flow proportionally to each basin, based on the basin's settling

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area, or 36%, 24% and 40% of plant flow to Basin #1, #2 and #3, respectively. However, short-circuiting has mandated that flows through Basin #3 be reduced. Additionally, the valves controlling flow split through the basins were set based on maximum flow (approximately 20 mgd with 4 pumps on). Therefore, unless the valves are manually adjusted, the percentage of flow to each basin varies at lower plant flowrates.

The slow mix basin upstream of Basin #1 and #2 has two compartments; the mixers installed as part of the original design have been removed. The water level in these basins is also very high, minimizing the head available for mixing. Flows from the slow mix basin are proportioned between Basin #1 and #2 using mud valves located on the end of each influent channel. Basin #1 and #2 are also equipped with interior baffling walls to ensure laminar flow through the sedimentation zone. Basin #1 has two baffle walls, Basin #2, only one.

Each of the Sedimentation Basins has several chemical application points. Lime slurry and potassium permanganate can be added in the slow mixing basin (influent to Basins #1 and #2). Lacking a mixing vault, all chemical injection for Basin #3 must occur in the static mix vault prior to the flow split. During the plant tour conducted on July 28, 2003, permanganate was being added in the static mix vault and at the slow mixing basin; all lime for pH adjustment was being added near the effluent launders in Basin #2, a procedure not commonly practiced at most WTPs due to the impacts on floc formation.

Water flows from the Sedimentation Basins to the filter influent. The settled water trough is continuous between the filters and is intended to allow water from each sedimentation basin to spread evenly between the filters. Isolation valves are installed to allow cleaning. In general, Filters 1-3 are fed by Basin #1, Filters 4 and 5 by Basin #2 and Filters 6-8 by Basin #3. Because Basin #3 is further from Basins #1 and #2, requiring a longer pipe connection, the amount of water mixing and sharing between Basins #1 and #2, and Basin #3 may be somewhat restricted.

The basins are each drained and cleaned twice per year. Cleaning is restricted to off-peak seasons, as the plant requires the full capacity to meet summer demands. As solids accumulate in the basins, the detention time decreases, probably reducing the solids removal and disinfection performance of the basins. A summary of basin design criteria is presented in ***Table 2-2***.

TABLE 2-2: BASIN DESIGN CRITERIA

Parameter	Basin 1	Basin 2	Basin 3
Width x Length (ft)	61 x 98	38 x 98	80 x 80
Avg. Water Depth (ft)	13	13	13
Surface Area, total (sf)	5,980	3,750	6,400
Total Volume (gal)	581,600	364,700	622,400
Nominal Rated Capacity (mgd)	7.2	4.8	8.0
Length:Width Ratio	1.6:1	2.6:1	1:1
Length:Depth Ratio	1:7.5	1:7.5	1:6.2
Mean Flow Velocity (ft/min)	0.84	0.90	0.71
Overflow Rate at Nominal Capacity (gpm/sf)	0.84	0.89	0.87
Theoretical Detention Time at Nominal Rated Capacity (20 mgd) (min)	116	109	112

Basins #1 and #2 are rectangular basins. Water enters at the south end of the basin. Laminar flow conditions are improved via two baffle walls, one at the inlet, the second approximately half way along the length of the basins (in Basin #1 only). Basin effluent collects in launders located on the north end of the basins. Sedimentation Basin #3 is the newest basin in the plant, built in 1983. Water enters this basin via a central 36-inch vertical pipe that discharges through ports located from 3 to 5.5 ft below the water surface. The water then flows under a circular 20-ft-diameter baffle that extends from just above the water surface to 8 ft below. Water exits from the basin into one continuous square launder located 10 feet inside of the basin walls on all sides. Water from this square launder collects in a common trough that flows to the filter influent trough. There are no automated solids removal mechanism installed inside any of the basins, though provisions for future upgrades were included in the design of Basin #3.

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Though the Sedimentation Basins were not designed for optimal flocculation or settling, the basins do provide effective removal of solids under most operating conditions. An optimal sedimentation basin is rectangular with a minimum length to width ratio of 4:1, a minimum length to depth ratio of 1:15 and a sufficient volume to keep mean flow velocity under 3.5 ft/min. Optimal basins provide approximately 20 to 30 minutes of flocculation and 90 to 120 minutes of sedimentation or a total of 120 to 150 minutes of detention time. Baffles are also recommended to ensure good flow distribution and prevent short-circuiting (Kawamura, 2001). Based on these criteria, it is expected that Basins #1 and #2 will remove more solids than Basin #3. With its square shape and radial flow, Basin #3 is vulnerable to short-circuiting, despite the large volume of the tank, the path length from inlet to outlet is relatively short. Also, when the hydraulic radius is large, as in Basin #3, stable flow is difficult to maintain.

Figure 2-10 presents the Sedimentation Basin performance between March 2002 and June 2003, between 9:00 a.m. and 3:00 p.m. (since the SCADA system was brought on-line); trendlines have been included in the figure for clarity. This selection of data was used to better represent operational conditions in the basins and minimize start-up/shut-down impacts on settled water turbidity. Normal operating hours are between 7 a.m. and 10 p.m. during the peak season, and 7 a.m. and 5 p.m. during off-peak season. As shown in the figure, Basin #1 consistently provides the highest water quality (i.e. lowest turbidities) throughout the year, Basin #3 the poorest. However, all basins struggle to maintain optimal water quality (≤2 NTU, currently proposed as target for future settled water turbidity requirements by the EPA), for filtration during the winter months when raw water turbidities are elevated. ***Figure 2-11*** presents a probability distribution of basin effluent turbidities, in addition to the raw water turbidities. In general, settled water turbidity <2 NTU is considered optimal for filter performance, and <4 NTU is considered acceptable for shorter durations. Sedimentation Basin #1, #2 and #3 provide <2 NTU water quality 70%, 55% and 30% of the time, respectively, and <4 NTU water quality 94%, 90% and 86%, respectively. All basins experience difficulties (settled water >4 NTU) when raw water turbidities exceed 10 NTU, which is common for this type of plant

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without formal flocculation preceding sedimentation and less than optimal sedimentation time an/or basin geometry. We have audited several plants in the Pacific Northwest with similar design characteristics; all have experienced similar treatment challenges during high raw water turbidity events. This increase in solids loading onto the filters typically results in increased backwash rates, shorter filter runs and lower overall plant efficiencies.

During the July 29, 2003 plant visit, raw water flow rates were between 15 and 20 mgd with basin effluent water qualities were 0.8 NTU, 1.1 NTU and 2.0 NTU for Basins #1, #2 and #3, respectively. Raw water turbidities during the visit were between 1 and 2 NTU, raw water temperature was approximately 68°F (20°C) and the alum dose was approximately 18 mg/L (as dry alum). All basins were relatively "full" of solids (6-8 feet), minimizing the effective volume of the basin required for solids removal. In all basins, large (potentially settable) floc was overflowing into the launders. The size and nature of the floc was fairly uniform from basin to basin with the exception of Basin #2 in the vicinity of the lime addition. In this section, significantly smaller floc was observed, likely resulting from the localized high pH zone. It was also noted that at 20 mgd, the launders in Basin #2 exhibited an oscillating motion propagated by surface waves in the basin (a problem previously corrected in Basin #1). The oscillation was measured to be less than 1 mm (from center) at the top edge of the launder, however the surface waves generated by this motion potentially disrupt laminar flows in the basin, diminishing basin performance. This problem could be addressed by installing cross supports to the launders.

Overall, the sedimentation basins provide satisfactory water for filtration during most of the year, as evident by adequate filtered water turbidities (discussed later in this report). All basins experience challenges with regard to short-circuiting (impacting solids removal and disinfection efficiencies), high solids loading (resulting from relatively high alum dosages), sub-optimal flocculation and seasonal turbidity spikes. The basins are not equipped with any type of on-line solids removal system; as solids accumulate in the basin, the effective volume of the basin is reduced, compromising flow characteristics

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and overall performance in the basin until solids are removed. Without having continuous sludge removal in the basins, bi-annual cleanings of the basins create large "slug" doses of solids to the equalization basin and to the lagoon, increasing the chances for NPDES permit violations.

2.4.3 Filter Performance

The plant has 8 mixed-media gravity filters of varying sizes and shapes, depending on the time of construction. Filters 1, 2 and 3 (also called the East Filters) were constructed in 1931 as part of the original construction. Filters 4 and 5 (also called the West Filters) were constructed as part of the 1950 plant expansion. The newer filters, Filters 6, 7 and 8, were added as part of the 1983 expansion project. It is uncommon for a WTP to have variable filter shapes as demands on the filter support systems common to all filters (i.e. backwash pump, surface wash pump, washwater conveyance system, etc.) will vary according to the filter surface area. The filters are operated by rate of flow control; butterfly valves on individual filter effluent pipes modulate to maintain a specific filtration rate. Overall filter flow is adjusted to maintain a constant water level elevation in the filter influent channel. Filter aid is dosed at the influent to each filter. The filters share a single backwash pump equipped with a VFD to provide variable flowrates depending on filter size and water temperature. There is currently no back-up supply for backwash water.

As part of the 1983 filter re-build project, each filter was designed to hold a 24-inch tri-media configuration with the following specifications:

- Top: 12 inches of 0.9 to 1.0 mm anthracite
- Intermediate: 9 inches of 0.40 to 0.50 mm sand
- Bottom: 3 inches of 0.25 to 0.35 mm garnet/ilmenite
- Support: 13 inches of graded gravel, including 3-5 different sizes

All filters are currently equipped with a proprietary underdrain system called "Hydrocone" produced by BIF. This underdrain system is comprised of 4' x 4' concrete

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panels with multiple cones in the floor that allow water entry/exit through them. Several boxes of replacement cones are stored in the plant office, but these are no longer commercially available or manufactured. This system is built above the filter floor with a plenum underneath to collect and distribute water. Filters 6, 7 and 8 were designed with an underdrain flume to distribute backwash water; Filters 1 through 5 simply rely on the front flume created underneath for water distribution. ***Figure 2-12*** presents a typical cross-section for each filter configuration; ***Table 2-3*** summarizes design criteria for each set of filters.

Filter media and support gravel for all of the filters was replaced between 1995 and 2001. There are limited records regarding the specification of media actually placed in the filters. However, operators indicated that the bottom "polishing" layer of ilmenite was only added to Filters 1, 2 and 3; a dual media configuration (anthracite over sand) was installed in Filters 4 through 8. Based on the effective size of the specified media, the anthracite and sand were slightly mismatched (i.e. the anthracite and sand layers are not expected to properly separate following backwash). Thus, the media installed is expected to intermix, promoting tighter media (less void spaces) lending a slightly higher initial headloss (i.e. shorter filter runs) and though inconsistent, potentially improved filtered water quality.

The filter backwash program includes a "ramp-up", surface wash, high rate and "ramp-down" period. General durations for each step are summarized below, actual durations may vary between filters.

- 0 – 4 minutes – Backwash "Ramp-up" Period (0 – 100% BW flow)
- 2 – 7 minutes – Surface Wash
- 4 – 15 minutes – High Rate Backwash (100% BW flow)
- 15 – 19 minutes – Backwash "Ramp-down" Period (100 – 0% BW flow)

TABLE 2-3: ORIGINAL FILTER DESIGN CRITERIA

Parameter	Filters 1-3	Filters 4 & 5	Filters 6-8
Length x Width (feet)	17 x 15	21 x 18	18 x 18
Surface Area, each filter (sf)	255	378	324
Surface Area, total (sf)	765	756	972
Nominal Media Depth (inches)	24	24	24
Support Gravel Depth (inches)	13	13	13
Underdrain Type	BIF Hydrocone	BIF Hydrocone	BIF Hydrocone
Rated Maximum Filtration Rate (w/ largest filter in backwash) (gpm/sf)	6.57	6.57	6.57
Rated Maximum Filter Flow, each (gpm)	1675	2480	2130
Combined Maximum Filter Flow (gpm)	4262	4212	5415
Distance from Troughs to Top of Media (inches)	37-38.5	36-37	36-37.5
Nominal Submergence Over Top of Media (feet)	4.25	4.25	3.88
Normal Maximum Operating Headloss (feet)	7.0	7.0	7.0
Maximum Backwash Flow (per O&M Manual recommendations) (gpm)	4,500	5,500	5,000
Maximum Backwash Rate (gpm/sf)	17.6	14.6	15.4
Surface Wash Type	"S"-type rotary	"S"-type rotary	"S"-type rotary
Surface Wash Diameter (ft)	7.0	8.5	8.5
Surface Wash Flow (gpm), approximate	200 - 300	230 - 350	230 - 350
Surface Wash Flow Rate (gpm/sf), approximate	0.8 – 1.2	0.6 – 0.9	0.7 – 1.0

According to plant staff, the maximum backwash rate is not currently varied seasonally to account for temperature and viscosity effects to achieve adequate bed expansion. As a rule of thumb, the backwash rate should be increased/decreased 2 percent for every 1-degree C increase/decrease in water temperature over/under 20°C (68°F). With normal winter water temperatures in the range of 45°F (7°C) and summer normal water temperatures in the range of 61°F (16°C), this represents an approximate 18 percent difference in optimum backwash rates seasonally. There is no backup backwash supply when/if the backwash pump is ever out of service. To date, there have been no such outages.

Filters are backwashed when the headloss exceeds 7.0 ft or when the turbidity of an individual filter reaches approximately 0.35 NTU. Filter runs are usually terminated by headloss during most of the year. Filter-to-waste is employed after each backwash to ensure the filter has been adequately rinsed, and typically lasts 5 – 10 minutes. Backwash rates listed in ***Table 2-4*** may have been appropriate for the original tri-media configuration, but are too low to achieve adequate fluidization of the dual media (with 1.0 – 1.1 mm anthracite) installed in Filters 4 through 8 as part of the 1995 filter replacement project. Optimal backwash rates for the installed media are presented later in this section. During the WTP survey, it was noted that backwash flows in excess of 4,500 gpm can not be tolerated in Filters 1, 2 and 3 due to "choking" in the washwater channel/piping.

Various filter performance indicators were reviewed and analyzed including filtered water turbidity, filter run lengths and backwash volumes. Results and conclusions from this analysis are presented in the following sections.

2.4.3.1 Turbidity

Each filter at the Grants Pass WTP is equipped with an on-line turbidimeter; another on-line turbidimeter located in the high service pump station (HSPS) measures finished water turbidity. Data from each of these on-line instruments is used for regulatory reporting. ***Figure 2-13*** presents a summary of daily maximum combined filtered water turbidities between January 1999 and July 2003, taken from the plant's regulatory summary sheets reported monthly to the DHS. As shown in the figure, the maximum daily turbidity has always been less than 0.90 NTU, and is usually less than 0.10 NTU. ***Figure 2-14*** presents a statistical summary of maximum daily plant effluent turbidities between January 1999 and July 2003. From the figure, the plant has produced 0.12 NTU water 95 percent of the time. The plant has normally performed well with respect to meeting the desired turbidity goal for optimal particulate removal.

Individual filtered water turbidities have only been recorded since March 2002, when the new SCADA system was brought on-line. ***Figure 2-15*** presents a statistical summary of individual filtered water turbidities recorded every 5-minutes. On-line measurements

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recorded following plant start-up and during individual backwash/filter-to-waste cycles were omitted from the data series. In general, there are no "problem" filters—all filters are generally performing well with regard to overall particulate removal. Filter 1 shows consistently lower filtered turbidities, possibly resulting from the smaller ilmenite media. Filter 4 & 5 (the largest filters) show consistently higher turbidities (approximately 0.02 NTU higher) relative to the other six filters, potentially due to the significantly higher pH values through these filters resulting from the lime addition in Basin #2. All filters are producing filtered water turbidities <0.15 NTU for 95 percent of the time. It should be noted that the values presented in the figures are subject to error associated with instrument calibration and flow variability. Therefore, many of these values should be considered "statistically similar".

2.4.3.2 Filter Production Efficiencies

To evaluate overall plant efficiency, a relationship between a filter's production, run lengths and backwash volume requirements is required. Based on numerous studies and detailed analysis, MWH developed the concept of Unit Filter Run Volume (UFRV) as a tool for determining whether a filter is performing efficiently.

In general, maximum net water production is desirable because it minimizes capital and operating costs. The principal parameters that impact net water production for a given filter and influent quality are filtration rate, filter run length and the amount of water used for backwash. The filter area required for a given plant capacity is determined by the net or effective filtration rate (R_e), which is the net amount of product water generated per unit time per unit of filter area (commonly expressed in gpm/sf). The effective filtration rate is contrasted with the design filtration rate (R_d), which is the maximum rate at which the filter is designed to pass water. The difference between the two rates is related to:

1. The volume of water that passes through each unit of filter area during the course of a filter run, typically expressed in gal/sf, and also referred to as the Unit Filter Run Volume (UFRV), and

2. The volume of backwash water required per unit of filter are, typically expressed in gal/sf, and also referred to as the Unit Backwash Volume (UBWV)

The following relationship can be developed for these parameters as follows:

$$R_e = R_d \times [(UFRV - UBWV)/UFRV]$$

Figure 2-16 illustrates the relationship between the production efficiency (R_e/R_d) and UFRV for various UBWVs from 100 gal/sf to 300 gal/sf. UBWV is calculated by multiplying the backwash flowrate (gpm) by the duration of backwash (min) and dividing by the total filter surface area. For reference, the current UBWV for the Filters 1, 2 and 3, Filters 4 and 5, and Filters 6, 7 and 8 are 235 gal/sf, 218 gal/sf and 231 gal/sf, respectively, based on current backwash procedures.

From the figure, it is apparent that a significant reduction in filter production efficiency results when the UFRV drops below 5,000 gal/sf. The plant production efficiency at 5,000 gal/sf is approximately 97% (at UBWV = 150 gal/sf). As a result, WTPs in which the UFRV is below 5,000 gal/sf must be designed with much larger washwater handling facilities, not only because the volume of washwater increases, but because the rate of change in backwash requirements increases rapidly if the UFRV is too low. For these reasons, MWH designs filters for an absolute minimum UFRV of 5,000 gal/sf with a preference for higher UFRVs for conventional filtration plants with sedimentation basins. Above a UFRV of 10,000 gal/sf, there is little increase in production efficiency, so major efforts are not usually taken to achieve very high UFRVs. Also, most WTPs would not let their filters run indefinitely between backwashes assuming that headloss and/or turbidity criteria are still being met. Usually, the maximum filter run length limit is set for approximately 3 to 4 days for operational and maintenance purposes.

The UFRV allows a comparison of water production at different filtration rates that contrasts with filter run lengths, which depend on rate. UFRV, which is a measure of

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filter throughput for a given filter run, is calculated as the product of the filtration rate and the filter run length. For example, a filter run of 24 hours (1440 minutes) at a filtration rate of 5.0 gpm/sf produces a UFRV of 7,200 gal/sf. ***Table 2-4*** lists the filter run lengths necessary to achieve the minimum UFRV goal of 5,000 gal/sf for the City's current situation with all 8 filters on-line and with one of the larger filters off-line for backwashing. It should be noted that if the City achieves the 5,000 gal/sf goal with an average UBWV of 150 gal/sf, the production efficiency (R_e/R_d) will be 97 percent, considered the minimum desirable filter production efficiency. [NOTE: A discussion of reducing the current UBWV values from approximately 230 gal/sf to 150 gal/sf are discussed later in this report.]

TABLE 2-4: MINIMUM FILTER RUN LENGTH TO ACHIEVE 5,000 GAL/SF UFRV

Filtration Rate (gpm/sf)	Average WTP Flow with all 8 filters on-line (mgd)	Average WTP Flow with largest filter off-line (mgd)	Minimum Filter Run Length to Achieve UFRV = 5,000 gal/sf (hours)
3.0	10.8	9.1	27.8
4.0	14.4	12.2	20.8
5.0	17.9	15.2	16.7
6.0	21.5	18.3	13.9
7.0	-	21.3	11.9

At the current rated maximum plant capacity of 20 mgd (with all 4 raw water pumps operating), the filters should operate for a minimum of 15 hours between backwashes to meet the 5,000 gal/sf UFRV criteria. During times of the year when the plant is operating at lower flows, the filters should operate for a minimum of 20 and 30 hours between backwashes for two pumps (10 mgd) or three pumps (15 mgd), respectively, to meet the 5,000 gal/sf criteria. It should be noted that the filtration rates required to deliver flows in excess of 15 mgd are relatively high for the shallow tri- or dual-media installed in each of the filters. High filtration rates result in high incremental headloss and short filter runs.

Plant operating records between January 1999 and July 2003 including raw water flow, plant production, backwash volumes and filter run lengths, were reviewed to determine

the filter production efficiencies and UFRVs. The plant production efficiencies were computed based on daily raw water and finished water flows. Backwash volumes were computed from the difference between influent and effluent daily flows. ***Figure 2-17*** represents weekly average backwash volumes as well as weekly filter production efficiencies. Seven-day running averages were used in lieu of daily averages to "normalize" the data. Also shown on the figure is the 97 percent production efficiency target.

In general, the overall plant filter production has been significantly less than 97 percent, and often as low as 80 percent. It can be seen that the efficiency of the filters generally drops in the winter when total production is lower and the water is colder and more turbid. The average UFRV for the filters during this period was less than 2,500 gal/sf, almost one half of the suggested minimum UFRV. UBWV is also higher than desired. This means that the filters are performing inefficiently, resulting in poor plant production efficiencies and excessive use of filtered water for backwashing (i.e. higher than desired UBWVs). This also indicates that the filters are being "stressed" beyond acceptable conditions when one filter is taken off-line for backwashing. During backwashing, the filtration rate through the remaining filters increases overloading the filters and exacerbating the short filter runs. Filter investigations conducted to help identify the reasons for this poor performance are summarized in the following section.

2.4.3.3 Special Filter System Analyses

Three of the eight filters, one from each of the three filter configurations, were evaluated during the 2-day WTP inspection conducted July 29-30, 2003. The filters were drained, media depth and the top of the gravel support layer were measured. Core samples were also collected from one location in each filter, both before and after backwashing, and floc retention was measured on all three filters. Backwash turbidity profiles were performed on two of the three filters analyzed. Sieve analysis was conducted on the media samples. Results from these analyses are presented below, results from the lab can be found in ***Appendix C***.

Media and Gravel Support Condition: A summary of observations made during inspection of the media and gravel support follows:

- All filters had significantly less media than expected. Existing media configurations for the three filters are summarized below.

  Filter 1: 10-12" anth / 8-10" sand / 0-1.5" ilmenite (≅ 18 – 20 inches total)

  Filter 5: 10-12" anth / 6-8" intermixed anth/sand (≅ 18 inches total)

  Filter 7: 6-8" anth / 10-12" intermixed anth/sand (≅ 18 inches total)
- Very little of the original ilmenite is remaining in Filter 1 (0-1.5 inches). Minor depressions in the filter media were observed following backwash (1-3" deep), indicating some minor variability in backwash flow distribution. Also, noticeable "cracking" in media following backwash was observed.
- Filters 1 and 7 lacked a distinct sand layer; all remaining sand media was intermixed with anthracite, less sand was present in Filter 7.
- In all filters, the top of media was too far below the surface wash "sweeps" (typically 4-6 inches below), potentially limiting surface wash efficiencies during the backwash cycle.
- Gravel support was not "upset" (i.e. gravel was uniformly distributed throughout each of the filters) and appeared to be in good condition indicating maximum backwash rates have not been exceeded historically. Gravel depth (from the lip of the trough to top of the gravel) were measured and recorded:

  Filter 1: 58.2 ± 3.0 inches

  Filter 5: 54.8 ± 1.0 inches

  Filter 7: 55.2 ± 1.1 inches

In general, the filter media and gravel support appeared to be in acceptable condition. All filters have lost media over the years, possibly due to carry-over during backwash. No significant disturbances in the gravel support were observed. Filters 1, 2 and 3 and Filters 6, 7 and 8 lack a distinct sand layer; the sand remaining in the filters is intermixed with the anthracite.

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Sieve and Specific Gravity Analysis of the Media: The objective of the sieve analysis was to identify the size of the existing media to help determine whether the sand and anthracite are properly "matched". Results from the sieve analysis, combined with the specific gravity, can be used to determine the appropriate backwash rate for the filters.

Current MWH sand and anthracite specifications for filter media call for a uniformity coefficient less than 1.4 and 1.4, and a specific gravity greater than 2.65 and 1.6, respectively. ***Table 2-5*** provides a summary of the results of the sieve analysis on the core samples from the three filters considered during this investigation.

TABLE 2-5: FILTER MEDIA ANALYSIS RESULTS

Media Layer	Filter 1		Filter 5		Filter 7		Apparent Specific Gravity[1]
	Effective Size (mm)	UC	Effective Size (mm)	UC	Effective Size (mm)	UC	
Sand	0.54	1.29	0.54	1.32	0.46	1.21	2.64
Anthracite	1.03	1.37	1.06	1.29	1.15	1.35	1.43

[1]Analyzed from the Filter 5 sand and anthracite samples

The media was also analyzed by a method commonly used to estimate filter performance, called the "L/d ratio" (depth (L) to diameter (d)). This dimensionless parameter provides a basis of comparing differing media types and sizes based on the depth and average diameter of the media. The 24-inch deep tri-media configuration specified as part of the 1983 improvements project had an L/d ratio of approximately 1,082 (=278 [for 12" of 0.95 anthracite] + 508 [for 9" of 0.45 sand] + 254 [for 3" of 0.30 ilmenite]). With an average of 18" of media remaining in the filters, and with some sand missing from Filter 1 and 7, the L/d ratio for the existing filters are calculated to be:

- **Filter 1: 708** (= 247 [for 10" of 1.03 anthracite] + 377 [for 8" of 0.54 sand] + 85 [for 1" of 0.30 ilmenite])
- **Filter 5: 524** (= 335 [for 14" of 1.06 anthracite] + 188 [for 4" of 0.54 sand])
- **Filter 7: 563** (= 287 [for 13" of 1.15 anthracite] + 276 [for 5" of 0.45 sand])

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It has been proven in many cases that a minimum L/d ratio of 1,000 is desirable and filter performance will suffer accordingly as L/d is reduced. The L/d ratio for the existing filters appears to be inadequate for meeting filtration performance goals, suggesting the filters have limited solids holding capacity. Additionally, it appears that the media in Filter 7 is not properly "matched", evident by the large intermixed zone observed. Sub-optimal backwash rates may also contribute to this intermixed zone, as the media relies on high rates (i.e. fluidization) to separate following backwash. Though this intermixed zone is capable of producing high quality filtered water, the headloss associated with intermixed media is much higher compared to distinct anthracite/sand layers (i.e. less void space for solids holding), leading to shorter filter runs and decreased efficiencies.

Backwash Efficiency: Backwash turbidity profiles were used to evaluate cleanliness of the media following backwash. ***Figure 2-18*** presents backwash turbidity profiles for Filter 5 and 7, taken as part of the recent WTP investigation; a turbidity profile for Filter 3, created during a brief previous filter survey was also included (Black and Veatch, 2003). *[Please Note: The Filter 3 profile was performed following a filter core investigation when the media was corrupted—the filter was completely drained of water, therefore the backwash regimen was significantly altered to accommodate the air entrapped in the media.]* For both Filter 5 and 7, a "low profile" (i.e. low peak curve) was observed, indicating an ineffective washing (Kawamura, 2001). Also, washwater turbidities <10 NTU were achieved approximately 8 minutes after backwash water began spilling into the trough. There is minimum benefit to continuing backwash once washwater turbidities have fallen below 10 NTU. This implies that the filter backwash duration could be reduced now to minimize backwash water usage, thereby minimizing the UBWV and increasing plant efficiency. These tests should be repeated during the winter, when the solids loading on the filters may be higher. A summary of observations made during filter inspection and backwash follows:

- Backwash water was evenly distributed throughout the filters; no "boiling" was observed during the backwash cycle.

- A small accumulation of mud-balls was observed along the walls in each of the filters, suggesting poor expansion of the media and minimal benefits from the surface wash sweeps. Prior to our site visit, plant staff had removed a large number of mud-balls as part of routine filter maintenance.

To evaluate the efficiency of backwash in cleaning the media, a floc retention profile analysis was conducted for Filters 1, 5 and 7. In the analysis, turbidity levels in solutions of solids extracted from various depths of media both before and after a backwash were used to evaluate backwash performance. The solids were collected by shaking 50 milliliters (mL) of media collected from various depths of the filter bed into a 500-ml flask containing 100 mL of tap water. Following shaking, the turbidity of the solution was measured. The data was then normalized to 100 mL of media. ***Figure 2-19*** shows the floc retention profiles both before and after backwash for Filters 1, 5 and 7; a profile created during a previous filter survey was also included (Black and Veatch, 2003). The figure also includes an "optimal" floc retention profile following a successful backwash (Kawamura, 2001).

In all three profiles, turbidity levels through the entire depth of the media prior to backwash were relatively consistent, suggesting good floc penetration (i.e. maximum solids removal). However, the measured turbidities are low compared to filters in similar plants, suggesting a relatively low overall volume of solids removed, corresponding to short filter run lengths. In profiles taken following backwash, it appears that only the top portion of the media is truly being cleaned (turbidity < 100 NTU) in Filters 1 and 5; no portion of Filter 7 is effectively cleaned. These results indicate that current backwash conditions are not adequately cleaning the media; the backwash rates are too low, limiting the expansion of the filter media during backwash.

Specific gravity analysis data, coupled with sieve analysis data, can be used to determine the optimum backwash rate for each of the filters. ***Table 2-6*** summarizes the current maximum backwash rate, as well as the calculated "optimal" backwash rates for the filters. Under optimal backwash conditions, the media bed is expanded by 35 to 50

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percent (Kawamura, 2001), promoting the agitation necessary to properly clean the media. Bed expansions during backwash, as recently measured by plant staff, are also reported in the table. Plant staff has experimented with backwash rates higher than those presented in the table, but this is not currently practiced due to pressure limitations/leaking in the existing backwash pipeline.

TABLE 2-6: BACKWASH SYSTEM DESIGN CRITERIA AND OPTIMAL RATES FOR EXISTING MEDIA CONFIGURATIONS

Filter Number	Filters 1-3	Filters 4 & 5	Filters 6-8
Current Backwash Flow (gpm)	4,500	5,500	5,000
Maximum Backwash Rate (gpm/sf)	17.6	14.6	15.4
Bed Expansion during Backwash[1] (in)	2 - 4	2	2
"Optimal" Backwash Rate[2] (gpm/sf)	18.5	18.5	19
"Optimal" Bed Expansion[3] (in)	6.3 - 10	6.3 - 10	6.3 - 10
"Optimal" Backwash Flow at 20°C (gpm)	4,720	7,000	6,160

[1]As measured by plant staff on July 11 and July 15, 2003 at varying backwash rates
[2]Based on sand/anthracite effective size, uniformity coefficients and specific gravity
[3]Assuming 18 – 20 inches of media

As shown, the current maximum backwash rates are sub-optimal, resulting in insufficient media expansion during backwash. Minimal bed expansion hinders adequate media agitation during, and separation following backwash. In addition, the minimal bed expansion also hinders the effectiveness of the surface wash, as the media is too far below the surface wash arms during backwash. The filters are not and can not be properly cleaned based on the media size. Poor cleaning leads to higher initial headloss, which reduces the available head for filtration, resulting in shorter filter runs. Relatively high filtration rates when one filter is out of service for backwash exacerbate the short filter runs. Backwashing at higher rates may not be possible with current filter configurations due to excessive media loss, backwash pump limitations (currently rated at 7,000 gpm) and waste washwater flow limitations to Filter 1, 2 and 3.

2.5 SUMMARY AND OBSERVATIONS

In general, the plant has performed well with regard to finished water quality, and has met the regulatory requirements for filtered water turbidity. However, plant production

efficiencies are typically 80 to 90 percent throughout the year, and generally decrease in the winter when total production is lower and the water is colder and more turbid. Plant efficiencies should be improved to minimize costs associated with plant operations (longer operation time, pumping and chemical costs, sludge production, etc.). Efficiencies of 97 percent are considered the minimum desirable filter production efficiency. Plant efficiencies can be improved by optimizing coagulation and increasing the filter run lengths via improvements to the filters and sedimentation basins. Some interim steps could also be taken to minimize the total volume of water used for backwash.

Presented below is a summary of historical plant performance and analyses presented in this section.

- Coagulation chemistry may be improved to reduce solids production and/or reduce chemical addition at the plant. To fully understand the possible benefits and costs of using alternative coagulants, pilot and/or full-scale tests should be conducted seasonally under different water quality conditions using a variety of chemicals/combinations to ensure that treatment requirements and performance are well understood. An "optimal" coagulation strategy will balance plant efficiency with coagulation chemical costs, disinfection requirements, sludge production and pH adjustment requirements.
- Overall, the sedimentation basins provide satisfactory water for filtration, as well as adequate contact time for disinfection during most of the year. All basins experience challenges with regard to short-circuiting, high solids loading (resulting from relatively high alum dosages), sub-optimal flocculation and seasonal turbidity spikes. The basins are not equipped with any type of on-line solids removal system; as solids accumulate in the basin, the effective volume of the basin is reduced, compromising flow characteristics and overall performance in the basin. The addition of formal flocculation, and/or additional settling time would also allow for lower alum doses.
- The plant has 8 mixed-media gravity filters of varying sizes, shapes and media configuration, depending on the time of construction. The filter media and

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underdrains appear to be in acceptable condition. However, none of the filters have optimal media configurations and several filters lack a sufficient sand layer.

- Based on our analysis, short filter runs result from relatively high filtration rates through a relatively shallow, dirty media. Filter media should be replaced with a new design, taking advantage of gravel-less systems to allow for deeper media bed.
- The filters are not and can not be properly cleaned. In addition, the minimal bed expansion hinders the effectiveness of the surface wash, as the media is too far below the surface wash arms during backwash. Poor cleaning leads to higher initial headloss, which reduces the available head for filtration, resulting in shorter filter runs and decreased plant efficiencies.
- The current maximum backwash rates are sub-optimal. Backwash rates for Filter 1, 2 and 3 are limited due to "choking" in the washwater pipelines. Backwash flowrates are currently limited to 7,000 gpm.
- As an interim step, the filter backwash duration could be reduced to minimize backwash water usage, thereby minimizing the UBWV and increasing plant efficiency.
- Excessive solids production and larger volumes of waste washwater are putting a "stress" on the current solids handling facilities. Solids production may be minimized through improved coagulation. Long-term alternatives for solids management must be developed (discussed in detail in **Section 6**).

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Figure 2-1
Grants Pass WTP:
1999-2003-- Daily Raw Water and Finished Water Flows

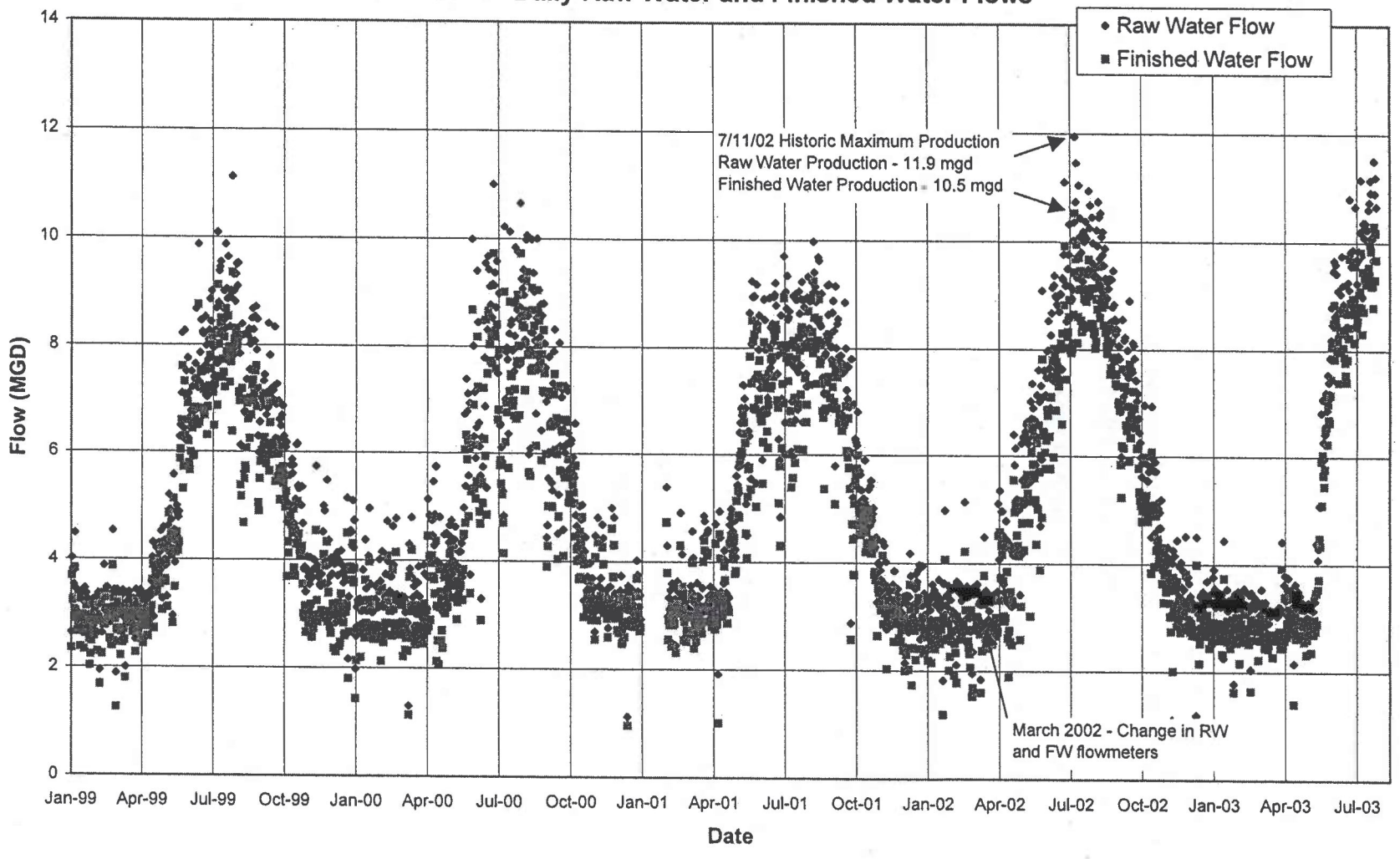

Figure 2-2
Grants Pass WTP:
1999-2003-- Average Daily Plant Raw Water Turbidity and Precipitation

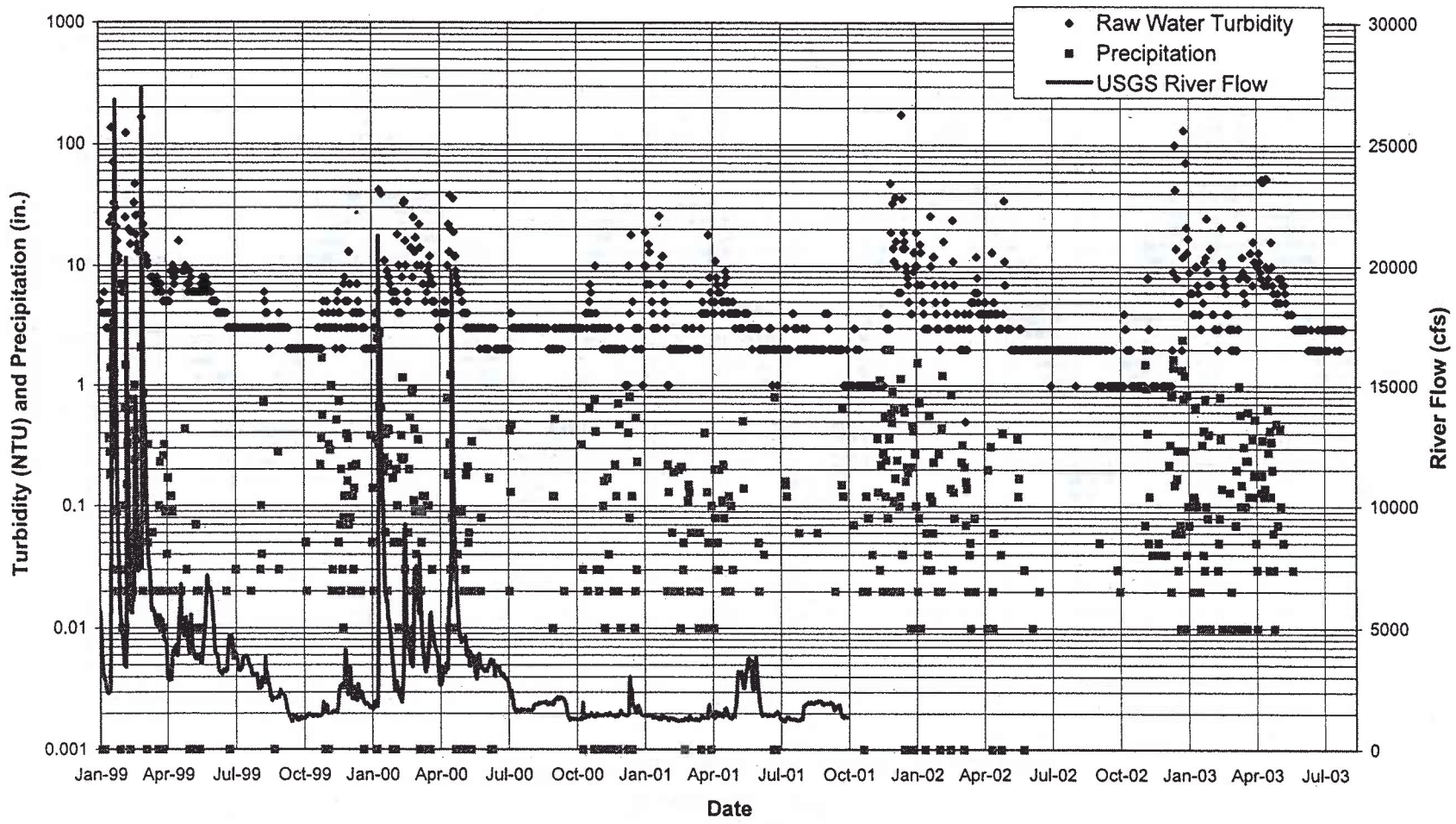

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Figure 2-3
Grants Pass WTP:
1999-2003-- Average Daily Raw Water Temperature

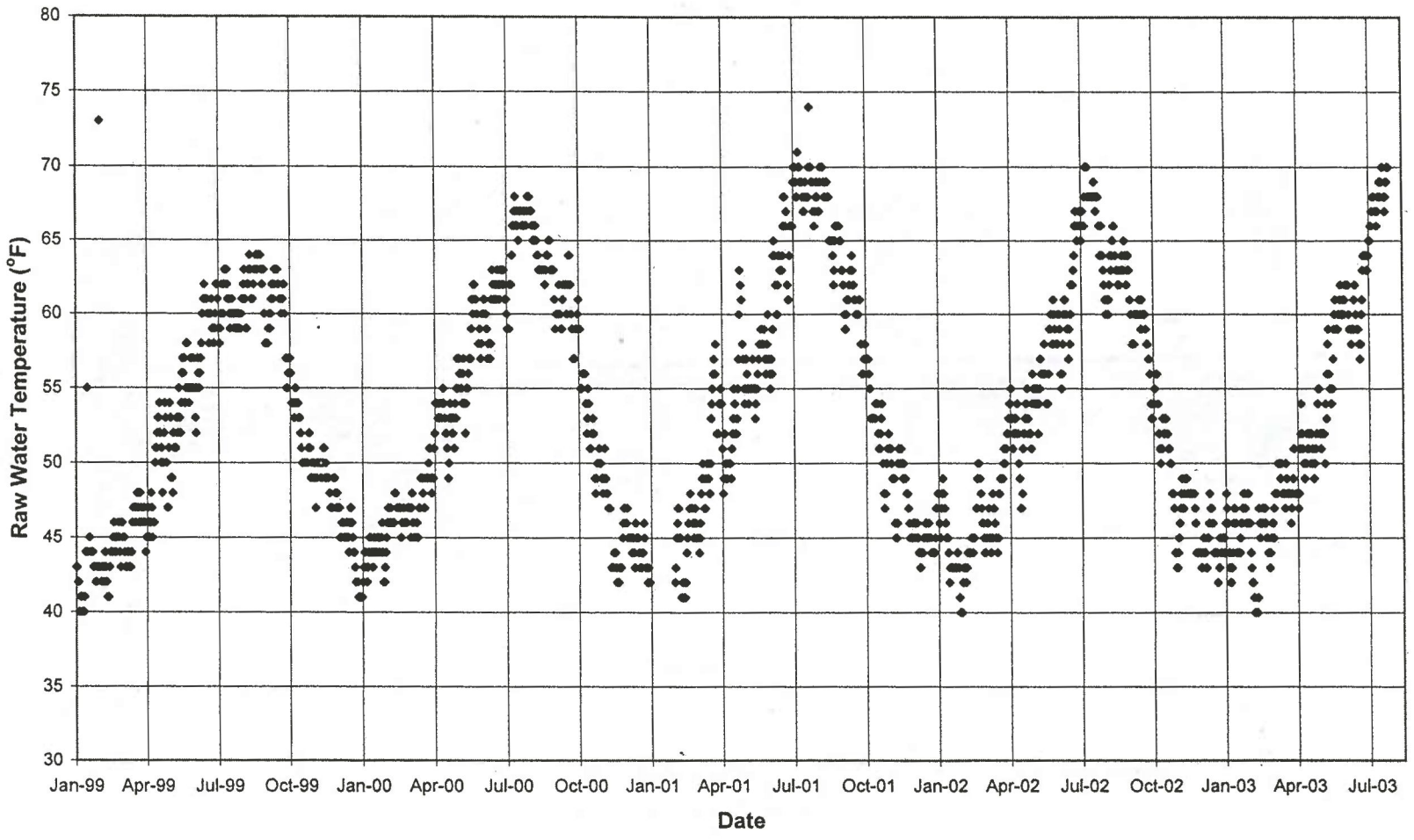

Figure 2-4
Grants Pass WTP:
1999-2003-- Average Daily Plant pH: Raw and Finished Water

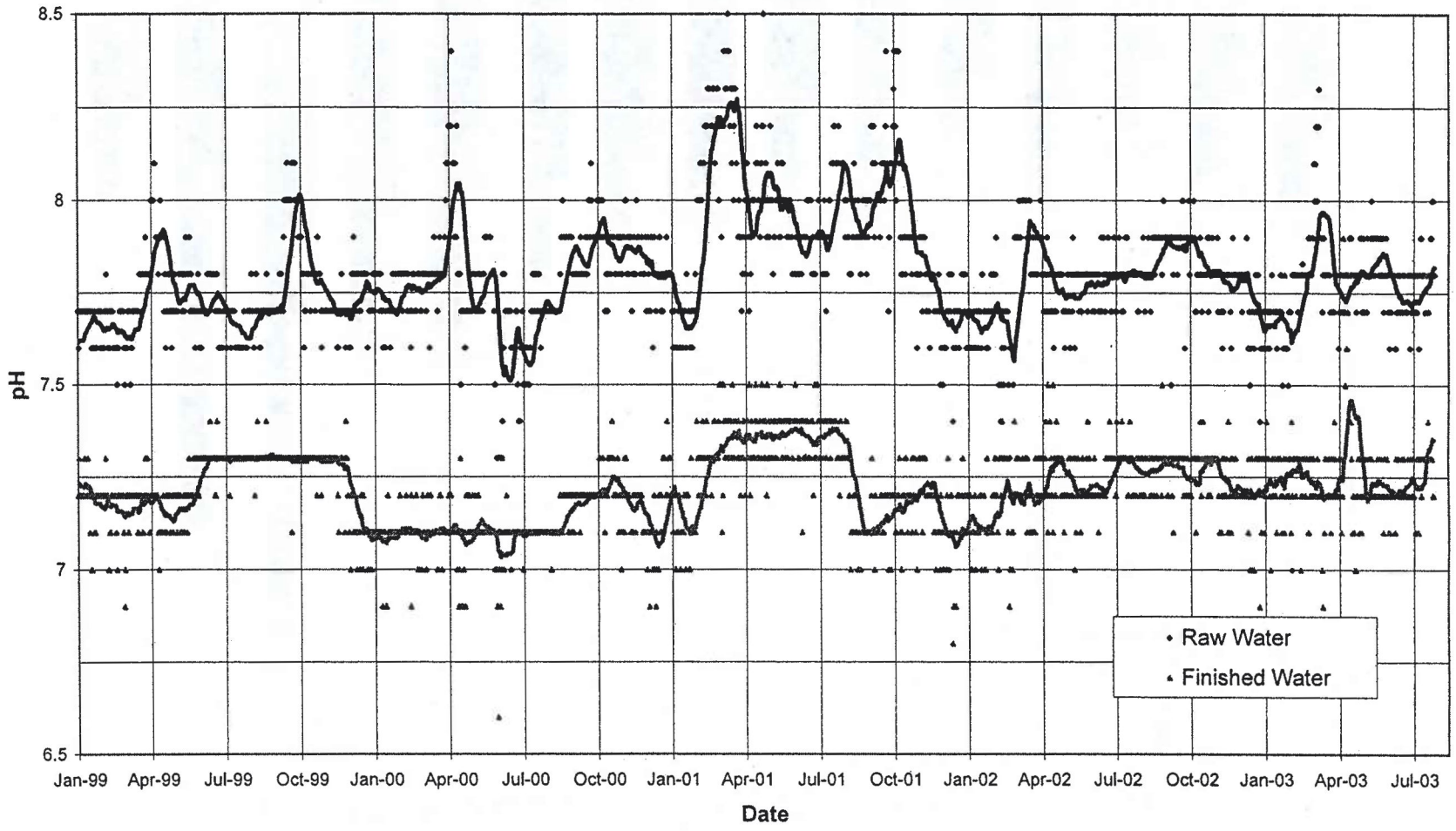

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Figure 2-5
Grants Pass WTP:
2001-2002 -- Plant Raw Water and Finished Water TOC and Removal Efficiency

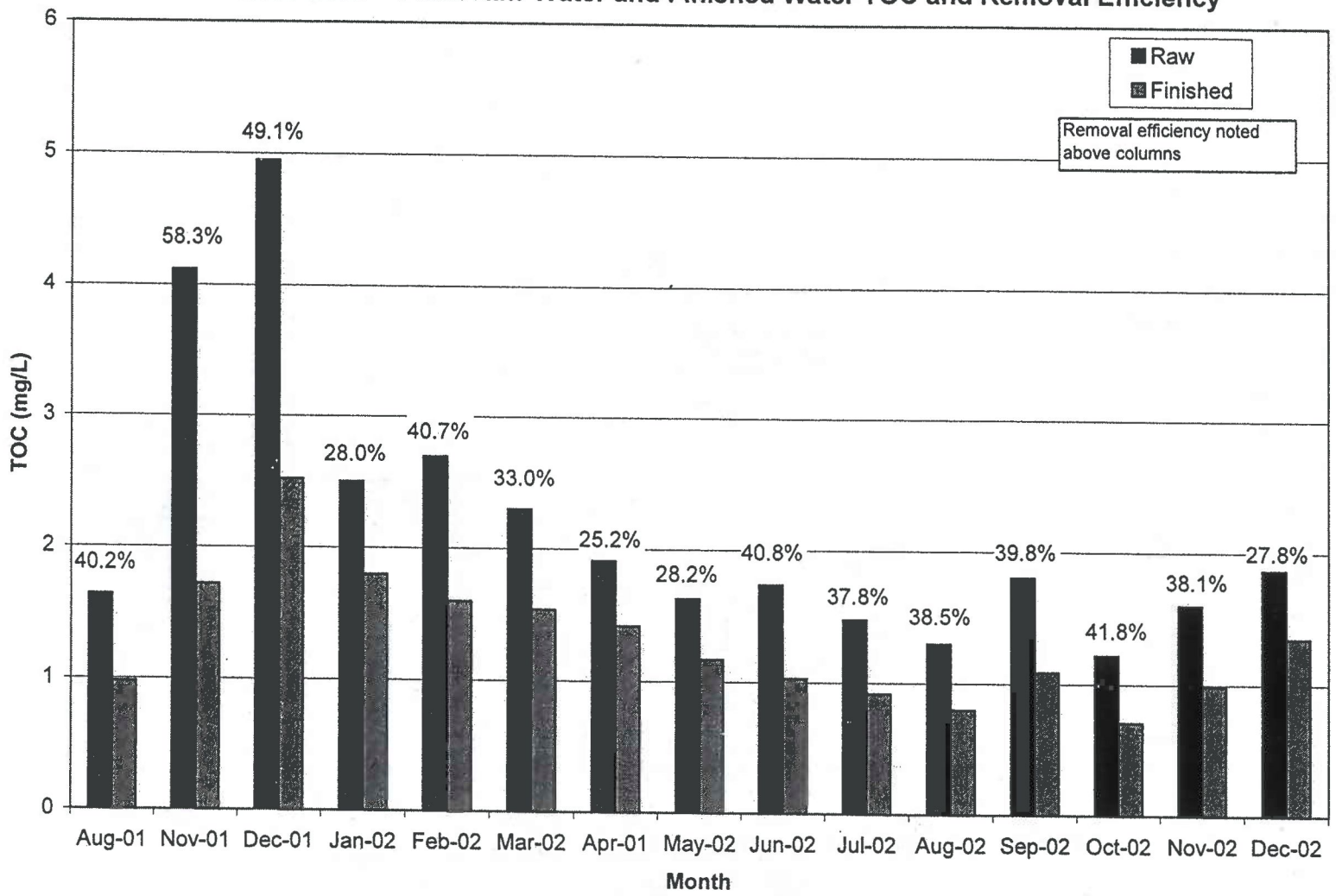
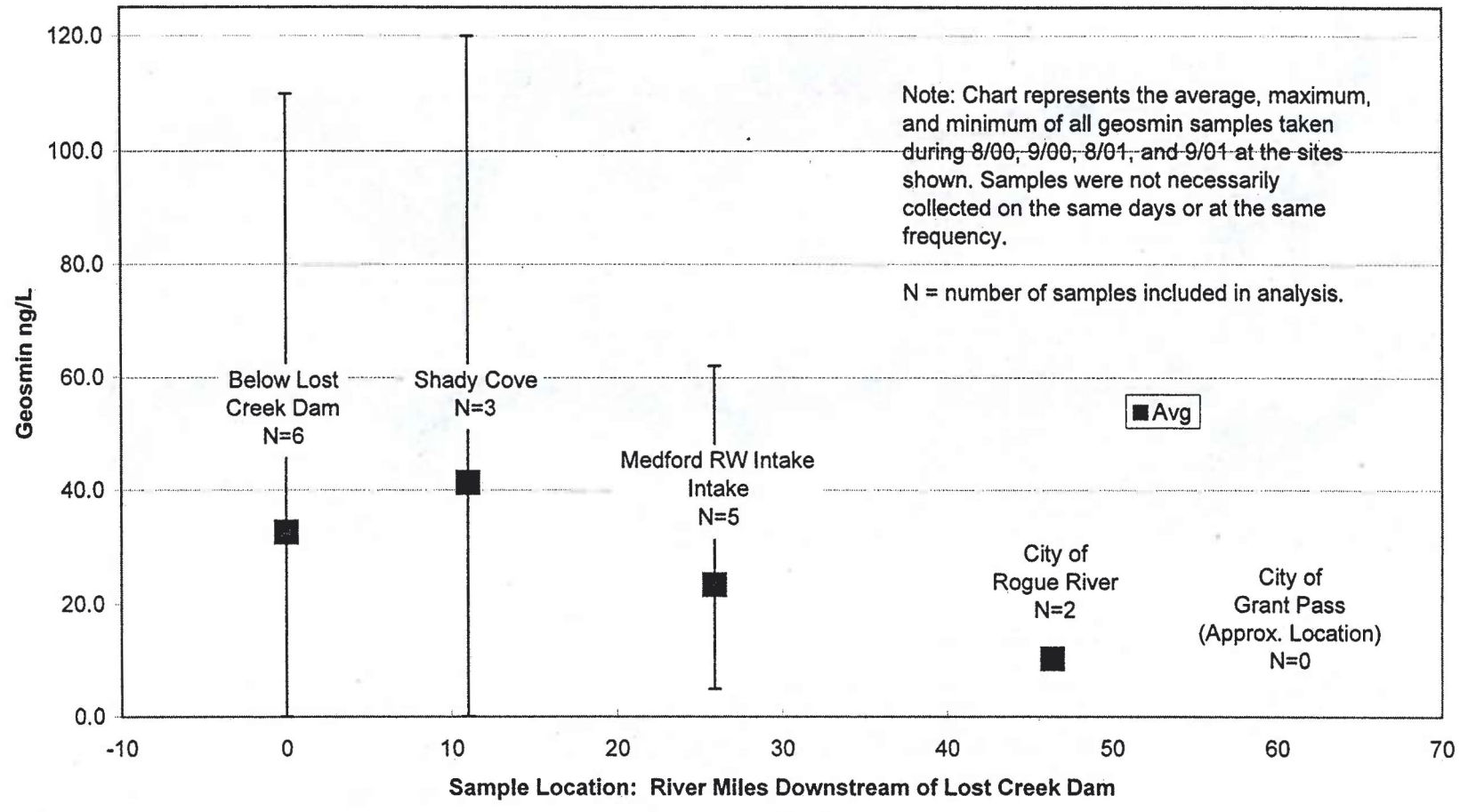


Figure 2-6
Rogue River Raw Water:
Geosmin Concentrations between Lost Creek Dam and City of Rogue River
Peak Taste and Odor Season Samples



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96 330

Figure 2-7
Grants Pass WTP:
1999-2003-- Plant Average Daily Alum and Filter Aid Polymer Dose

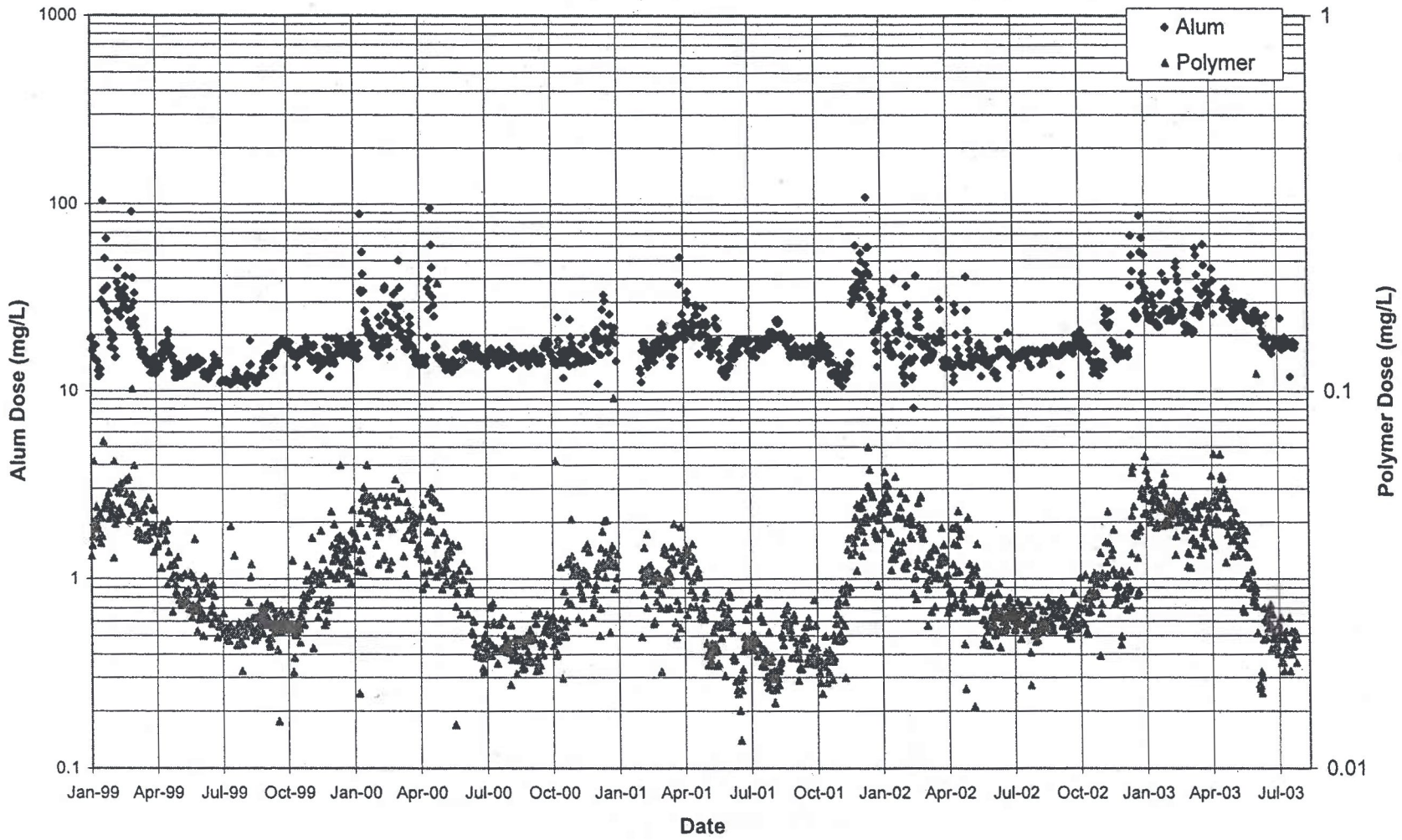
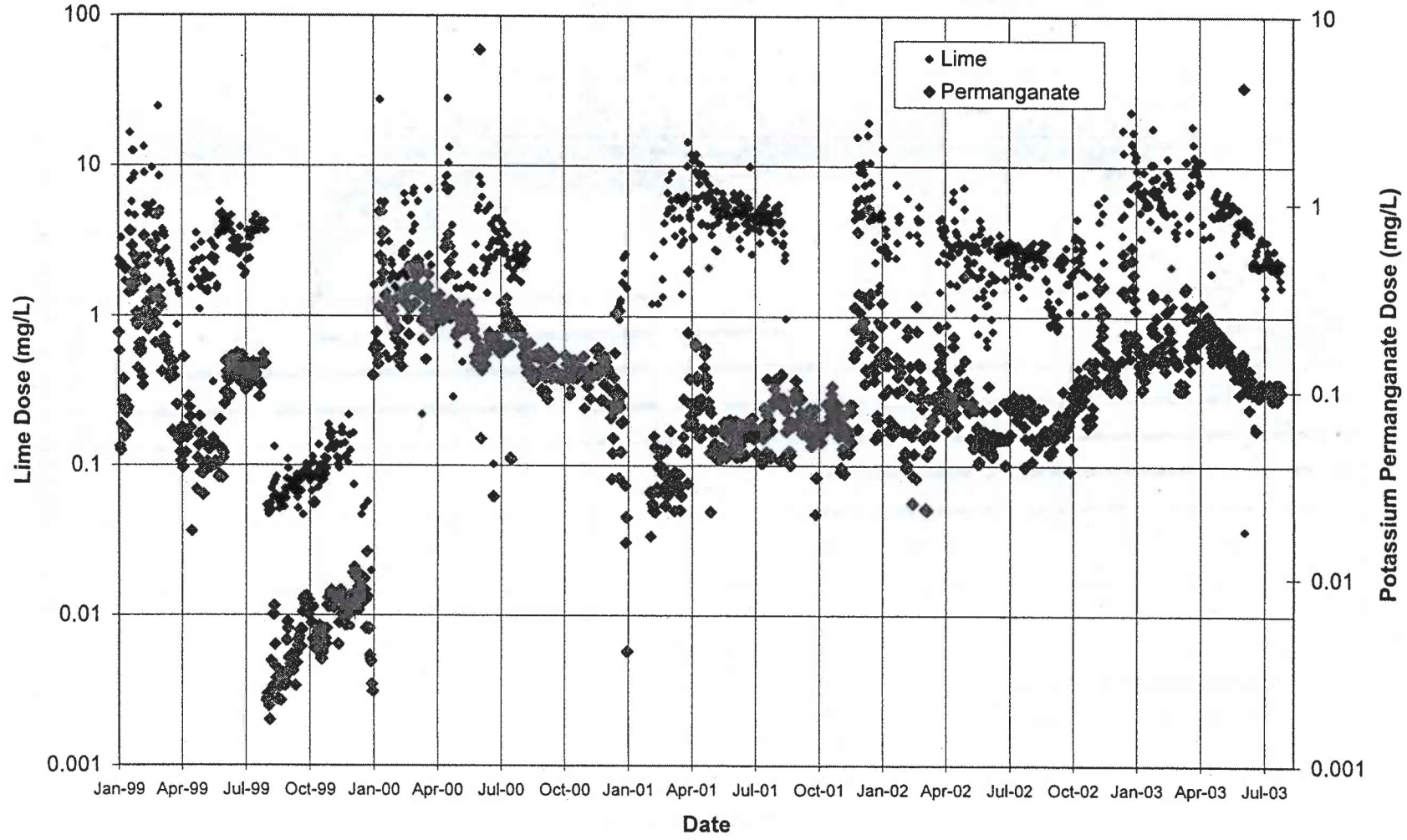


Figure 2-8
Grants Pass WTP:
1999-2003-- Plant Average Daily Lime and Potassium Permanganate Dose



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2003000

Figure 2-9
Grants Pass WTP:
1999-2003-- Plant Average Daily Chlorine Residuals

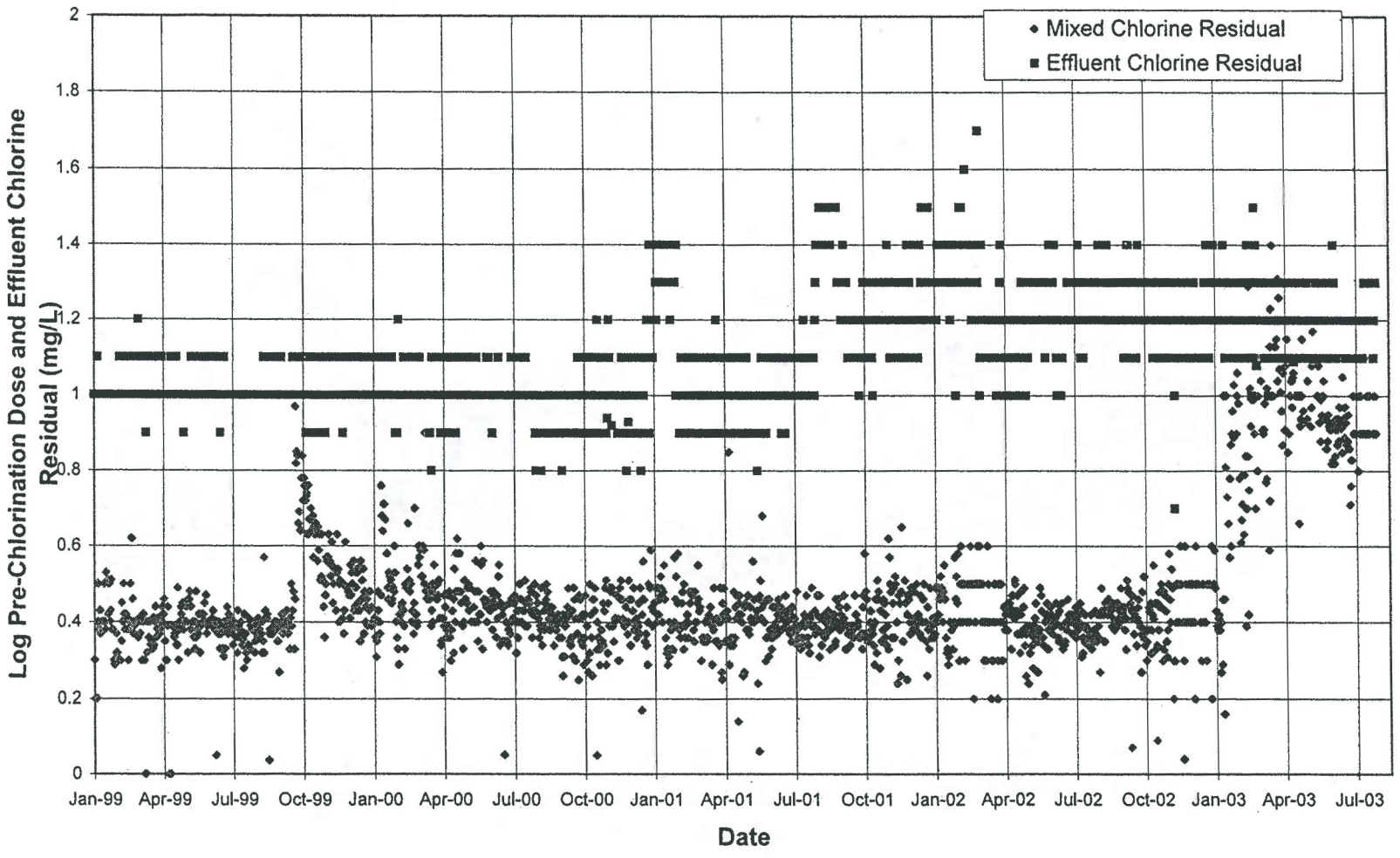

Figure 2-10
Grants Pass WTP:
April 2002 - June 2003-- Average Daily Contact Basin Effluent Turbidity
9am-3pm Continuous Monitoring.

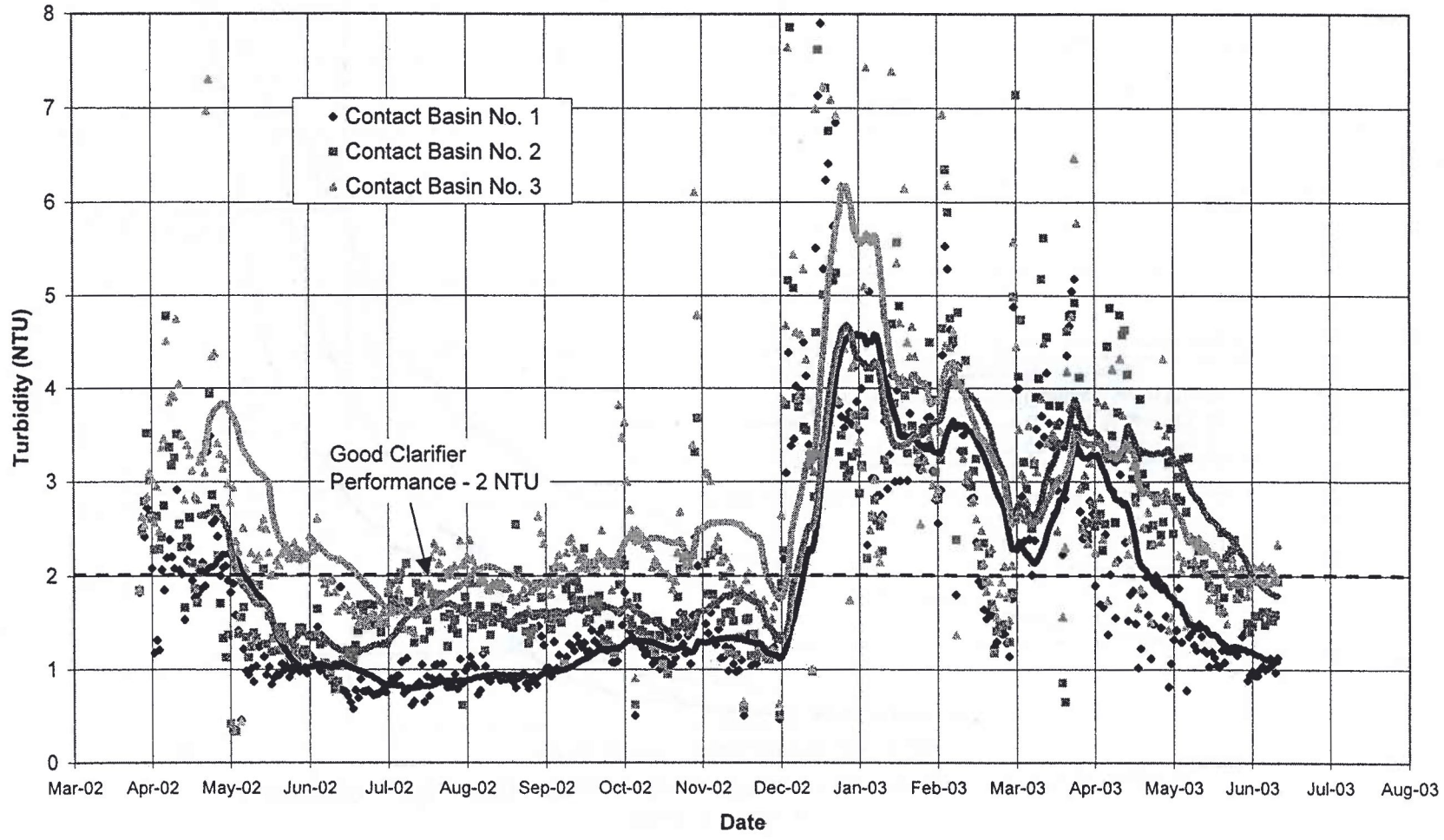

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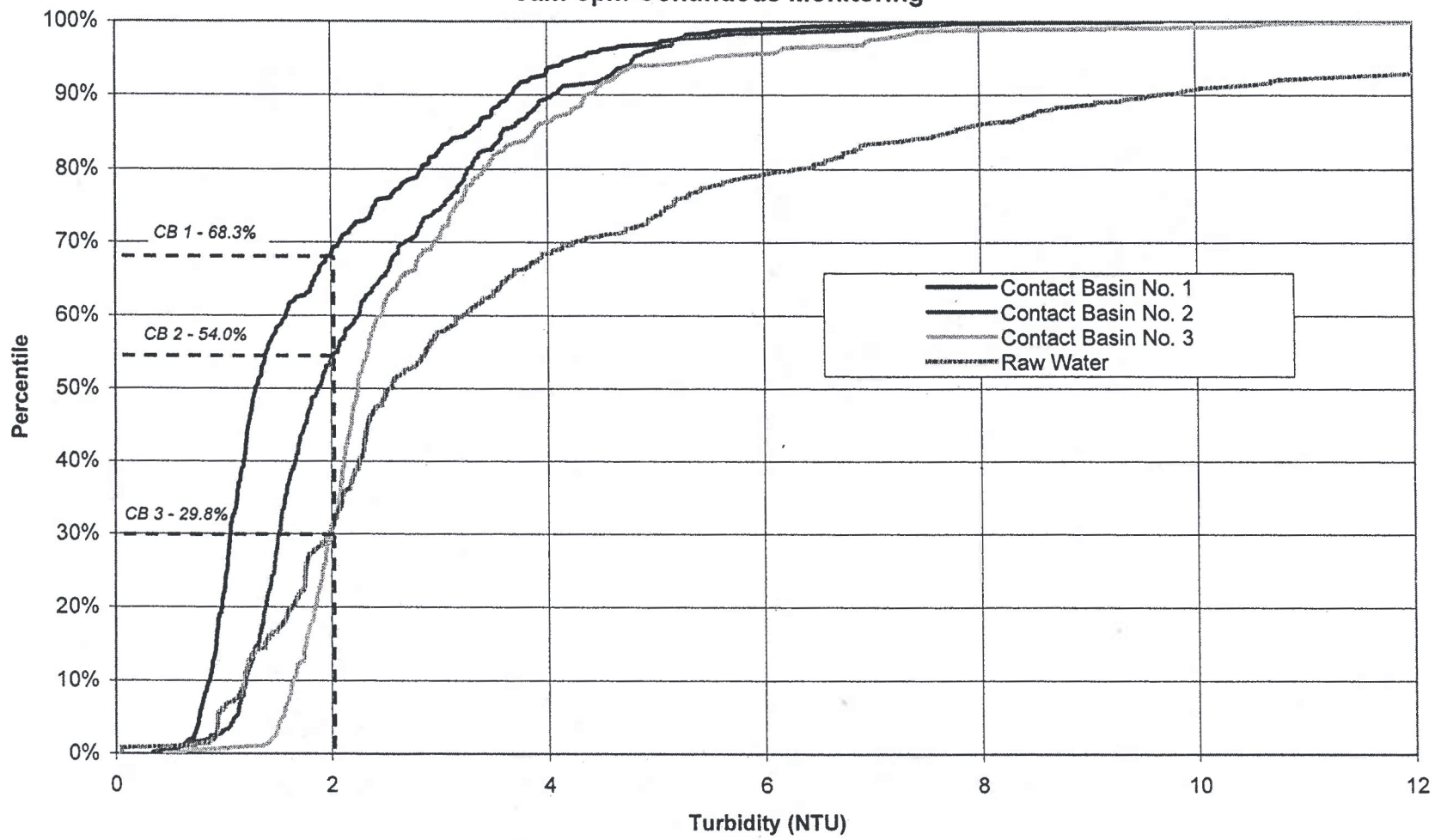
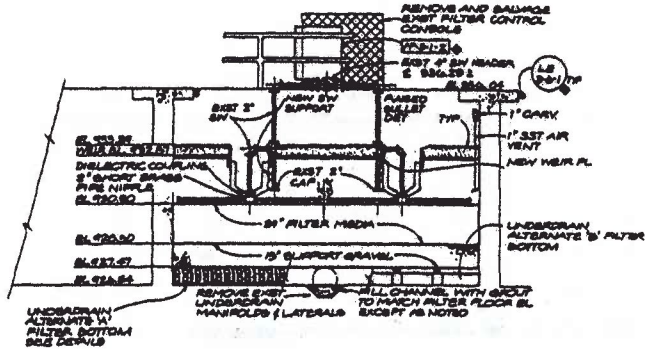
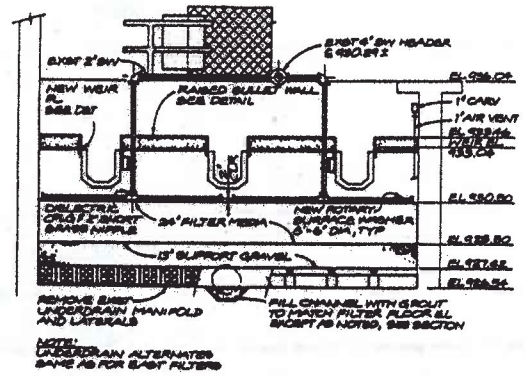
Figure 2-11
Grants Pass WTP:
April 2002 - June 2003-- Contact Basin Effluent Turbidity Probability Distibutions
9am-3pm Continuous Monitoring

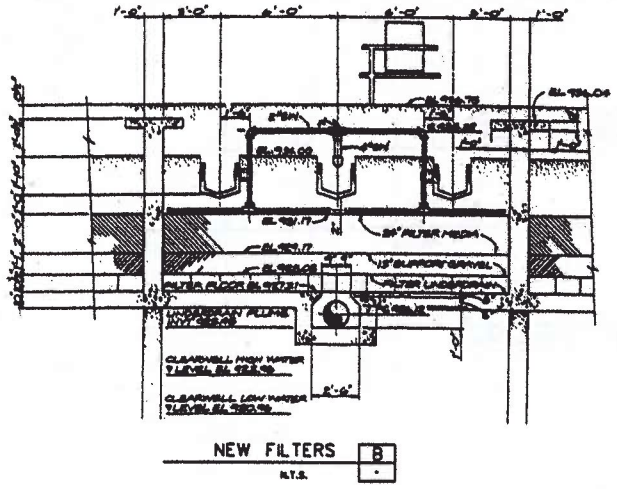




EAST FILTERS A
N.T.S.



WEST FILTERS C
N.T.S.



NEW FILTERS B
N.T.S.

NOTES:

1. FILTER DIMENSIONS:
   - EAST FILTERS (1-3): 17' x 15' = 255 sf EA.
   - WEST FILTERS (4, 5): 21' x 18' = 378 sf EA.
   - NEW FILTERS (6-8): 18' x 18' = 342 sf EA.
   - TOTAL SURFACE AREA: 2493 SF
2. BIF HYDROCONE UNDERDRAIN

MWH
MONTGOMERY WATSON HARZA

Figure 2-12
City Of Grants Pass
WTP Facility Plan
Existing Filter Cross-Sections

SOE 306

906 306

Figure 2-13
Grants Pass WTP:
1999-2003-- Daily Maximum Finished Water Turbidity

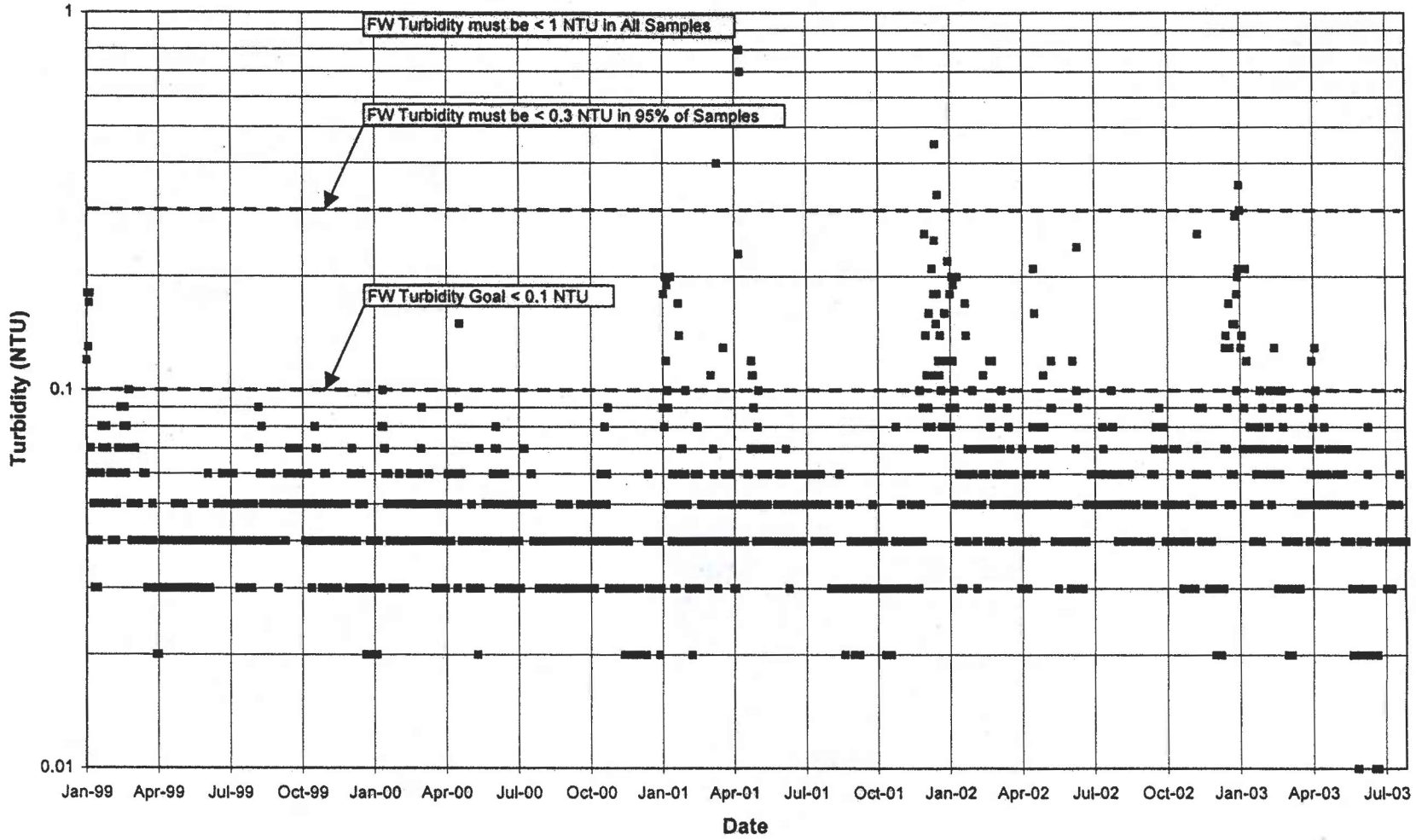
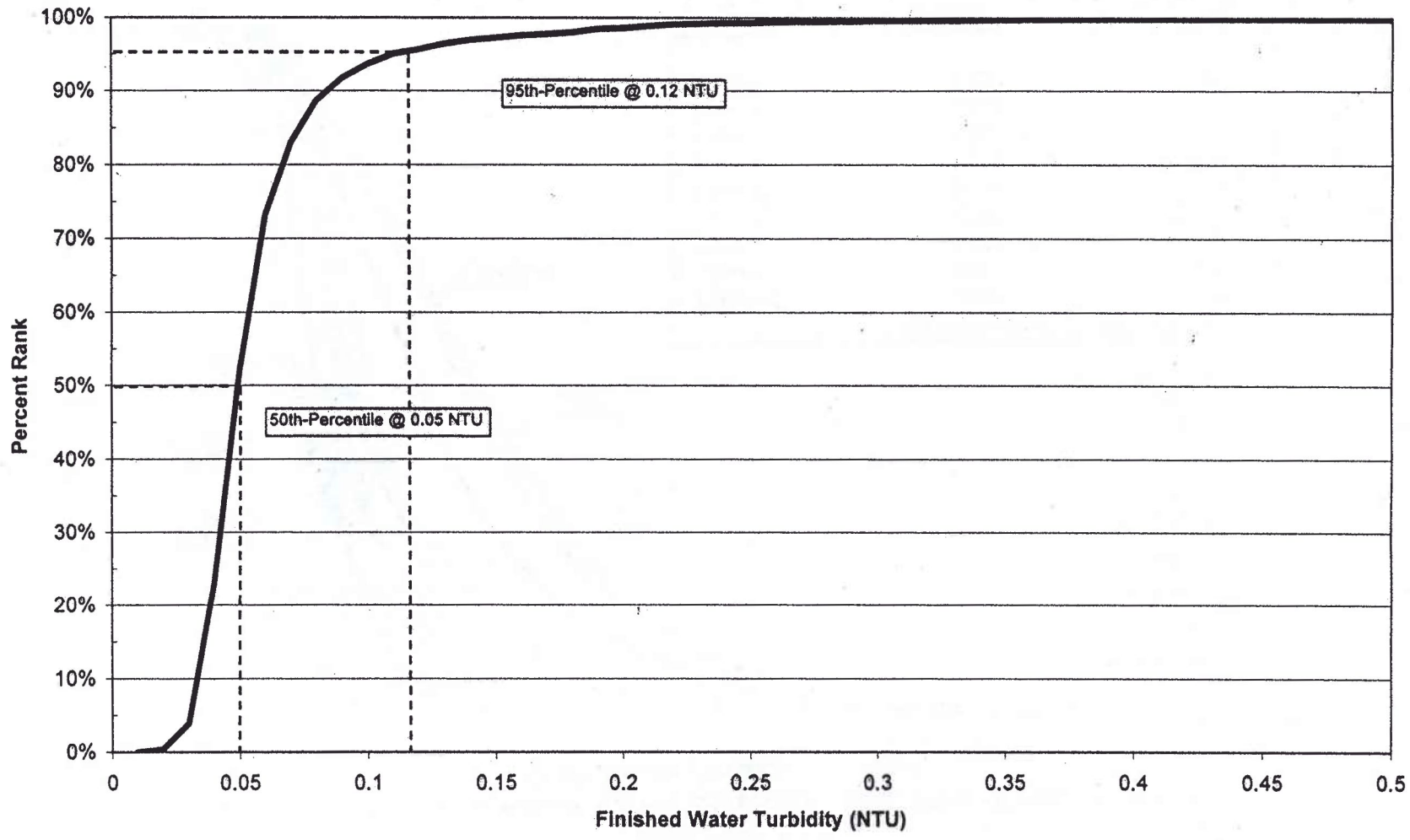


Figure 2-14
Grants Pass WTP:
1999-2003-- Average Daily Plant Finished Water Turbidity Probability Distribution



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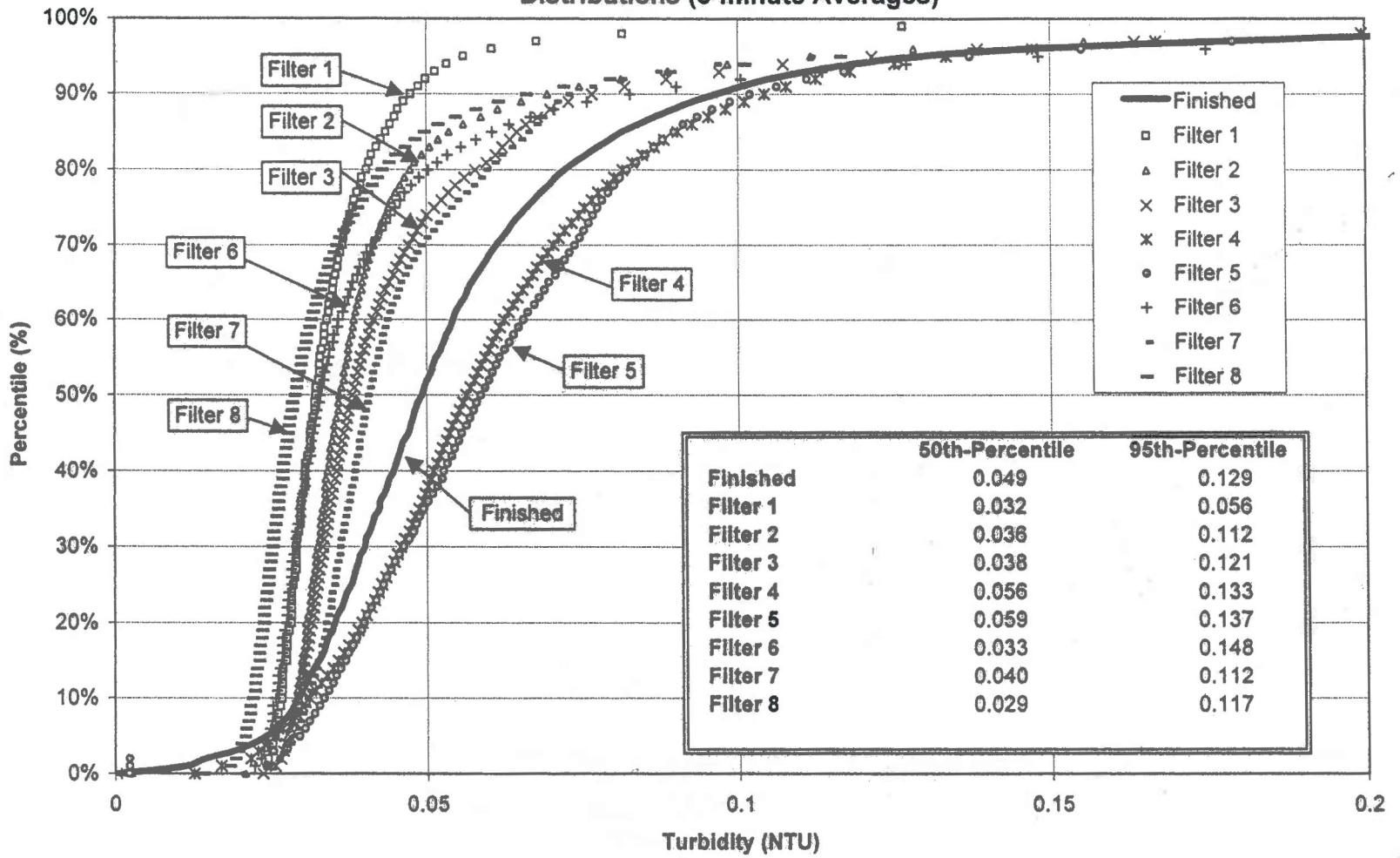
808 360

Figure 2-15
Grants Pass WTP:
April 2002 - June 2003 -- Individual Filter Effluent and Plant Effluent Turbidity Probability Distributions (5-minute Averages)



	50th-Percentile	95th-Percentile
Finished	0.049	0.129
Filter 1	0.032	0.056
Filter 2	0.036	0.112
Filter 3	0.038	0.121
Filter 4	0.056	0.133
Filter 5	0.059	0.137
Filter 6	0.033	0.148
Filter 7	0.040	0.112
Filter 8	0.029	0.117

Figure 2-16
Grants Pass WTP
Influence of Limiting Unit Filter Run Volume and Unit Backwash Volume on Production Efficiency

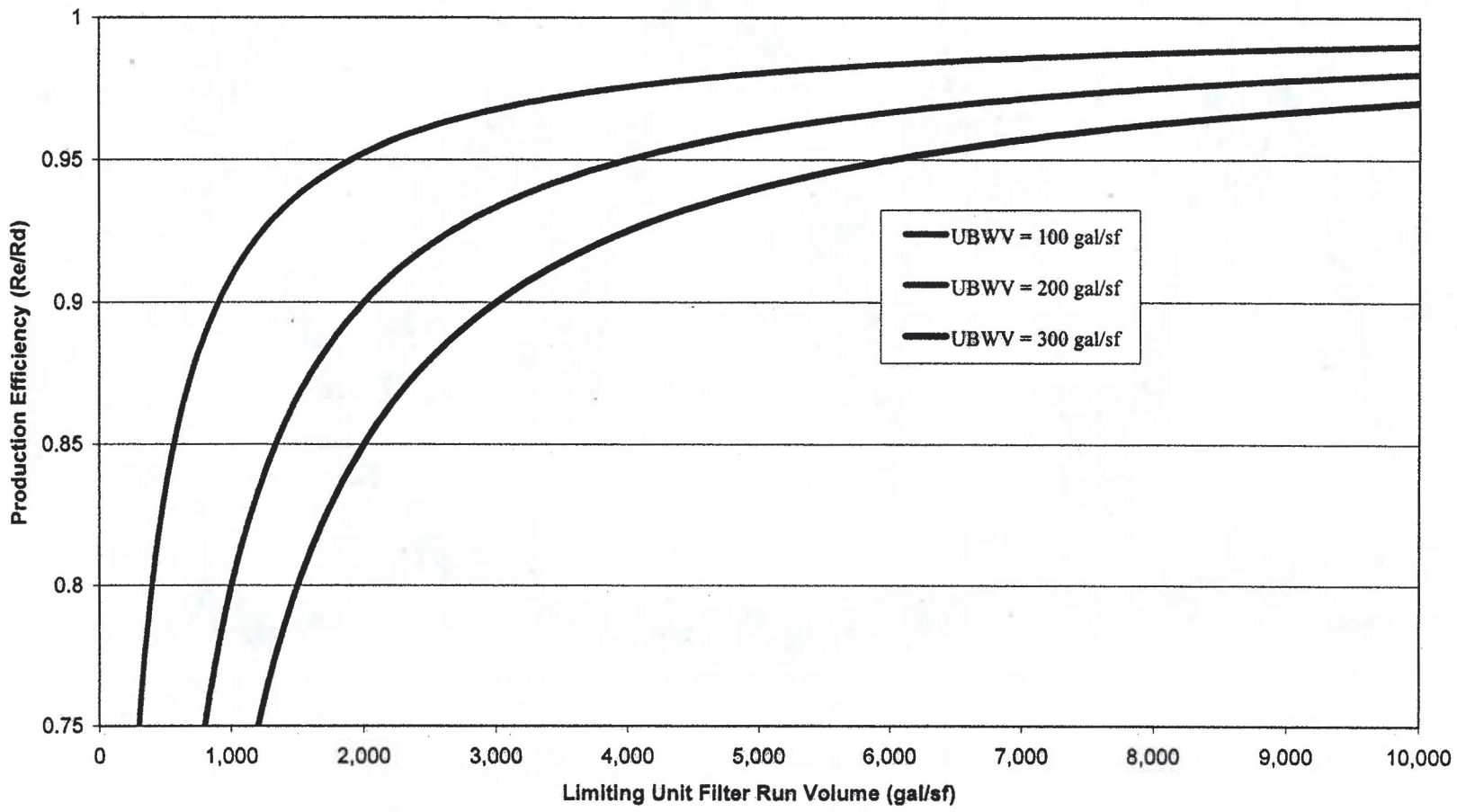


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Figure 2-17
Grants Pass WTP:
1999-2003-- Plant Weekly Average Filter Production Efficiency and Weekly Backwash Volume

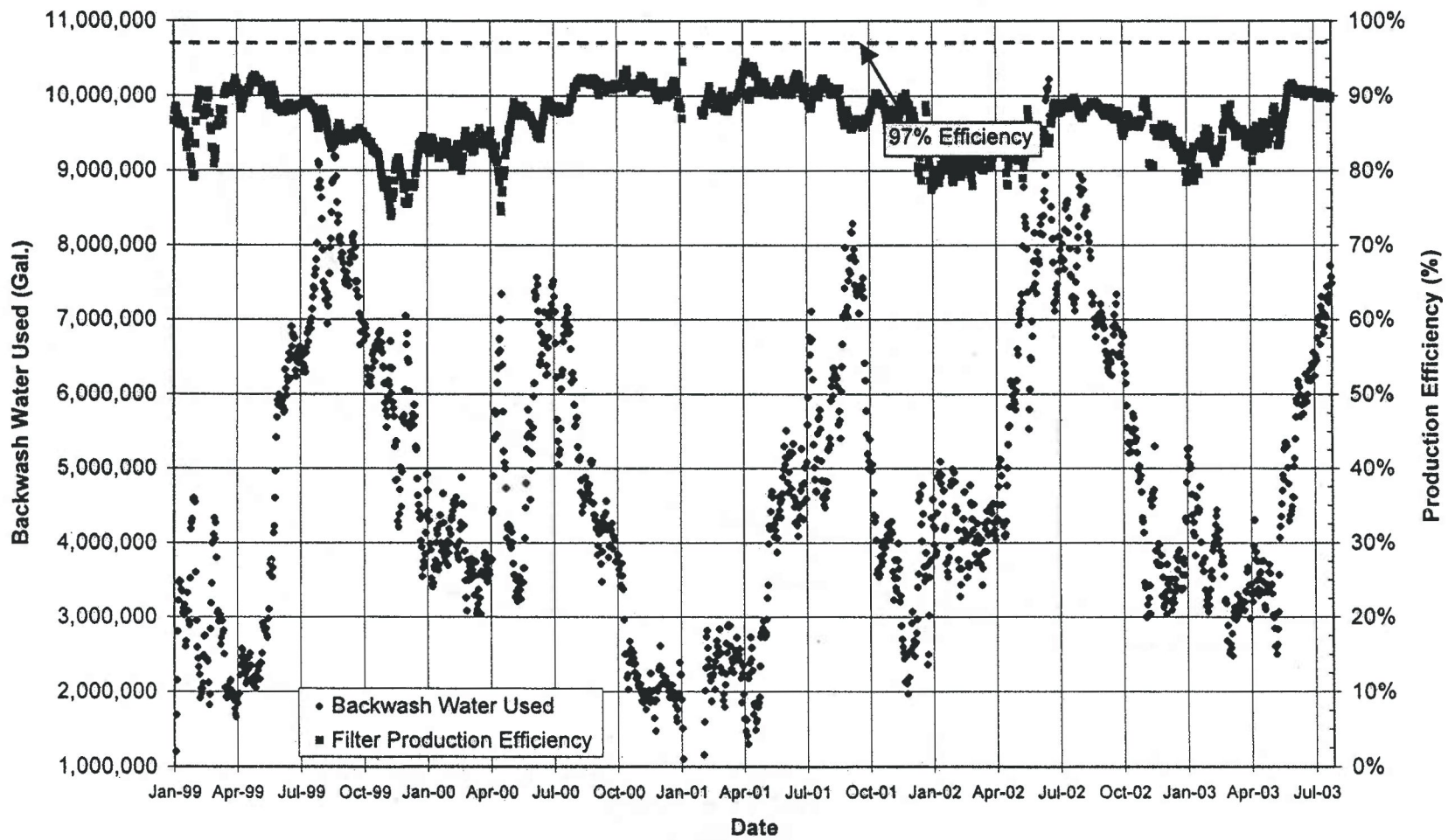
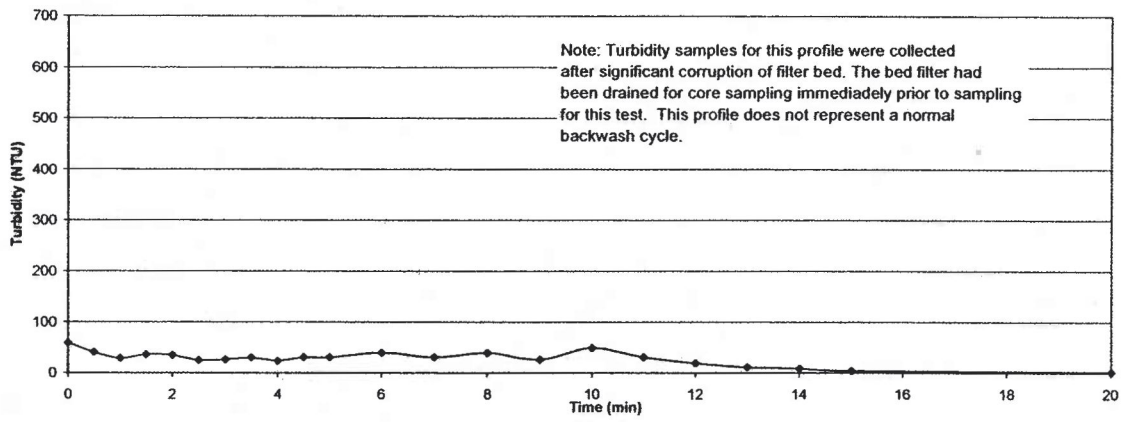
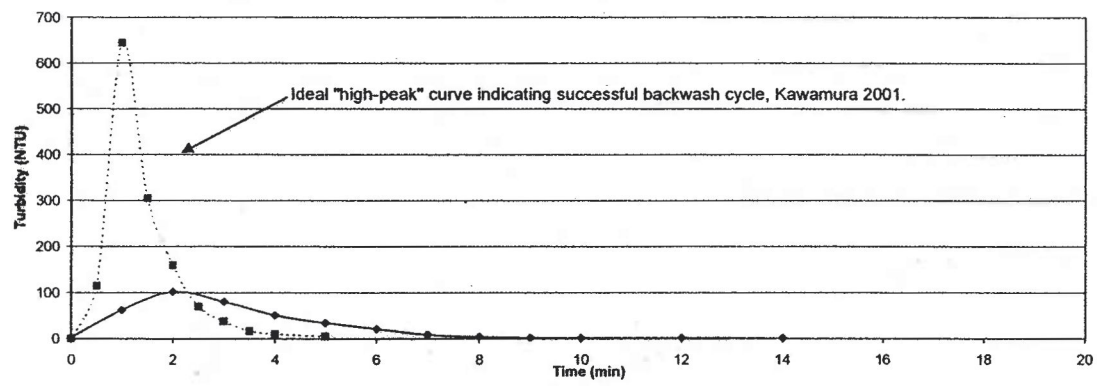


Figure 2-18
Grants Pass WTP: Filter Performance Evaluation
Backwash Turbidity Profiles

Filter No. 3 (Black & Veatch) May 28, 2003



Filter No. 5 (MWH) July 30, 2003



Filter No. 7 (MWH) July 30, 2003

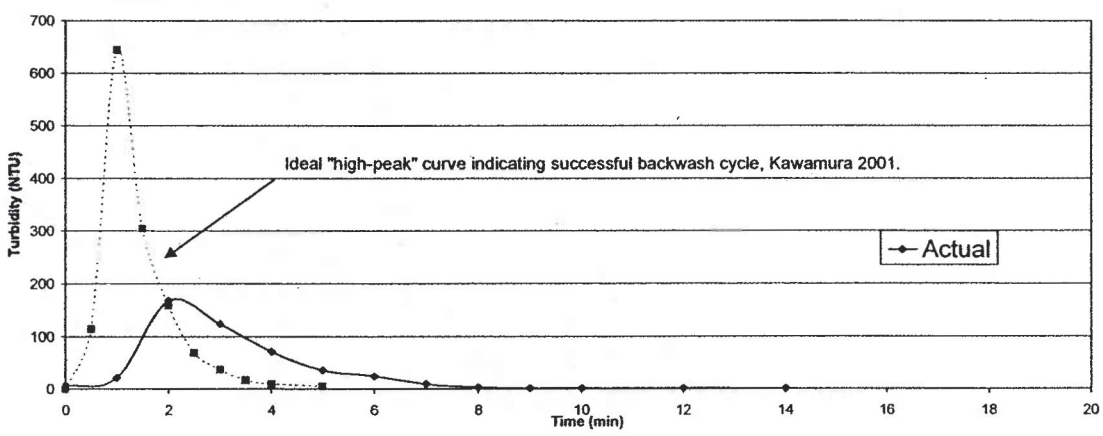
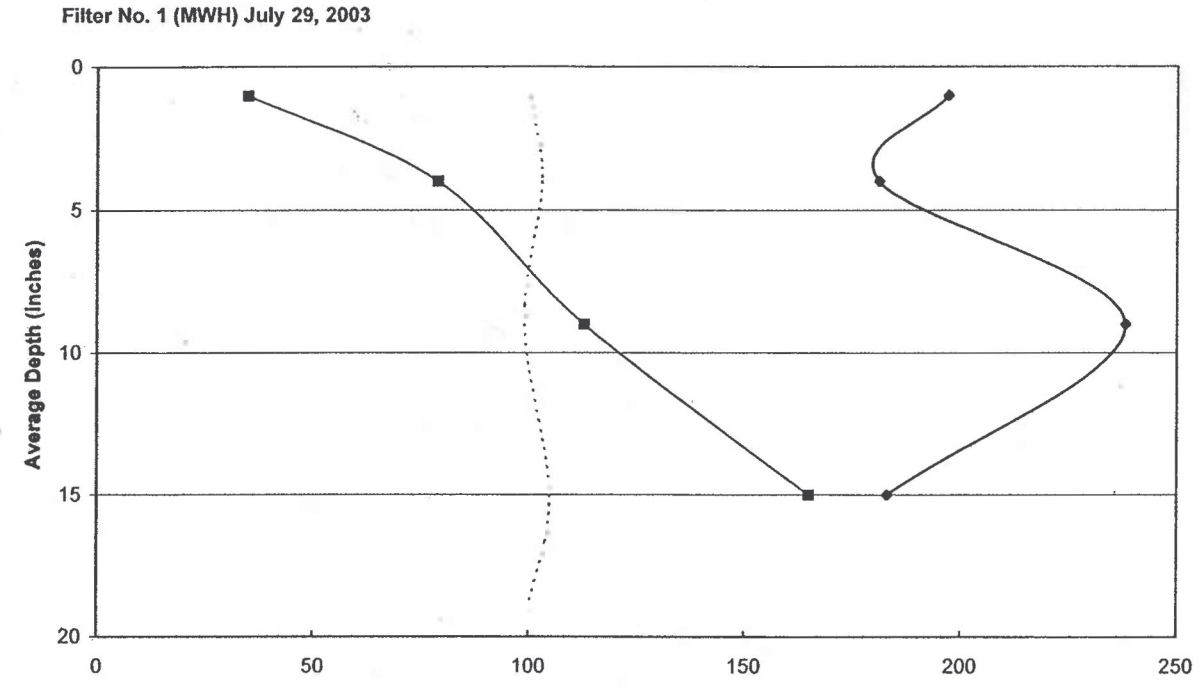


Figure 2-19:
Grants Pass WTP: Filter Performance Evaluation
Floc Retention Profiles



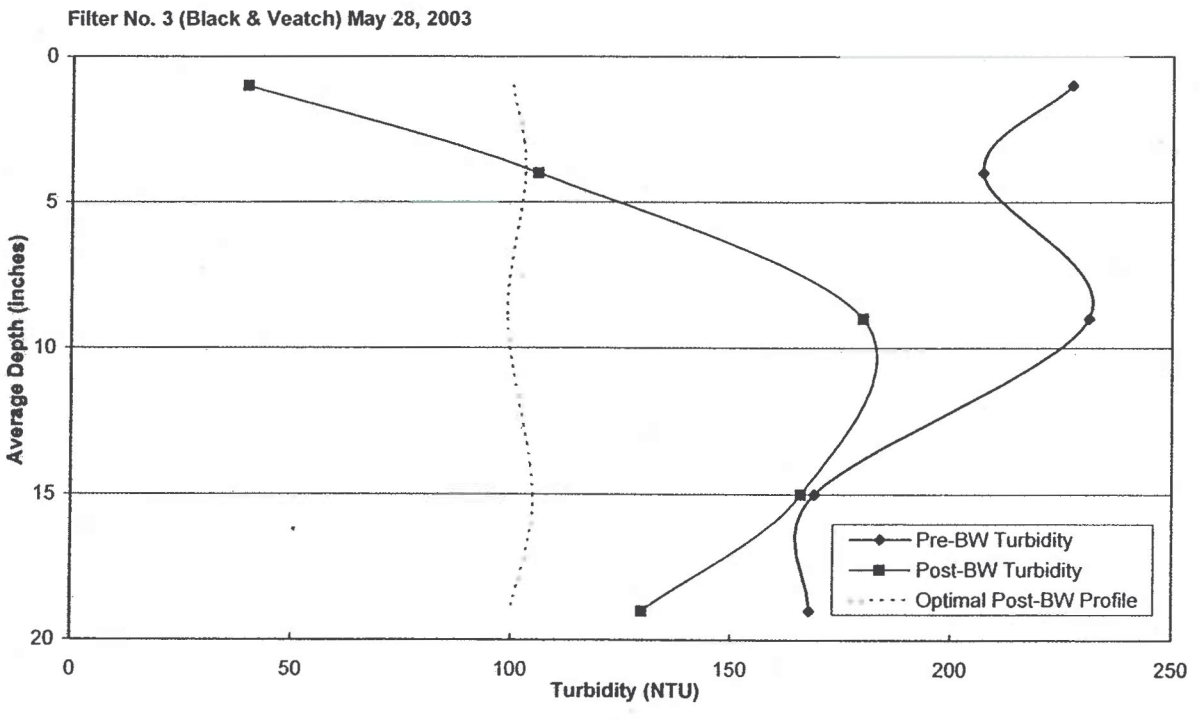
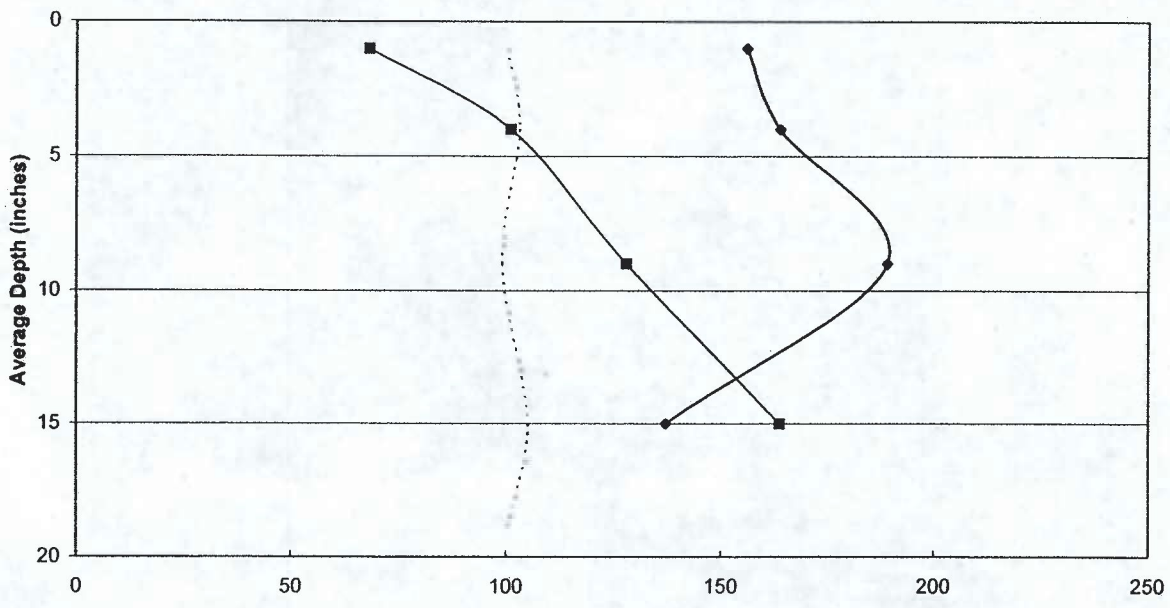
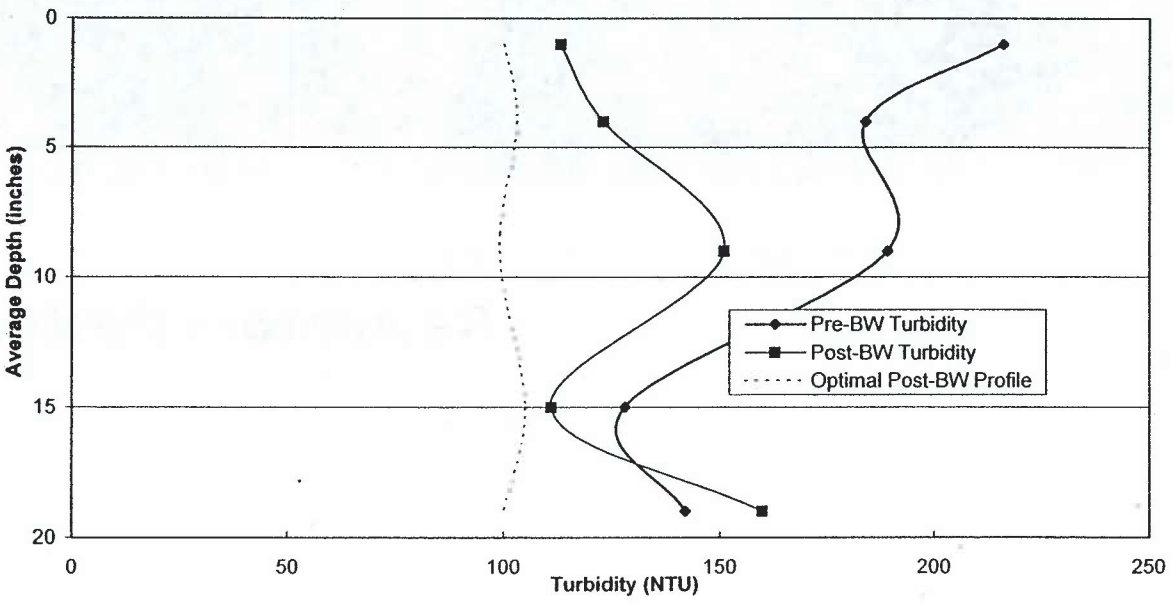


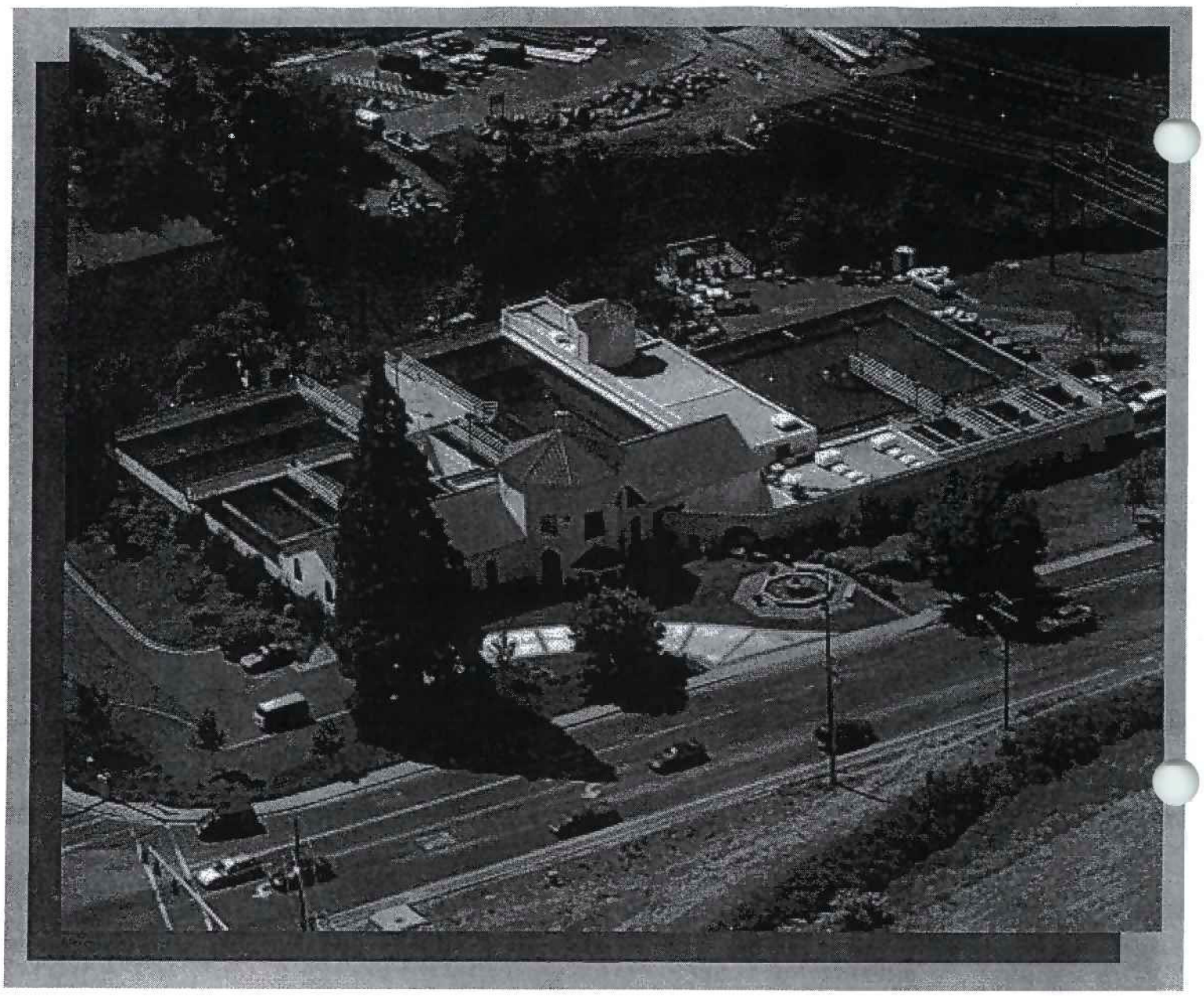
Figure 2-19 (Cont.)
Grants Pass WTP: Filter Performance Evaluation
Floc Retention Profiles

Filter No. 5 (MWH) July 29, 2003



Filter No. 7 (MWH) July 29, 2003





Regulatory Review

3 REGULATORY REVIEW

This section provides a general overview of current drinking water regulations under the Oregon Drinking Water Quality Act (OAR 333-061 – Rules for Public Water Systems), as well as anticipated future regulations. In addition, other regulatory compliance issues, including National Pollutant Discharge Elimination System (NPDES) and Endangered Species Act (ESA) are reviewed. The discussion of each regulation is followed by an assessment of historic compliance, or in the case of future regulations, anticipated compliance. Recommended process/monitoring improvements to ensure continued compliance with all existing and anticipated regulatory requirements are discussed where appropriate. This regulatory summary is current as of July 2003.

3.1 EXISTING DRINKING WATER REGULATIONS

Currently enforced national drinking water regulations that have implications for the City of Grants Pass WTP (City) are listed below:

- National Primary Drinking Water Regulations (1975)
- Secondary Drinking Water Regulations (1979, 1991)
- Phase I, II, and V Regulations for IOCs, SOCs, and VOCs (1987, 1991, 1992, respectively)
- Surface Water Treatment Rule (1989)
- Total Coliform Rule (1989)
- Lead and Copper Rule (1991)
- Consumer Confidence Reports Rule (1998)
- Stage 1 Disinfectants/Disinfectant By-Product Rule (1998) – supercedes Total Trihalomethane Rule (1979)
- Interim Enhanced Surface Water Treatment Rule (1999)
- Unregulated Contaminants Monitoring Rule (1999)

With the exception of the Unregulated Contaminants Monitoring Rule, the water quality standards established under these national regulations have been adopted into the Oregon

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REGULATORY REVIEW

Drinking Water Quality Act (OHS 333-061) by the Department of Human Services (DHS) Drinking Water Program (formerly Oregon Health Division). In addition to implementation, DHS is also responsible for enforcing these national water quality standards. If a system is found to be in violation, DHS will issue a Notice of Violation. If violations are accumulated, the system is considered a "significant non-complier", and an administrative order (for monitoring violations), or remedial order (where plant improvements are required), is issued. A schedule for compliance is included in the order. If the schedule is not met, civil penalties (i.e. fines) will be issued. Enforcement of the Unregulated Contaminants Monitoring Rule has recently become the responsibility of the US EPA.

There are currently drinking water quality standards for 95 primary and 12 secondary contaminants in the State of Oregon. Under the Oregon Drinking Water Quality Act, each contaminant has either an associated established maximum contaminant level (MCL) or recommended treatment technique (TT). These contaminants are grouped into the following general categories.

- Microbial Contaminants,
- Disinfectants and Disinfection By-Products,
- Inorganic Chemicals,
- Organic Chemicals, and
- Radiologic Contaminants.

Table 3-1 summarizes the primary and secondary drinking water contaminants regulated under Oregon Drinking Water Quality Act. Note that not every contaminant has a corresponding MCL; some contaminants have recommended TT in lieu of an MCL. The following is a discussion of these state-regulated contaminants, as well as the federally monitored unregulated contaminants.

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TABLE 3-1
OREGON DRINKING WATER ACT (333-061-0030):
MAXIMUM CONTAMINANT LEVELS AND ACTION LEVELS

Contaminant	MCL[a]	Sampling Frequency
Inorganic Contaminants (IOCs)		
Antimony	0.006	Annually
Arsenic	0.05	Annually
Asbestos (fibers > 10µm)	7 MFL	9 years
Barium	2.0	Annually
Beryllium	0.004	Annually
Cadmium	0.005	Annually
Chromium (total)	0.1	Annually
Copper	1.3[1]	see text
Cyanide	0.2	Annually
Fluoride	4.0	Annually
Lead	0.015[1]	see text
Mercury	0.002	Annually
Nickel	0.1[2]	Annually
Nitrate (as N)	10.0	Quarterly
Nitrate+ Nitrite (as N)	10.0	Quarterly
Nitrite (as N)	1.0	Quarterly
Selenium	0.05	Annually
Thallium	0.002	Annually
Organic (Synthetic) Compounds (SOCs)		
Acrylamide	TT	Annually, if applicable
Alachlor	0.002	Twice in 3 years
Atrazine	0.003	Twice in 3 years
Benzo(a)pyrene (PAHs)	0.0002	Twice in 3 years
Carbofuran	0.04	Twice in 3 years
Chlordane	0.002	Twice in 3 years
2,4-D	0.07	Twice in 3 years
Dalapon	0.2	Twice in 3 years
Di (2-ethylhexyl) adipate	0.5	Twice in 3 years
Di (2-ethylhexyl) phthalate	0.006	Twice in 3 years
Dinoseb	0.007	Twice in 3 years
Diquat	0.02	Twice in 3 years
Endothall	0.1	Twice in 3 years
Endrin	0.002	Twice in 3 years
Epichlorohydrin	TT	Annually, if applicable
Ethylene dibromide (EDB)	0.00005	Twice in 3 years
Glyphosate	0.7	Twice in 3 years
Heptachlor	0.0004	Twice in 3 years
Heptachlor epoxide	0.0002	Twice in 3 years
Hexachlorobenzene	0.001	Twice in 3 years
Hexachlorocyclopentadiene	0.05	Twice in 3 years
Lindane	0.0002	Twice in 3 years
Methoxychlor	0.4	Twice in 3 years
Oxymyl (Vydate)	0.2	Twice in 3 years
Pentachlorophenol	0.001	Twice in 3 years
Picloram	0.5	Twice in 3 years
Polychlorinated biphenyls (PCBs)	0.0005	Twice in 3 years
Simazine	0.004	Twice in 3 years
2,3,7,8,-TCDD (Dioxin)	0.00000003	Risk dependent
Toxaphene	0.005	Twice in 3 years
2,4,5-TP (Silvex)	0.05	Twice in 3 years

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TABLE 3-1
OREGON DRINKING WATER ACT (333-061-0030):
MAXIMUM CONTAMINANT LEVELS AND ACTION LEVELS

Contaminant	MCL[a]	Sampling Frequency
Organic (Volatile) Contaminants (VOCs)		
Benzene	0.005	Annually
Carbon tetrachloride	0.005	Annually
Dibromochloropropane(DBCP)	0.0002	Annually
p-Dichlorobenzene	0.075	Annually
o-Dichlorobenzene	0.6	Annually
1,2-Dichloroethane	0.005	Annually
1,1-Dichloroethylene	0.007	Annually
cis-1,2-Dichloroethylene	0.07	Annually
trans-1,2 Dichloroethylene	0.1	Annually
Dichloromethane	0.005	Annually
1,2-Dichloropropane	0.005	Annually
Ethylbenzene	0.7	Annually
Styrene	0.1	Annually
Tetrachloroethylene	0.005	Annually
Toluene	1.0	Annually
1,2,4-Trichlorobenzene	0.07	Annually
1,1,1-Trichloroethane	0.2	Annually
1,1,2-Trichloroethane	0.005	Annually
Trichloroethylene	0.005	Annually
Vinyl chloride	0.002	Annually
Xylenes (total)	10.0	Annually
Radionuclides		
Gross alpha	15 pCi/L	4 years
Beta particle/photon activity	4 mrem/yr	4 years
Iodine - 131	3 pCi/L	4 years
Radium-226 + 228	5 pCi/L[3]	4 years
Strontium 90	8 pCi/L	4 years
Tritium	20,000 pCi/L	4 years
Uranium	30 ug/L	
Disinfectant Residuals and Disinfection By-Products (DBPs)		
Raw Water Total Organic Carbon	-	Monthly
Bromate	0.01	Quarterly
Chlorite	1.0	Quarterly
Haloacetic Acids (HAA_5)	0.06	Quarterly
Monochloroacetic Acid	-	-
Dichloroacetic Acid	-	-
Trichloroacetic Acid	-	-
Monobromoacetic Acid	-	-
Dibromoacetic Acid	-	-
Total Trihalomethanes (TTHM)	0.08	Quarterly
Bromodichloromethane	-	-
Bromoform	-	-
Chloroform	-	-
Dibromochloromethane	-	-

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TABLE 3-1
OREGON DRINKING WATER ACT (333-061-0030):
MAXIMUM CONTAMINANT LEVELS AND ACTION LEVELS

Contaminant	MCL [a]	Sampling Frequency
Microbial Contaminants		
Giardia lamblia	TT	-
Cryptosporidium	TT	-
Legionella	TT	-
Heterotrophic plate count	TT	-
Turbidity	TT	see text
Viruses	TT	-
Total Coliform	< 5% positive	40/month
Fecal Coliform	Confirmed Presence	-
E. Coli	Confirmed Presence	If TC Positive
Secondary (Recommended) Standards		
Color-Color Units	15	-
Corrosivity	Non-corrosive	-
Foaming Agents	0.5	-
pH	6.5 - 8.5	-
Hardness (as $CaCO_3$)	250	-
Odor	3 TON	-
Total Dissolved Solids	500	-
Aluminum	0.05 -0.2	-
Chloride	250	-
Fluoride	2.0	-
Iron	0.3	-
Manganese	0.05	-
Silver	0.1	-
Sulfate	250	-
Zinc	5.0	-

[a] Values reported in mg/L, unless otherwise specified
[1] Action Level
[2] MCL currently being re-evaluated by the EPA

3.1.1 Microbial Contaminants

3.1.1.1 Regulatory History

The National Primary Drinking Water Regulations (NPDWR) (December, 24, 1975) represented the first set of drinking water regulations promulgated by the United States Environmental Protection Agency (EPA); the MCLs established in the NPDWR were adopted into Oregon Law September 24, 1982. However, the microbial requirements outlined in the NPDWR have since been superceded by new federal regulations. The Total Coliform Rule, published on the Federal Register on June 16, 1989 and adopted in Oregon on January 1, 1991, supercedes the coliform requirements established in the

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NPDWR, and includes microbial testing and control measures. Similarly, increasingly rigid requirements for turbidity have evolved since the adoption of the NPDWR. The Surface Water Treatment Rule (SWTR) (June 29, 1989) and the Interim Enhanced Surface Water Treatment Rule (IESWTR) (December 16, 1998), adopted in Oregon on January 1, 1991 and July 15, 2000, respectively, both supercede the NPDWR and outline improved filter monitoring and performance, as well as disinfection requirements.

3.1.1.2 Monitoring Requirements – Coliform Bacteria

The Oregon Drinking Water Quality Act requires that the City collect a minimum of 25 samples per month from representative sites throughout the distribution system. If a routine sample is positive for total coliform, the City must collect a set of three repeat samples: one from the original site, one within 5 service connections upstream of the original site, and one within 5 service connections downstream of the original site.

The repeat samples must be collected within 24 hours of notification of the positive result. Further, any routine or repeat coliform positive samples must be analyzed for the presence of fecal coliform or *E. coli* as an indicator organism. When a system learns of the presence of fecal coliform or *E. coli*, the system must notify the State by the end of the same day.

In Oregon, the total coliform MCL is violated if:

1. More than 1 sample collected within a single month are coliform positive (non-acute violation),
2. A repeat sample following a total coliform positive contains fecal coliform or *E. coli* (acute violation), or
3. A repeat sample following a fecal coliform positive or *E. coli* positive contains total coliform (acute violation).

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3.1.1.3 Monitoring Requirements – Surface Water Treatment

All public water systems using surface water sources are required to comply with the Oregon Drinking Water Quality Act's treatment performance and disinfection requirements. Four specific areas are addressed within the Act, including:

- Overall filtration performance,
- Individual filtration performance,
- Disinfection performance, and
- Disinfection profiling and benchmarking.

These are discussed in detail below.

Overall Filtration Performance: Current overall filtration performance standards require that the turbidity measurements from the combined filter effluent must be measured in four hour intervals by grab sampling or continuous monitoring. 95 percent of these turbidity readings must be less than or equal to 0.3 NTU, and may never exceed 1.0 NTU. In addition, treatment strategies, in combination with disinfection, must consistently remove/inactivate 99.9 percent (3-log) of *Giardia*, 99.99 percent (4-log) of viruses and 99 percent (2-log) removal (i.e. no inactivation) of *Cryptosporidium*. Each utility is required to submit a report to the State on a monthly basis and identify any exceptions.

Individual Filter Performance: Oregon law requires continuous, on-line measurement of turbidity for each individual filter. This data must be recorded every fifteen minutes. If there is a failure in the turbidity monitoring equipment, the system may conduct grab sampling every 4 hours in lieu, but for not more than five working days following the failure. Each utility is required to submit a report to the State on a monthly basis and identify any exceptions. Exceptions under Oregon law occur when:

1. Individual filter effluent turbidity exceeds 1.0 NTU in two consecutive measurements, 15 minutes apart at any time during the filter operation.

2. Individual filter effluent turbidity exceeds 0.5 NTU in two consecutive measurements, 15 minutes apart, after 4 hours of operation following backwash
3. If the individual filter effluent turbidity exceeds 1.0 NTU in two consecutive measurements, 15 minutes apart, at any time during the filter operation for three consecutive months.
4. If the individual filter effluent turbidity exceeds 2.0 NTU in two consecutive measurements, 15 minutes apart, at any time during the filter operation for two consecutive months.

Disinfection Performance: The Oregon Drinking Water Quality Act requires all utilities served by a surface water supply to achieve a minimum of 99.9 percent (3-log) reduction in *Giardia lamblia* cysts, 99.99 percent (4-log) reduction in viruses and 99 percent (2-log) removal of *Cryptosporidium* cysts during drinking water treatment. Removal credit is awarded to WTPs based on the types of processes provided by the plants. For conventional plants with filter to waste capabilities, such as the Grants Pass WTP, a 2.5-log, 2.0-log and 2.0-log removal credit is usually granted for *Giardia lamblia*, viruses and *Cryptosporidium*, respectively. The remaining reduction in pathogenic organisms must come in the form of disinfection and/or inactivation. For Grants Pass, a minimum of 0.5-log inactivation of *Giardia* and 2.0-log inactivation of viruses is required prior to the first customer; *Giardia* inactivation typically governs disinfection through the WTP.

In order to determine the level of inactivation achieved during chemical disinfection, the EPA developed the "CT" concept. "CT" is the product of disinfectant residual measured at the outlet of a disinfection section and the time in which 10 percent (by volume) of an added tracer passes through the section, known as the T_{10}. To remain in compliance with disinfection performance standards, the following criteria must be met:

1. Disinfection residual must be continuously recorded at the entry point to the distribution system, and must never fall below 0.2 mg/L.

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2. CT must be calculated every day. To ensure that the values are conservative, the highest flow rate and minimum clearwell volume recorded for the day must be used in the calculation; tracer studies should be used to verify hydraulic efficiencies through the various treatment trains.
3. CT calculated must be sufficient to meet the needed removal/inactivation levels.
4. The residual disinfectant concentration in the distribution system cannot be undetectable in more than 5 percent of the samples. For simplicity, samples should be collected at coliform bacteria monitoring points.

Disinfection Profiling and Benchmarking: The purpose of disinfection profiling and benchmarking is to develop a process to assure that there is no significant reduction in microbial protection as a result of major disinfection process modifications. Disinfection process modification may be driven to meet the new MCLs for total trihalomethane (TTHMs) and five haloacetic acids (HAA_5) from the recently adopted Disinfectants/Disinfection By-products Rule. Surface water systems serving 10,000 people or more were required to develop four quarters of TTHM and HAA_5 data by April 2001. If the observed TTHM or HAA_5 RAA exceed 80-percent of the new MCLs (≥0.064 mg/L and/or ≥0.048 mg/L for TTHM and HAA_5, respectively), a disinfection profile will need to be developed. The preliminary DBP data submitted by Grants Pass is presented and discussed in the Disinfectant/Disinfection By-product portion of this regulatory review.

The disinfection profile is developed using a minimum of one year of daily *Giardia lamblia* log inactivation. Daily log inactivations are used to calculate the average monthly log inactivation. The month with the lowest average log inactivation will be identified as the critical period or benchmark. This profile and benchmark must be submitted to the State; if a utility decides to make changes to the disinfection practices, then the utility must consult with the State to ensure that microbial protection is not compromised. The City completed its profile using four years of *Giardia* inactivation data tabulated by month (1999-2002) and submitted to DHS in compliance with the rule.

3.1.1.4 Analysis of Grants Pass's Compliance History, Coliform Rule

Coliform Bacteria: Historic microbial testing results for the City were obtained through the DHS; these results date back as far as January 1995. Three coliform sampling violations are on record at the DHS, dated February 28, 1995, October 31, 1995 and July 31, 1996. In all cases, violations correspond to an inadequate number of samples submitted to the State. No violations with regard to coliform presence in drinking water are on record. In fact, no coliform has been detected in any of the submitted samples to date. Historic treatment data indicates consistent compliance with the Oregon Drinking Water Quality Act's coliform bacteria requirements.

3.1.1.5 Analysis of Grants Pass's Compliance History --Surface Water Treatment

Overall Filter Performance: Combined filtered water turbidity is measured prior to the point of entry into the distribution system. A statistical analysis was performed on the average daily filtered water turbidity data collected from January 1999 through July 2003 to determine regulatory compliance. ***Figure 2-14*** presents the results of this statistical analysis. *[Please note: regulations in place between January 1999 and January 2000 required combined filter effluent turbidity to be less than 0.3 NTU in 95 percent of the measurements, never to exceed 1.0 NTU, and had no requirements for individual filter performance.]*

From ***Figure 2-14,*** turbidity values of 0.119 NTU are achieved 95 percent of the time, consequently the City has met and/or exceeded all regulatory filtration standards in place at the time the data was collected.

Individual Filter Performance: The on-line turbidimeters necessary for monitoring the individual filtered water turbidity have been installed at the City's WTP. ***Figure 2-15*** presents a statistical summary of individual filter performance measured at 5 minute intervals between 9 a.m. and 3 p.m. between April 2002 and July 2003. The data indicates that there are no "problem" filters; all filters are performing well with regard to the new regulatory requirements.

Disinfection Performance: CT-*achieved* through the WTP is calculated daily. Once calculated, this value is compared to the CT-*required*; if CT-achieved is greater than the CT-required, then compliance is achieved. The CT-required value is based on the CT tables presented in the SWTR Guidance Manual for 0.5-log inactivation of *Giardia* with free chlorine (included in ***Appendix A***), maximum daily chlorine residual, minimum daily raw water temperature and maximum daily pH. ***Figure 3-1*** presents the historic results of both the required and calculated values for log-inactivation for the Grants Pass WTP. As shown in the Figure, CT was consistently met at the Grants Pass WTP during the January 1999 to August 2003 period of record evaluated for this study. Also, the Grants Pass WTP has no violations with regard to disinfection residual monitoring or residual concentrations in the distribution system.

The following equations were historically used to calculate CT-achieved through the plant:

1. **T** (min) = $\dfrac{[T_{10}/T_{Basin}\ (\text{Reactor Basin Volume}) + T_{10}/T_{CW}\ (\text{Clearwell Volume})](\text{gal})}{\text{Plant Flow (gpm)}}$
2. **C** (mg/L) = Minimum In-plant Chlorine Residual
3. $\mathbf{CT}_{achieved}$ (mg/L-min) = **C x T**

Where:

T_{10}/T_{Basin} = 0.5 (OHD *1993 Comprehensive Performance Evaluation*, 1993),

T_{10}/T_{CW} = 0.7 (OHD *1993 Comprehensive Performance Evaluation*, 1993),

Plant Flow = Maximum Instantaneous Raw Water Flow for the day in question.

Assumptions inherent in the above equation follow:

- Surface overflow rate and T_{10}/T through each Basin is equal (i.e. detention time through each basin is equal).
- No CT is achieved through the filters or HSPS.
- Water quality parameters affecting the CT-required (i.e. pH, water temperature, chlorine concentrations) do not change through the treatment plant.

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On June 24th, 2003, a tracer study was performed on the existing clearwell (B&V, July 2003). A copy of this study is included in ***Appendix B***; results from this study are summarized below:

- At a plant flow rate of 10.5 mgd (2 raw water pumps on-line), $T_{10}/T = 0.60$
- At a plant flow rate of 20.0 mgd (4 raw water pumps on-line), $T_{10}/T = 0.50$

These T_{10}/T values are lower than those previously assigned during the 1993 Comprehensive Performance Evaluation, and will reduce the level of CT-achieved through the WTP. On July 24, 2003, City staff met with a representative from the Oregon Department of Human Services (DHS) Drinking Water Program to discuss incorporating these results into the CT calculations. At that time, a conservative value for T_{10}/T of 0.50 was adopted for the clearwell.

CT Recommendations: Adjustments to the way in which CT is calculated at the plant to more accurately represent actual microbial inactivation will offset the hydraulic inefficiencies in the clearwell. Historically, to determine CT-required through the plant, the "worst case" conditions (i.e. highest pH, lowest temperature, and highest chlorine residual) throughout the WTP must be considered. Since chemicals affecting these parameters are often added at various stages in the treatment train, the City may benefit from breaking the overall treatment train into various "disinfection sections"; these disinfection sections are defined by the points of chemical injections. For example, if pH is adjusted from 7.0 to 7.5 in the combined filtered water effluent, the existing CT calculation would require that a pH of 7.5 be considered in determining CT-required throughout the entire plant. By defining the Basins as a distinct disinfection section, a pH of 7.0 (the measured pH through the Basins) can be used when determining CT-required through the Basins, significantly reducing the CT-required for this section, increasing the overall log inactivation. CT through the filters could also be considered. This approach would involve the incorporation of measured (either grab or on-line) values of chlorine residual, pH and temperature at the "end" of individual disinfection sections into the

overall CT calculation. In addition, CT calculations for disinfection sections are slightly more complicated than those previously used at the WTP.

CT Calculation and Optimization: To assist the City in calculating CT using "disinfection sections", an electronic CT model was prepared by MWH; a CD containing this model was delivered to the WTP supervisor. This model allows an operator to input measured values within the plant (i.e. plant flows, pH, chlorine concentration, water temperature, etc.) for various disinfection sections throughout the treatment process train. Based on these parameters, the model calculates the overall log-inactivation achieved through each component of the treatment process train, as well as overall inactivation through the plant. This calculation involves interpolations of CT-required values presented in the SWTR Guidance Manual of 0.5-log inactivation of Giardia. These tables are included as a worksheet in the CT model; a hard copy of the tables are provided in ***Appendix A***. Hydraulic efficiencies through the clearwell were taken from the recent tracer test studies at the plant (B&V July, 2003). Though the model was designed to help operators calculate CT compliance at the plant, it can also serve as a tool to help establish seasonal CT trends and optimize overall plant performance (i.e. adequate microbial inactivation with limited disinfection by-product formation). The following analysis was performed help optimize CT through the WTP.

Two water treatment scenarios typically create challenges for CT compliance: Winter conditions (low temperatures at relatively low flows), and Summer conditions (high temperatures at relatively high flows). The CT model was used to help summarize pre-chlorination constraints during these "worst-case" conditions, as well as moderate conditions in the spring/fall. ***Table 3-2*** presents the ranges of conditions that were assumed throughout this analysis.

TABLE 3-2: WATER QUALITY AND FLOW RANGES CONSIDERED FOR "WORST-CASE" CT ANALYSIS

Parameter	Units	Summer	Spring/Fall	Winter
Pre-Chlorine Residual at Filter Influent	mg/L	0.1 – 0.8	0.1 - 1.0	0.1 - 1.2
Minimum Temperature[1]	°C	15.0	10.0	5.0
pH	-	6.5 - 8.0	6.5 - 8.0	6.5 - 8.0
Flow	MGD	5 - 20	5 - 20	5 - 20

[1]Temperatures represent the "worst-case" (i.e. coldest) water temperatures observed during the seasons.

In addition to the above parameters, the following assumptions were made throughout this analysis:

- A finished water pH of 7.2 and finished water chlorine concentration of 1.0 mg/L were maintained through the clearwell.
- Filtration rates (gpm/sf) were assumed constant through each of the 8 filters; flows through the Basins were assumed to be proportional to the settling area of each basin (i.e. 36%, 24% and 40% of the plant flow is directed to Basin #1, #2 and #3, respectively.) T_{10}/T values of 0.5 were assumed through each of these basins based on SWTR Guidance Manual recommendations for well baffled basins.
- Calculations were performed with all filters on-line; CT-achieved through the filters, though relatively small, was considered in the overall CT-achieved through the plant. No CT credit was given to the wetwell beneath the filters.
- Water temperature does not change throughout the plant (i.e. FW temp = RW temp).
- Clearwell level was maintained at 13.5 feet.

Preliminary results from the CT analysis for the Grants Pass WTP are discussed in the following subsections. Please note: this analysis is limited to those "worst-case" temperatures presented in ***Table 3-2***. Since overall CT requirements are highly temperature dependent, this analysis should only be used to help establish trends in plant CT performance.

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Winter Conditions: ***Figure 3-2*** presents the required pre-chlorination residual at the filter inlet needed to maintain at least 0.5-log inactivation of *Giardia lamblia* over a range of flowrates and pH values for the "worst-case" Winter conditions (5 °C). As shown in the Figure, at low flows (≤10 mgd), the plant can tolerate a wide range in pH, while maintaining a relatively low chlorine residual throughout the basins (< 0.3 mg/L, as measured at the filter inlet). However, at higher flows (>10 mgd), pH has a greater effect on the chlorine residual required to achieve 0.5-log inactivation.

Spring/Fall Conditions: ***Figure 3-3*** presents the required pre-chlorination residual at the filter inlet needed to maintain at least 0.5-log inactivation of *Giardia lamblia* over a range of flowrates and pH values for the "worst-case" Spring/Fall conditions (10 °C). As shown in the Figure, a chlorine residual below 0.3 mg/L is sufficient to achieve adequate CT over the entire range of pH and flowrate up to 20 mgd.

Summer Conditions: ***Figure 3-4*** presents similar results for the "worst-case" summer conditions (15 °C). As shown in the Figure, the relatively warmer water allows for greater operator flexibility with regard to plant flow and pH adjustment at flows up to 20 mgd (minimum chlorine residual = 0.13 mg/L).

In general, a portion of the plant's CT must be achieved through pre-chlorination; without a chlorine residual in the filter influent, the plant would be unable to achieve 0.5-log inactivation of *Giardia lamblia*. However, if "in-plant" DBP formation is to be minimized via reducing pre-chlorination, the plant has two options:

- Operate at lower flowrates by running the plant for longer periods, potentially increasing operational costs at the plant.
- If the plant is operated at higher flowrates to minimize operational costs, the use of pH adjustment should be delayed until after filtration and/or a higher chlorine residual could be maintained through the Clearwell.

The decision to make adjustments to the existing disinfection system will ultimately depend on the City's ability to meet the future D/DBP requirements and the desire to further reduce DBP concentrations in the distribution system. This issue is discussed in detail in the following section.

3.1.2 Disinfectants and Disinfection By-products

3.1.2.1 Regulatory History

The Federal Total Trihalomethane Rule (TTHM Rule) was published on the Federal Register in November 1979; Oregon adopted the MCLs established in this law in September 1982. The purpose of the rule was to limit exposure to chemical by-products of disinfection treatment present resulting from disinfection treatment practices. The TTHM Rule set an MCL for TTHM of 0.10 mg/L based on a running annual average of quarterly sampling of each source water in a given system. However, these MCLs were recently superceded when the State of Oregon adopted the Stage 1 Disinfectants/Disinfection By-products Rule (D/DBPR) on July 15, 2000. The D/DBPR added an MCL of 0.06 mg/L for haloacetic acids (HAA_5), and reduced the MCLs associated with TTHM to 0.80 mg/L in an effort to address the risk trade-offs with disinfection by-products control and the levels of pathogenic microorganisms and particulate matter (turbidity) in drinking water.

3.1.2.2 Monitoring Requirements

The Oregon Drinking Water Quality Act requires monitoring of disinfection by-products. For the Grants Pass WTP, current sampling number/frequency requirements for DBPs are the same as was required under the TTHM Rule. That is, four samples per quarter for each source water, with one sample representative of the maximum residence time in the distribution system and the remaining samples collected in the distribution system representative of the entire system (i.e. average residence time). Compliance is based on a running annual average of quarterly samples. To remain in compliance, the running average for TTHMs and HAA_5 must never exceed 0.08 mg/L and 0.060 mg/L, respectively.

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For both TTHM and HAA_5, monitoring frequency may be reduced if samples representing the longest system detention times contain less than 80 percent of the new MCL (0.068 mg/L and 0.048 mg/L, for TTHM and HAA_5, respectively). ***Table 3-3*** shows the compounds and corresponding MCLs under the amended rule.

TABLE 3-3: STAGE 1 D/DBP RULE MAXIMUM CONTAMINANT LEVELS

Contaminant	Maximum Contaminant Level (MCL) (mg/L)
Total Trihalomethanes[1] (TTHMs)	0.080
Haloacetic Acids[2] (HAAs)	0.060

[1]"Total Trihalomethanes" includes the sum of concentrations of chloroform, bromodichloromethane, dibromochloromethane, and bromoform.

[2]"Haloacetic acids" includes the sum of concentrations of: monochloroacetic, dichloroacetic, trichloroacetic, monobromoacetic, and dibromoacetic acids.

The Oregon Drinking Water Quality Act also regulates the Maximum Residual Disinfectant Levels (MRDLs) present in the distribution system. Since Grants Pass uses chlorine for disinfection, a maximum of 4.0 mg/L (as Cl_2) is allowed. Monitoring and compliance for the MRDLs of chlorine is similar to that required under the Total Coliform Rule (TCR). Utilities are required to collect these disinfection residual samples at the same location and frequency as coliform samples.

In addition to DBP MCLs and MRDLs, conventional WTPs that have surface water as a supply are required to remove specific amounts of organic material through their treatment process. The percent of removal required depends on source water TOC and alkalinity. ***Table 3-4*** provides a summary of the removal requirements.

TABLE 3-4: TOC REMOVAL REQUIREMENTS (PERCENT)

Raw Water TOC (mg/L)	Alkalinity		
	0 – 60	60 – 120	> 120
2.0 – 4.0	35	25	15
4.0 – 8.0	45	35	25
> 8.0	50	40	30

Compliance with this treatment requirement must be calculated as a running annual average (RAA) on a quarterly basis, after 12 months of data are available. Systems having raw water TOC concentrations < 2.0 mg/L may be exempted from any TOC removal requirements. Potential revisions to the TOC monitoring requirements presented in the Stage 1 Rule are proposed in the Stage 2 D/DBP Rule, as discussed in the Future Regulations portion of this report.

3.1.2.3 Historic Compliance

On average, the reported running quarterly annual averages for TTHM were 0.032 mg/L between January 1999 and June 2003. A maximum quarterly annual average of 0.069 mg/L was observed in May 2001, exceeding the 0.064 mg/L "cut-off" for reduced monitoring under the rule. The minimum, 0.009 mg/L, was recorded on February 2000. No instances of TTHM MCL exceedence are on record; quarterly average TTHM concentrations have consistently been lower than the allowable MCLs. Based on previous monitoring results, the City was eligible for reduced TTHM monitoring in the distribution system from September 2000 to November 2001.

Limited HAA_5 data is available for review, as HAA_5 was only recently adopted into the regulations. Though continuous quarterly sampling began in February 2002, running quarterly annual averages could not be calculated until November 2002 (when four quarters of data became available). However, the running quarterly annual averages for HAA_5 since November 2002 average 0.039 mg/L, well below the allowable MCL for HAA_5 of 0.060 mg/L.

Historical raw and finished water TOC sampling between February 2000 and December 2002 indicate that TOC levels in the Rogue River may occasionally exceed the "trigger" level of 2.0 mg/L during the winter months. However, between November 2001 and March 2002, when TOC levels exceeded 2.0 mg/L (requiring enhanced coagulation for a minimum of 35 percent TOC removal with alkalinities averaging 37.5 mg/L as $CaCO_3$), TOC removal efficiencies averaged 42 percent. The average raw water TOC throughout the sample period was 2.03 mg/L; removal efficiencies averaged 38 percent. ***Table 3-5***

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presents a summary of this data; actual TOC monitoring results, including quarterly samples taken in 2001 are presented in ***Figure 2-5***. The City should continue to monitor its raw and finished water TOC on a monthly basis to ensure continued TOC removal compliance through the plant. As previously mentioned, the City should also consider monitoring UV_{254} (a surrogate parameter for TOC) in the raw and finished water on a daily basis to better understand TOC removal through the WTP.

TABLE 3-5: SUMMARY OF HISTORICAL TOC SAMPLING RESULTS

Parameter	Raw Water TOC	Finished Water TOC	Removal Efficiency
	(mgd)	(mg/L)	(%)
Sample Dates	Feb 00 – Dec 02	Feb 00 – Dec 02	Feb 00 – Dec 02
Number of Samples	16	15	15
Average	2.14	1.31	37.86
Max	4.95	2.52	58.3
Min	1.22	0.71	25.7

To qualify for reduced monitoring of DBPs in the distribution system, Grants Pass must report concentrations of DBPs representative of the longest detention time in the system at 80 percent or less than the new MCLs (<0.064 mg/L and <0.048 mg/L for TTHM and HAA_5, respectively). Based on water quality test results between February 2000 and December 2003, 30 percent of TTHM samples taken from the Merlin Landfill (presumably, the end of the distribution system) exceed this lower limit; 33 percent of HAA samples from this same site exceeded this lower limit. Though the City may be eligible for reduced monitoring of DBPs in the future, it is recommended that DBPs continue to be monitored quarterly, if not monthly, to better quantify the impacts of adjustments in the disinfection strategy on the formation of DBPs throughout the year.

DBP Control: Though current DBP compliance is not an issue, the City may elect to further control DBP levels in the distribution system by minimizing the "in-plant" DBP formation via adjustments to the pre-chlorination system. To better understand the impacts of pre-chlorination on DBP levels measured in the distribution system, a

summary of relevant plant operational data during recent DBP sampling is presented in *Table 3-6*.

TABLE 3-6: AVERAGE PLANT OPERATIONAL DATA DURING RECENT DBP SAMPLING

Sample Date	DBP Average[1]		Plant Production[2]	Pre-Cl2 residual[2,3]	pH[2,3]	Temperature[2]
	TTHM (mg/L)	HAA5 (mg/L)	(mgd)	(mg/L)		(oC)
6/19/03	0.041	0.038	8.95	0.93	6.99	15.95
3/17/03	0.054	0.062	3.05	0.94	7.03	10.63
11/20/02	0.037	0.043	3.73	0.49	7.01	9.05

[1]Average of results from four monitoring sites, representing system averages on the date of sampling. Monitoring sites include: New Hope PS, Water Restoration Plant, Fire Station and Merlin Landfill.
[2]Represents average operating conditions for one week, up to and including the sampling date.
[3]Measured at the basin influent.
NOTE: Chlorine residual in the finished water was 1.0 – 1.1 mg/L for each of the sample periods analyzed.

As shown in *Table 3-6*, when system demands are high, as in the most recent DBP sample (June 2003), detention time in the distribution system is short, reducing the reaction time and minimizing DBP levels in the distribution system. These relatively low levels were observed despite the relatively high pre-chlorination residuals through the plant. When similar pre-chlorination residuals were observed during low flows (i.e. relatively long detention times in distribution system), as in the March 2003 sample, DBP levels were relatively high. However, when lower pre-chlorination residuals were maintained during low flows (November 2002), DBP concentrations were significantly lower. The following conclusions can be drawn from this analysis:

- At higher flows (e.g. relatively short distribution system detention times), and relatively high pre-chlorine residuals (~1.0 mg/L), resulting DBP levels in the distribution system are below current and future MCLs.
- At lower flows (e.g. relatively long distribution system detention times), pre-chlorine residuals appear to have a significant impact on overall DBP formation in the distribution system. Decreasing the pre-chlorine residual from 0.94 mg/L to 0.49 mg/L appears to reduce TTHM and HAA_5 concentrations by approximately 30%.
- Distribution system detention time (i.e. water age) should be minimized to help reduce DBP formation in the distribution system.

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It appears that the City may be able to "control" DBP levels in the distribution system by optimizing pre-chlorination levels at the plant, and minimizing water age in the distribution system. However, these efforts must be carefully balanced with plant disinfection performance to continue to reliably meet CT.

3.1.3 Lead and Copper

3.1.3.1 Regulatory History

On December 24, 1975, the National Primary Drinking Water Regulations (NPDWR) established the first lead MCL at 0.05 mg/L. This MCL was adopted into Oregon Law September 24, 1982. In 1991, the Lead and Copper Rule (LCR) was promulgated by the EPA to reduce lead and copper concentrations in drinking water. Oregon adopted the LCR on December 7, 1992, without exception. Lead and copper regulations, under the Oregon Drinking Water Quality Act, require utilities to implement optimal corrosion control treatment that minimizes the lead and copper concentrations at user's taps, while ensuring that the treatment efforts do not cause the water system to violate other existing water regulations.

3.1.3.2 Monitoring Requirements

Rather than establishing maximum contaminant levels (MCLs) for lead and copper, action levels for lead and copper were created. The action level for lead has been established at 0.015 mg/L, while the action level for copper is 1.3 mg/L. Utilities are required to conduct monitoring for lead and copper from taps in "high risk" homes. Two rounds of initial sampling were required during 1992-94, collected at 6-month intervals; annual sampling was required after these initial efforts. Following three years of annual sampling, samples are to be taken every three years. The action level for either compound is "exceeded" when, in a given monitoring period, more than 10-percent of the samples are greater than the action level.

Sampling requirements of the LCR are based on the population served by the utility. For Grants Pass (population between 10,001 and 100,000), Oregon law required 60 initial sampling sites; subsequent monitoring could be reduced to 30 sites provided initial sampling efforts demonstrate that lead and copper action levels are not exceeded. Water systems unable to meet action levels must either integrate corrosion control strategies into their treatment process train, or develop alternate source of water.

3.1.3.3 Historic Compliance

Initial lead and copper sampling began in Grants Pass in the fall of 1992. Since, lead and copper samples have been collected per Oregon Drinking Water Quality Act requirements. Action levels for lead and copper were not exceeded in any samples collected; monitoring requirements for the City have been reduced.

Through treatment process optimization at the City's WTP, lead and copper concentrations have remained low since the adoption of the LCR. Using lime for pH adjustment, a target pH of 7.2 for LCR compliance has been maintained. The most recent measurements, taken on July 19, 2002, report 90th percentile values of 0.0050 mg/L and 0.5270 mg/L, for lead and copper, respectively. These values are well below the current action levels for lead and copper.

3.1.4 Inorganic Contaminants

3.1.4.1 Regulatory History

All of the original MCLs established for inorganic contaminants (IOCs) in the NPDWR have been replaced by subsequent regulations. Excepting arsenic, the MCLs for all regulated IOCs under the Oregon Drinking Water Quality Act were adopted from the Safe Drinking Water Act (SDWA). MCLs for IOCs outlined in the Phases II (promulgated July 1, 1991) and Phase V (promulgated July 19, 1992) of the SDWA amended the Oregon Drinking Water Quality Act on June 6, 1992 and January 14, 1994, respectively.

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Impacts of the recently adopted arsenic MCL are also discussed in this section, though compliance with this new MCL is not required until January 2006. The rule reduces the arsenic MCL from 50 ug/L to 10 ug/L.

The intent of the Oregon Drinking Water Quality Act, with regard to IOCs, is to control the levels of minerals and metals in drinking water that create health concerns. For most IOCs, these health concerns result after long-term (lifetime) exposure to the compounds. However, the risks associated with nitrates are acute. Thus, additional monitoring requirements for nitrate/nitrite are included in Oregon law.

3.1.4.2 Monitoring Requirements

Monitoring requirements and MCLs for regulated IOCs are contained in ***Table 3-1***. All community water systems that rely on surface water systems for source water, must sample quarterly for nitrate/nitrite. For water systems that contain asbestos-cement (AC) water pipes samples testing for asbestos fibers must be taken every nine years. Monitoring for and compliance with the new arsenic MCL is required by January 2006. Concentrations of all other IOCs must be measured annually. Quarterly follow-up testing is required for any contaminants that are detected.

3.1.4.3 Historic Compliance

The Grants Pass WTP has remained in compliance with regard to all IOC MCLs during the period evaluated. Excepting nitrate, no there are no detection of IOC on record at the DHS. Nitrate/Nitrite concentrations in the treated water average 0.9 mg/L-N; a maximum of 1.52 mg/L-N was recorded on March 7, 2001.

Grants Pass has no record of installing of AC pipe; all historic concentrations of asbestos were below detection limits.

Arsenic has not been historically detected in the raw water at concentrations above the detection limit. Thus, the recent changes to the arsenic MCL should not impact the Grants Pass WTP.

3.1.5 Organic Contaminants

3.1.5.1 Regulatory History

All of the original MCLs established for organic contaminants, both volatile and synthetic, in the NPDWR have been replaced by subsequent regulations. MCLs for 53 different organic contaminants under the Oregon Drinking Water Quality Act were adopted from the Safe Drinking Water Act (SDWA).

Phase I Regulations of the SDWA, promulgated in June 8, 1987, established MCLs for eight volatile organic chemicals (VOCs); these MCLs were adopted into Oregon Law November 13, 1989. Phase II Regulations were promulgated in July 1, 1991 and established final standards for 10 VOCs and 18 synthetic organic chemicals (SOCs). Phase V Regulations were promulgated on July 7, 1992 and included MCLs for three VOCs and 15 SOCs.

3.1.5.2 Monitoring Requirements

Monitoring requirements and MCLs for SOCs and VOCs are contained in ***Table 3-1***. The City is required to sample VOC's annually and SOC's twice every 3 years. Quarterly follow-up testing is required for any contaminants that are detected.

3.1.5.3 Historic Compliance

No concentration of regulated VOCs or SOCs above the detection limit is on record between April 2000 and March 2003.

3.1.6 Radiologic Contaminants

3.1.6.1 Regulatory History

The original MCLs adopted from the NPDWR by Oregon on September 24, 1982 are still in effect in the Oregon Drinking Water Quality Act today. These rules were revised in October, 2002 to include a new MCL for Uranium, and to clarify and modify monitoring requirements. Together, these established MCLs seek to minimize the cancer risk associated with long-term exposure to six natural and man-made radiologic contaminants.

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3.1.6.2 Monitoring Requirements

Monitoring requirements and MCLs for Radiologic Contaminants are contained in ***Table 3-1***. Monitoring for radionuclides is required once every four years from surface water sources. If gross alpha is measured below 5 picocuries per liter (pCi/L), no radium analyses are required. Additionally, only systems with elevated risks (i.e. impacts by man-made radiation sources) must sample for beta/photon radiation.

3.1.6.3 Historic Compliance

The most recent radiologic samples were taken on November 9, 2000, no radiologic contaminants were present at concentrations above the detection level. Additional sampling for Radium/Uranium was performed on October 24, 2002, again, no radium or uranium was detected in the samples. Grants Pass has fully complied with all DHS radiologic standards.

3.1.7 Federally Monitored Unregulated Contaminants

3.1.7.1 Regulatory History

The Direct Final Unregulated Contaminant Monitoring Rule was published by the EPA in the March 12, 2002, *Federal Register*. The 1996 Amendments to the SDWA required EPA to promulgate revisions to the existing monitoring requirements for unregulated contaminants every 5 years. This Rule will not be adopted into Oregon's Drinking Water Quality Act as the rule will be enforced by the EPA.

3.1.7.2 Monitoring Requirements

The Unregulated Contaminant Monitoring Rule includes a new list of contaminants to be monitored, procedures for selecting a national representative sample of public water systems and procedures for incorporating the monitoring results into the National Contaminant Occurrence Database. The contaminants for monitoring are divided into three lists; see ***Table 3-7***. List 1 contaminants are to be monitored by all public water systems serving over 10,000 people and a smaller group of public water systems serving less than 10,000 people. List 2 contaminants are to be monitored by a representative group of 300 randomly chosen public water systems. List 3 is to be monitored at 200

"vulnerable" systems across the country. The EPA has not requested that Grants Pass monitor List 2 and List 3 contaminants.

For chemical contaminants, surface water systems shall monitor quarterly for one year and ground water systems shall monitor two times six months apart. For microbiological contaminants, systems shall monitor twice, six months apart. For all chemical constituents in Lists 1 and 2, monitoring shall be conducted at the entry point to the distribution system. For microbiological contaminants in List 1, monitoring would be conducted near the end of the distribution system and at a representative site within the distribution system. Sampling was to be conducted over a year-long period from 2001 to 2003. The Rule will be revised again in 2004.

TABLE 3-7: UNREGULATED CONTAMINANT MONITORING RULE MONITORING LIST

LIST 1 Assessment Monitoring of Contaminants with Available Methods	LIST 2 Screening Survey of Contaminants Projected to have Methods by Date of Program Implementation	LIST 3 Pre-Screen Testing of Contaminants Needing Research on Methods
(1) 2,4-dinitrotoluene	(13) Diuron	(29) Algae and toxins
(2) 2,6-dinitrotoluene	(14) Linuron	(30) Echoviruses
(3) DCPA mono acid	(15) Prometon	(31) Coxsackieviruses
(4) DCPA di acid	(16) 2,4,6-trichlorophenol	(32) Helicobacter pylori
(5) 4,4'-DDE	(17) 2,4-dichlorophenol	(33) Microsporidia
(6) EPTC	(18) 2,4-dinitrophenol	(34) Caliciviruses
(7) Molinate	(19) 2-methyl-1-phenol	(35) Adenoviruses
(8) MTBE	(20) Alachlor ESA	(36) Lead-210
(9) Nitrobenzene	(21) 1,2-diphenylhydrazine	(37) Polonium-210
(10) Terbacil	(22) Diazinon	
(11) Acetochlor	(23) Disulfoton	
(12) Perchlorate	(24) Fonofos	
	(25) Terbufos	
	(26) Aeromonas Hydrophila	
	(27) Polonium	
	(28) RDX	

3.1.7.3 Historic Compliance

The City was only required by the EPA to sample for List 1 contaminants. Unregulated contaminant monitoring has been performed quarterly since 2001; the City has remained

in compliance with Unregulated Contaminants monitoring requirements. None of the List 1 constituents were detected in the Grants Pass water system.

3.2 FUTURE DRINKING WATER QUALITY REGULATIONS

The 1996 Amendments to the Safe Drinking Water Act required some new rules and changed the schedule for rules already under development. A summary of pending rules, estimates of the timetables for promulgation, and projected effects on the City of Grants Pass are presented below. Future regulations discussed herein include:

- Long-Term Stage 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)
- Stage 2 Disinfection By-Product Rule (Stage 2 D/DBPR)

3.2.1 Enhanced Surface Water Treatment Rule

The purpose of the Enhanced Surface Water Treatment Rule (ESWTR) is to further improve the control of microbial pathogens in drinking water, especially *Cryptosporidium*. The ESWTR was split into 2 phases: Long Term 1 and Long Term 2. The final Long Term 1 ESWTR was published in November 2000. The Long Term 1 ESWTR only applies to public water systems serving less than 10,000 people and therefore does not effect Grants Pass. The Long Term 2 ESWTR was proposed in 2001, with the final proposed rule published in July 2003.

Compliance with the new rule will be tied to the availability of sufficient analytical capacity and the availability of software for transferring, storing and evaluating the results of all microbial analyses. The final agreement also requires EPA to develop support material and guidance manuals for the use of UV disinfection, a relatively new disinfection technology and listed as one of the "best available technologies" for *Cryptosporidium* inactivation in the rule. In addition, the final agreement indicates that systems will address the Stage 2-D/DBPR and the LT2ESWTR requirements concurrently to protect public health and optimize technology choice decisions. Thus, compliance with the new rule is expected between 2004 and 2011.

3.2.1.1 Anticipated Compliance Requirements

Many revisions to the LT2ESWTR have been made since the first publication. The most recent requirements that apply to the City of Grants Pass include:

1. Further increase filtration and disinfection performance criteria for all systems; disinfection criteria based on system (i.e. raw water) vulnerability to microbial contaminants. Incorporate raw water *Cryptosporidium* into sampling regimen.
2. Potential *Cryptosporidium* inactivation requirements.
3. Incorporation of a multi-barrier disinfection strategy.

To quantify system vulnerability, a 24-month intensive monitoring program for *Cryptosporidium* will be required to help classify plants into different source water concentration ranges (or "bins"); monitoring will need to begin in 2003-2004. For smaller systems, *E. coli* may serve as a possible indicator. To assist plants, a "Toolbox" of proven control measures for meeting treatment requirements will be available, including watershed control options, treatment options, filter performance, and challenge tests. ***Table 3-8*** presents the proposed treatment requirements for conventional plants based on results from the monitoring program.

TABLE 3-8: LT2ESWTR TREATMENT REQUIREMENTS FOR CONVENTIONAL PLANTS

Bin Number	Sample Results (# *Crypto* oocyst/L Raw Water)	Treatment Requirements
Bin #1	< 0.075	No Additional Treatment Required
Bin #2	0.0075 – <1.0	1-log Reduction
Bin #3	1.0 – 3.0	2-log reduction (1-log from disinfection)
Bin #4	> 3.0	2.5-log reduction (1-log from disinfection)

Non-disinfection related reduction can be achieved through one or more alternatives presented in the LT2ESWTR "Toolbox", below.

- Watershed control - 0.5 log.
- Alternative source/intake management - can get lower bin assignment.
- Off-stream storage - 0.5 log, 1.0 log based on hydraulic residence time.

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- Pre-sedimentation basin (w/ coagulation) - 0.5 log
- Lime softening - 0.5 log
- Lower finished water turbidity - 0.5 log for CFE of 0.15 NTU (95% of the time), or 1.0 log for individual filter effluent less than/equal to 0.15 NTU (95% of the time). Cannot get credit for both.
- Membranes - Challenge test.

Surface water systems serving >10,000 people will need to conduct 24-months of continuous monitoring, plus one additional month, to determine the source water concentration of *Cryptosporidium* for a given system. In addition, the rule requires that two samples be submitted during the first round of sampling: a field sample and a matrix "spike". The matrix spike is a one-time sample used to quantify the methods detection levels for a particular water quality; the effectiveness of the method will vary according to raw water alkalinity, pH, turbidity, etc. This sample is "spiked" with a known concentration of *Giardia*/*Cryptosporidium*, and the recovery levels measured (the assumption is that the "background" levels of *Giardia*/*Cryptosporidium* are the same between the field and matrix "spike"). Recently, the Grants Pass Laboratory was approved for *Cryptosporidium* monitoring under the new rule (EPA Method 1623).

In addition to raw water monitoring requirements, the LT2ESWTR will require all systems to perform disinfection profiling. Disinfection profiling was required for public water systems who measured TTHM or HAA_5 levels in excess of 80-percent of the new MCLs (≥0.064 mg/L and/or ≥0.048 mg/L for TTHM and HAA_5, respectively), during preliminary testing as part of the Interim ESWTR. The specific requirements for disinfection profiling were previously discussed in this report (Section 3.1.1). The City will need to work with DHS to establish an annual disinfection profile based on future modifications to the disinfection through the WTP to meet the new LT2ESWTR requirements, if any.

3.2.1.2 *Implications for the Grants Pass WTP*

The City began the 24-month *Cryptosporidium* monitoring program in the Rogue River in September 2003. Results from this sampling effort are summarized in ***Table 3-9***.

TABLE 3-9: LT2ESWTR BIN CLASSIFICATION FOR GRANTS PASS

Sample Date	*Giardia* cysts[1] (# cyst/L)	*Cryptosporidium* oocysts[1,2] (# oocyst/L)	LT2ESWTR Bin Classification[3]
9/16/03	3.2	0.10	Bin #2
10/27/03	0.4	<0.075[4]	Bin #1
11/12/03	0.8	<0.075[4]	Bin #1
12/9/03	0.5	0.20	Bin #2
1/13/04	0.3	<0.075[4]	Bin #1
2/10/04	0.1	0.10	Bin #2
3/9/04	0.6	<0.075[4]	Bin #1

[1]Includes empty cysts, cysts/oocysts with amorphous structure and cysts/oocysts with internal structure.
[2]Processed according to EPA Method 1623 for Detecting Giardia Cysts and Cryptosporidium oocysts.
[3]If the monthly results were equal to the 12-month RAA reported to the State.
[4]Detection limit for Method 1623

Based on the limited sampling data, it appears unlikely that the Rogue River contains *Cryptosporidium* oocysts at concentrations above the upper limit for Bin #2 classification (1.0 oocysts/L); the Grants Pass WTP will likely fall into either Bin #1 or Bin #2. Twenty-four months of sampling will need to be performed prior to Bin classification.

If the City is placed into Bin #2, treatment requirements under the new rule can be met via operational improvements at the plant. More rigid standards for individual filtered water turbidity (<0.10 NTU 95% of the time) will account for the required 1.0-log additional removal treatment requirement. Currently, individual filter effluent turbidities average 0.12 NTU, and range from 0.056 to 0.148 NTU (see ***Figure 2-15***). Filter improvements may be required to enhance filter performance in the future, if media loss continues over time. To better prepare for the LT2ESWTR, the installation of particle counters on the individual filter effluent lines is recommended to better understand the removal of particles/pathogenic organisms through the WTP, and to better predict turbidity breakthrough.

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Classification of Bin #3 or Bin #4 is highly unlikely based on the above results. However, if Grants Pass is classified in Bin #3 or Bin #4 and therefore required to inactivate for *Cryptosporidium*, installation of a disinfectant stronger than chlorine (e.g. ozone, chlorine dioxide, ultraviolet (UV) irradiation, etc.) may be necessary, as chlorine is a relatively ineffective disinfectant for *Cryptosporidium*. Alternatives for *Cryptosporidium* inactivation are discussed in **Section 6** of this report. Improvements to address future disinfection compliance are recommended as a "place holder" for planning purposes, until sufficient data can be collected to verify the need for such improvements.

3.2.2 Stage 2—Disinfection By-Products Rule

The purpose of the Stage 2 Disinfection By-product (D/DBP) Rule is to further reduce health risks associated with disinfection by-products. The draft was released in February 2001. A Final Stage 2 Rule was expected in the Fall 2003, but has now been delayed until 2004 at the earliest. Compliance with the new Rule is expected by May 2008.

3.2.2.1 Anticipated Compliance Requirements

For Grants Pass, compliance with the proposed Stage 2-D/DBP Rule is expected to occur in several Phases, as described below:

- **Monitoring**: Monitoring location requirements for DBPs will change to sites representing peak levels (i.e. maximum water age) within the distribution system, as identified in an Initial Distribution System Evaluation (IDSE); Grants Pass will need to work with DHS to complete an IDSE. A one year monitoring program including sampling every sixty days, including the peak historic month, will be required for surface water systems serving greater than 10,000. Compliance with these new monitoring locations is expected in 2004.
- **Phase I:** Meet locational running annual average (LRAA) for DBPs at each new sample point identified as part of the IDSE for TTHM and HAA_5 concentrations of 0.120 mg/L and 100 mg/L, respectively. Calculating LRAA entails averaging the quarterly annual results for each individual monitoring site, and reporting results from the monitoring site with the highest LRAA. Compliance with Phase I is expected in May 2005.

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- **Phase II:** Meet LRAA at each sampling point identified as part of the IDSE for TTHM and HAA5 concentrations of 0.080 mg/L and 0.060 mg/L, respectively. Compliance with Phase II is expected in May 2008 or in May 2010 with a 2-year extension.

3.2.2.2 Implications for the Grants Pass WTP

To help estimate the implications of the Stage 2 D/DBP Rule on the Grants Pass WTP, LRAAs were calculated from the historical data, when possible. ***Table 3-10*** presents the results of this analysis, as well as the quarterly annual averages currently reported to DHS.

TABLE 3-10: RECENT RESULTS FROM TTHM/HAA5 MONITORING, QAA AND LRAA RESULTS

Sampling Dates	Units	Quarterly Annual Average[1]	Locational Running Annual Average[2]
TTHM			
11/24/03	mg/L	0.041	0.056
9/8/03	mg/L	0.042	0.059
6/19/03	mg/L	0.040	0.060
3/17/03	mg/L	0.039	0.064
11/20/02	mg/L	0.032	0.055
HAA5			
11/24/03	mg/L	0.041	0.045
9/8/03	mg/L	0.042	0.046
6/19/03	mg/L	0.042	0.046
3/17/03	mg/L	0.042	0.048
11/20/02	mg/L	0.034	0.042

[1]As currently reported to DHS under the Stage 1 D/DBP Rule
[2]Based on Merlin Landfill data, as might be reported to DHS under the future Stage 2 D/DBP Rule; values need to be confirmed following monitoring results from new sites identified under the ISDE.

Based on the results in ***Table 3-10***, the Grants Pass WTP should be able to achieve future Phase I MCLs (0.120 mg/L and 0.100 mg/L for TTHM and HAA_5, respectively), as well as Phase II MCLs (0.080 mg/L and 0.060 mg/L for TTHM and HAA_5, respectively). However, several issues may impact these measured DBP concentrations in the future, including:

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- Additions to the distribution system at the "ends" of the system may increase the overall "age" of the water in the distribution system at the outer reaches.
- The current monitoring sites include one site, the Merlin Landfill, that probably represents the maximum DBP concentrations in the distribution system. LRAAs from the Merlin Landfill were presented in ***Table 3-10***. However, locations with higher levels of DBPs may be identified as new monitoring sites under the IDSE. Until the IDSE is completed and new monitoring sites are identified, the possibility of measuring higher DBP levels in the distribution system exists.

Impacts from these and other future changes affecting detention time in the distribution system should be closely monitored. Improvements to address future DBP regulatory compliance is recommended as a "place holder" for planning purposes, until sufficient data can be collected to verify the need for such improvements.

3.3 OTHER COMPLIANCE ISSUES

3.3.1 NPDES Discharge Permit

Plant solids from waste washwater, filter-to-waste and the sedimentation basins are collected/consolidated in one sludge lagoon (Medco Mill Pond) located across the street from the WTP. This sludge lagoon discharges decant/overflow into Skunk Creek and eventually into the Rogue River. An NPDES permit was issued for this discharge stream. Historic compliance with NPDES permit requirements has been maintained during the four-year period evaluated for this report. However, the lagoon is currently "at capacity" (i.e. full) and needs to be cleaned; potential short-circuiting through the lagoon is threatening the release of solids and/or chlorine into Skunk Creek, which would be in violation of the current NPDES permit.

Improvements to ensure continued compliance with the NPDES permit for both immediate dredging needs, as well as long-term solids handling improvements, are discussed in detail in **Section 6** of this report.

3.3.2 Intake and Screen

Recent environmental regulations have been promulgated to protect threatened and endangered species including several anadromous fish (salmon and steelhead) which populate the Rogue River. These new rules include specific requirements for river intakes and diversions to avoid the potential "take" of these species, especially juvenile fish. Important features of an acceptable intake system include maximum approach velocity, maximum screen opening size and a sweeping velocity to ensure that juvenile fish are not trapped in front of the intake.

When passing more than 9.2 mgd through the single intake opening at the Grants Pass WTP, the new criteria for approach velocity is exceeded. Since the plant rarely operates at instantaneous rates less than 10 mgd (2 pumps running), the approach velocity criteria is always exceeded. The existing travelling screen opening size is slightly exceeded and the sweeping velocity is not acceptable. A detailed analysis of the intake facilities for the WTP are summarized in a Technical Memorandum (TM) entitled *Review of Rogue River Intake and Pump Station* (MWH, 2003), and is included in ***Appendix D***. Alternatives for improving the intake to meet existing and future regulatory requirements presented in the TM are summarized in **Section 6** of this report.

3.4 SUMMARY AND RECOMMENDATIONS

In general, the Grants Pass WTP has consistently met all existing water quality regulations. One to three years of additional water quality monitoring will be required to determine the impacts from near-term, future requirements regarding disinfection efficiencies (CT), *Cryptosporidium* removal/inactivation and disinfection byproducts (DBPs). Areas to analyze further and perhaps make capital and/or operational and maintenance improvements include:

- Tracking TOC removal through the treatment plant,
- Further optimization of chlorine residuals and CT through the plant,
- Update Disinfection Profile based on disinfection adjustments (i.e. location of pH adjustment, increased chlorine residual through the Basins, etc.),

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- Continue raw water quality monitoring of *Cryptosporidium,* and
- Complete IDSE and increase DBP monitoring frequency in the distribution system. Coordinate DBP sampling with TOC sampling to better understand/quantify impacts of TOC on DBP formation.

A summary of additional water quality monitoring and treatment requirements resulting from existing and future regulations are listed in ***Table 3-11.***

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TABLE 3-11: ADDITIONAL WATER QUALITY MONITORING AND TREATMENT REQUIREMENTS

Regulation	Additional Requirements
Existing Regulations	
Oregon Drinking Water Quality Act (OHS 448 – Water Systems)	
Microbial Contaminants	❑ No additional requirements
Disinfectants and Disinfection By-products	❑ Incorporate daily UV_{254} monitoring as a surrogate for TOC to better quantify TOC removal through the plant. ❑ Begin monthly monitoring of raw water TOC, coordinate sampling with DBP sampling efforts. ❑ Increase DBP monitoring frequency to one month to better quantify impacts of pre-chlorination on DBP formation.
Lead and Copper	❑ No additional requirements
Inorganic Chemicals	❑ No additional requirements
Organic Chemicals	❑ No additional requirements.
Radiologic Contaminants	❑ No additional requirements
Unregulated Contaminants Monitoring Rule	❑ No additional requirements
Future Regulations	
Enhanced Surface Water Treatment Rule	❑ Raw water sampling regimen to determine raw water vulnerability to microbial contamination. ❑ Include *Cryptosporidium* sampling in raw water. Based on vulnerability, plant may need to meet more strict filtered effluent turbidities (<0.10 NTU 95% of the time). Particle counters may be required to better monitor particle removal through the plant. ❑ Perform Disinfection Profiling
Stage 2—Disinfection By-Products Rule	❑ Work with DHS to develop IDSE for future monitoring to better understand compliance issues associated with the Stage 2 D/DBP Rule. ❑ Changes in DBP sampling monitoring frequency, location and compliance reporting.

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Figure 3-1
Grants Pass WTP:
1999-2003 -- Historical Log Inactivation of *Giardia*

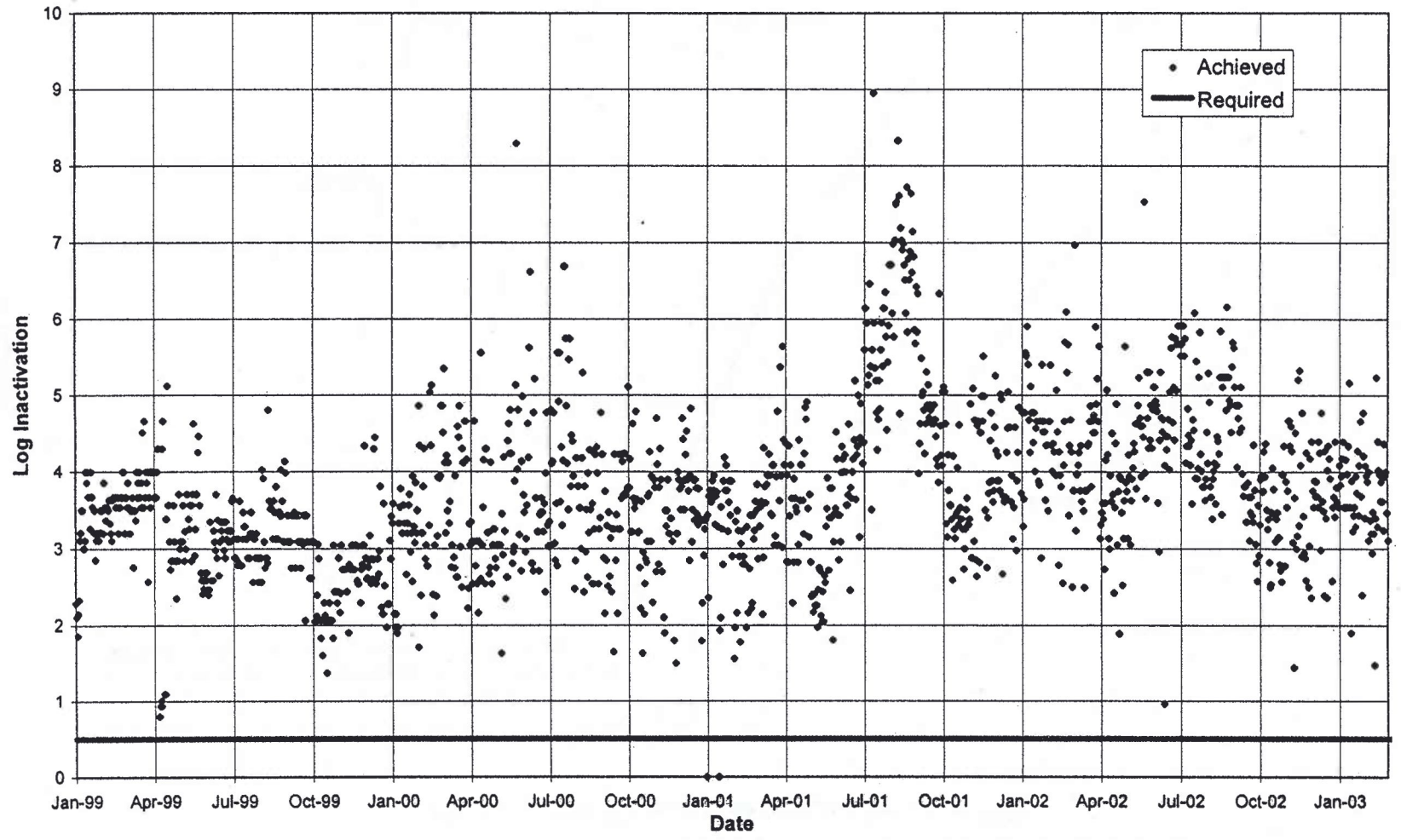

Figure 3-2
Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia
Worst-Case Winter Condition (Temp = 5°C)

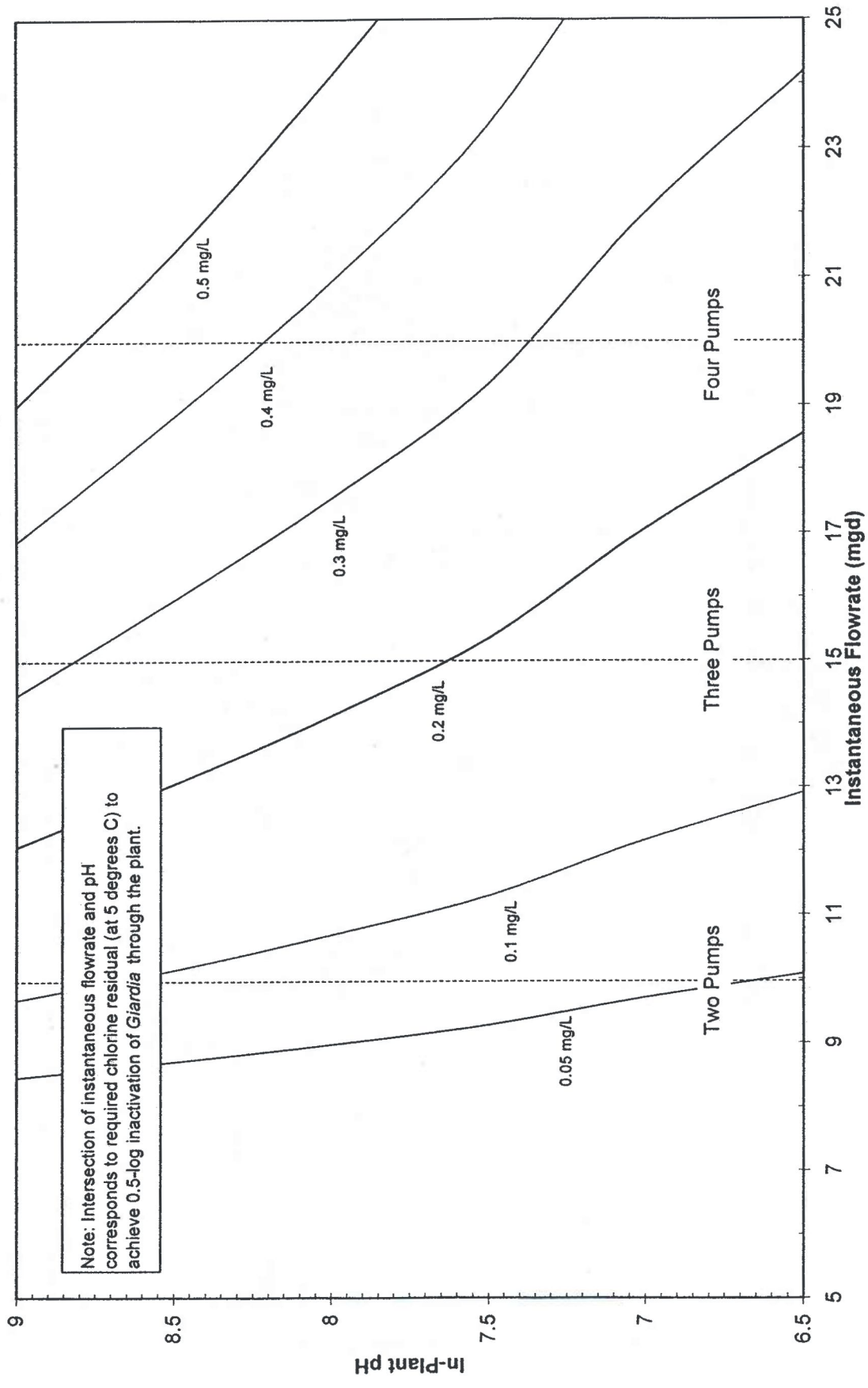

Figure 3-3
Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia
Worst-Case Spring/Fall Condition (Temp = 10°C)

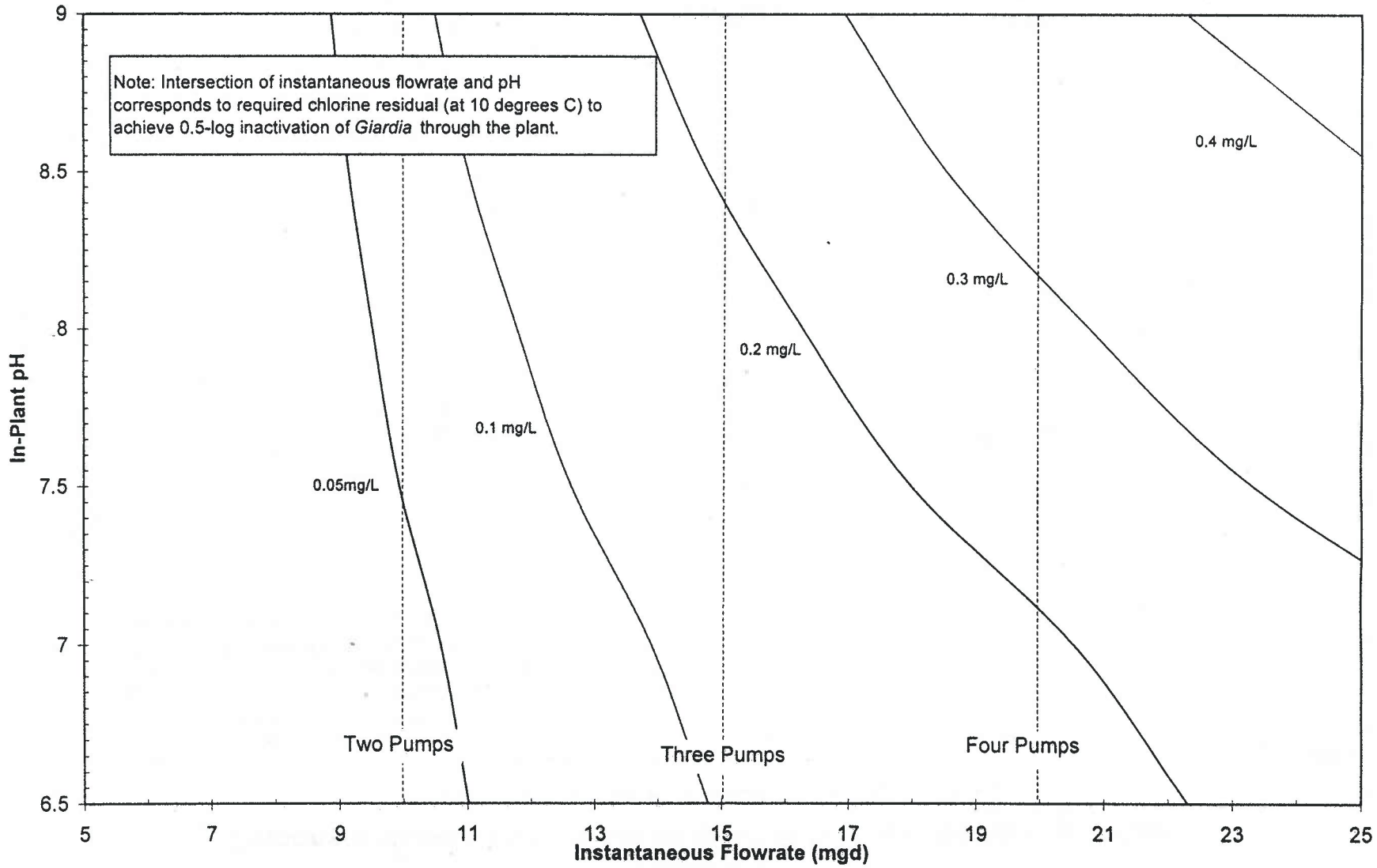

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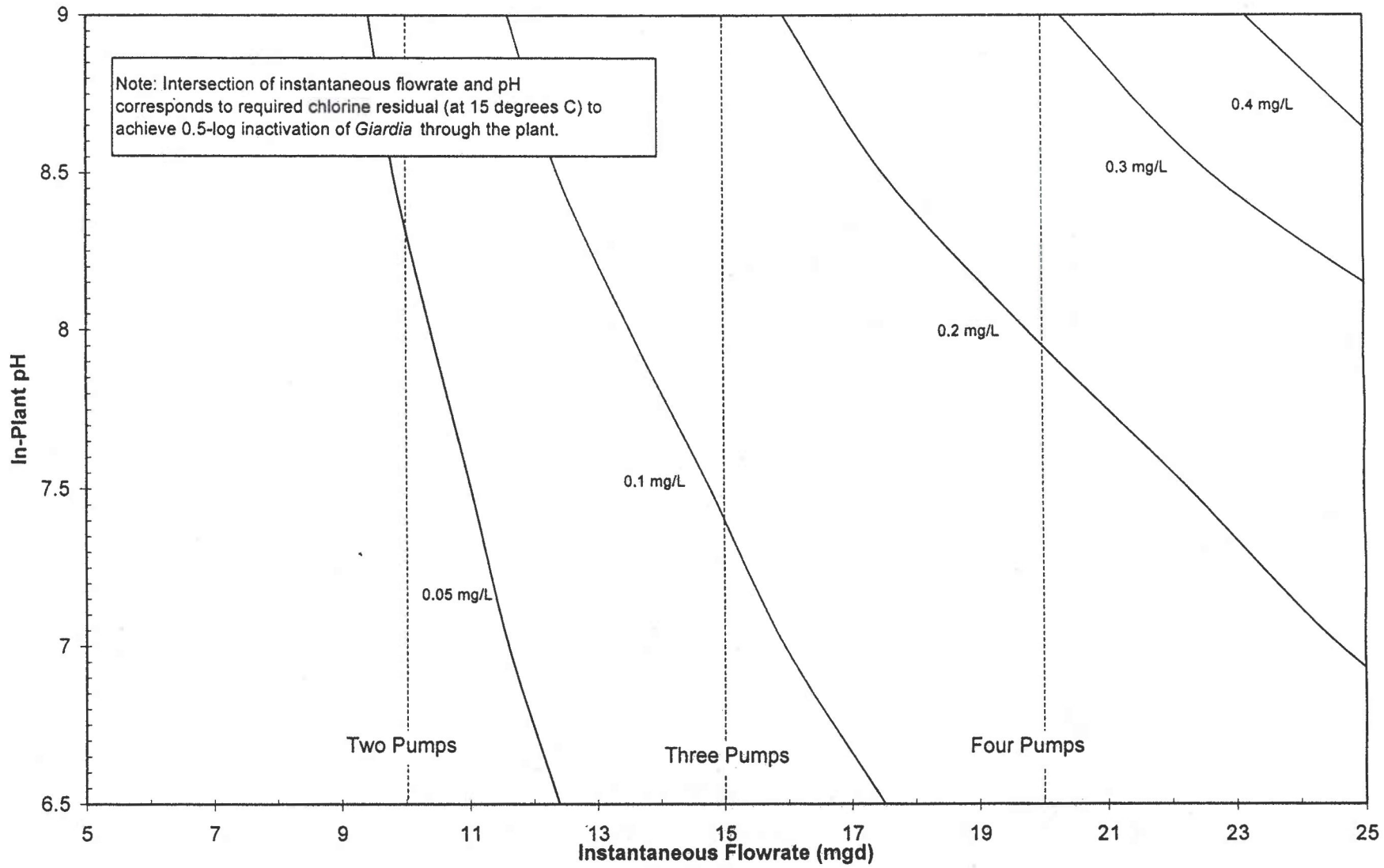
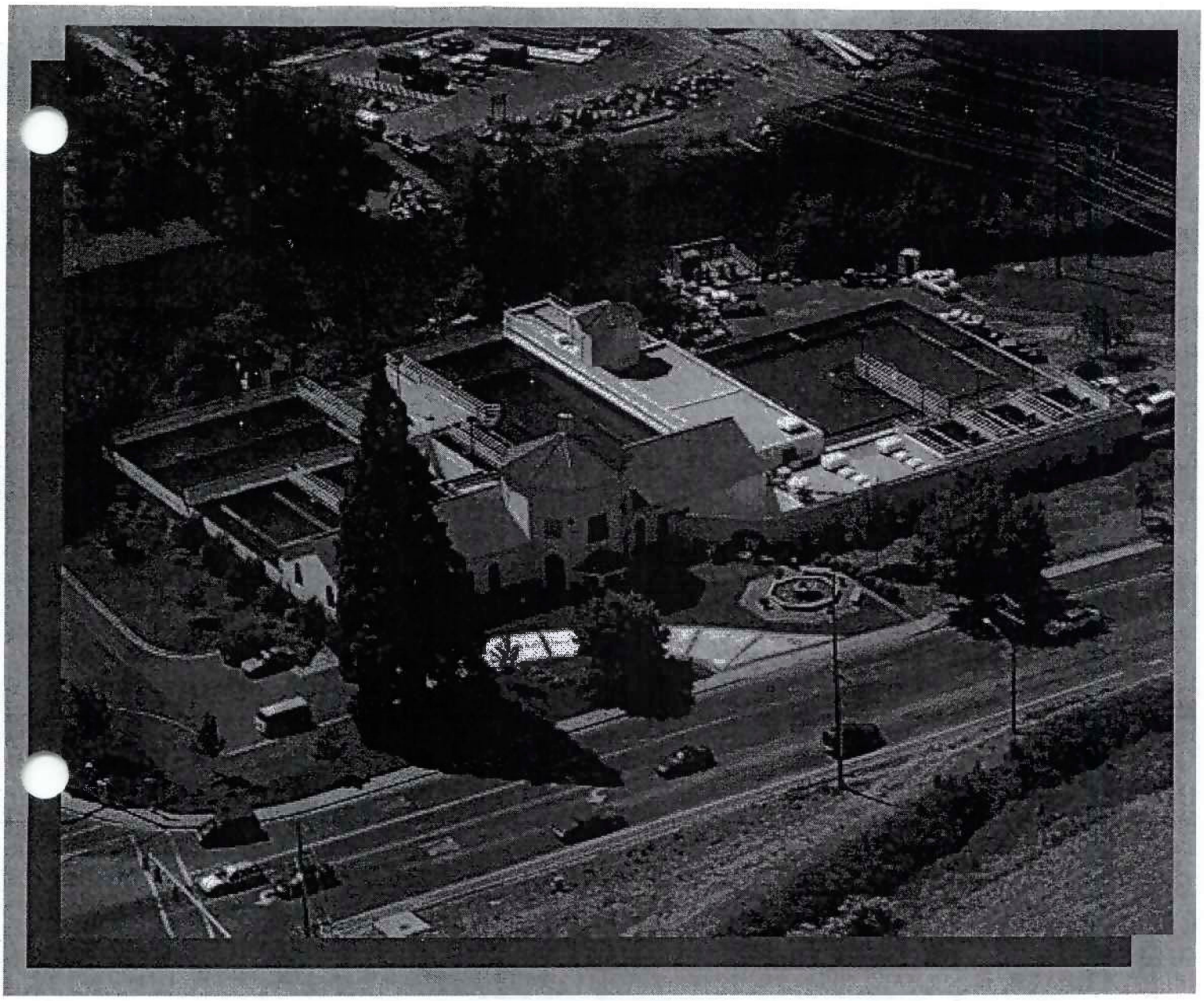
Figure 3-4
Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia
Worst-Case Summer Condition (Temp = 15°C)





Capacity Review

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4 CAPACITY REVIEW

A review of the capacity of the Grants Pass WTP was performed to determine the current capacity and possible future capacity given the constraints and limitations of each process and the interconnected system as a whole. Each process or support system will have its own *process* capacity relative to certain design or operating criteria/parameters which are independent of other unit processes. The *hydraulic* capacity is related to the piping, pumping, volume and flow control systems, which limit the ability of the water to flow through the interconnected system as whole. Each type of capacity is discussed and evaluated herein.

4.1 HYDRAULIC CAPACITY EVALUATION

The Grants Pass WTP can currently pump a maximum instantaneous flow rate of approximately 20 mgd, both into and from the WTP. The plant currently operates on a daily start/stop basis, as necessary, to fill storage reservoirs in the distribution system. Therefore, the operating schedule fluctuates with seasonal demands. During the winter months, the plant generally operates seven days per week, for an eight-hour period at instantaneous flowrates of either 10 or 15 mgd. Operational hours are extended during the high demand summer months, when the plant must operate in excess of twelve hours daily at flowrates of either 15 or 20 mgd in order to meet system demands.

Based on this information, the 20 mgd maximum capacity was used for shorter-term planning upgrades at the existing plant site. As demands continue to increase, the plant will have to operate for longer durations. As peak day demands approach 20 mgd, the existing plant will need to be expanded and/or an alternative site will need to be developed for adding more treatment capacity. Alternatively, aquifer storage and recovery (ASR), if technically feasible, could be implemented to meet peak demands in order to defer an expansion. Capacity expansion alternatives are discussed in detail in **Section 6** of this report.

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This section of the report evaluates the existing plant hydraulic capacity and analyzes its ability to possibly accept higher flow rates. Potential "bottlenecks" to hydraulic capacity expansion are identified and suggested improvements are mentioned if they appear to be feasible. The hydraulic capacity evaluation needs to be integrated with the process capacity evaluation to determine the range of feasible options for maintaining and possibly increasing the reliable plant production capacity.

4.1.1 Existing Hydraulic Profile

Figure 4.1 presents the hydraulic profile of the plant developed during the design of the 1983 expansion/upgrade for a maximum instantaneous flow of 20 mgd. The key hydraulic control features of the plant include:

- River water levels and intake pumping capacity
- Hydraulic capacity of the 36-inch raw water pipeline and chemical mixing vault
- Hydraulic capacity of the 24-inch pipeline delivering water to the Mixing Basin/Influent Channels to Basins #1 and #2, and the 30-inch and 24-inch pipeline capacity to Basins #3
- Hydraulic capacity of the overflow weirs in Basins #1, #2 and #3
- Hydraulic capacity of the pipe spools delivering water from the Basins to the filters
- Filter and pipe gallery hydraulics, including minimum water level inside the filters, for optimum performance and adequate available headloss for filter operations
- Filter underdrain and piping system capacity to the clearwell
- High service pumping capacity from the clearwell into the distribution system
- Finished water pipeline capacity to the distribution system
- Backwash piping and pumping capacity
- Washwater and Solids Handling

Hydraulic capacity issues associated with each of these features is described in detail below.

4.1.2 Intake and Raw Water Pumping Capacity

The existing intake was constructed in the 1983 when the plant was expanded and upgraded to replace an older intake located immediately upstream. The intake is equipped with 4 identical 75 hp vertical turbine pumps (Worthington 15HH-340); the design operating condition for each pump is 3,200 gpm at 65 feet total dynamic head (TDH), for a design total of 18.4 mgd. The actual TDH values are considerably lower than this design point, therefore the pumps have the ability to pump higher flows up to the current observed maximum of 20 mgd. Following the 2001-02 SCADA system improvements, raw water flows with all four pumps on-line are approximately 20 mgd.

The intake was constructed with space for two additional pumps and with two submerged openings to the river, but only one opening is equipped with a travelling screen. The other intake opening is currently equipped with a fixed screen and is normally sealed off from the river. Space is available to add another travelling screen for this opening if so desired.

The existing intake opening appears to be too small to meet the current minimum approach velocity requirements to protect juvenile salmonid fish species, when pumping at rates greater than 10 mgd. Detailed discussion of the hydraulic and regulatory limitations of the intake screen, and improvement options, is presented in ***Appendix D***.

There is currently no reliability/redundancy in the raw water pumps at flows of 20 mgd (i.e. with all pumps on-line). Therefore, according to current planning and operating conventions within the water industry, we would define the firm, reliable pumping capacity to be 15 mgd, assuming one pump is out of service. The plant will need to increase its pumping capacity to reliably deliver raw water to the WTP once demands exceed 15 mgd. As previously mentioned, there is room for two additional raw water pumps at the intake facility. Assuming two pumps of similar capacity (approximately 5 mgd each) are installed, the firm pumping capacity can be increased to 25 mgd, with a maximum pumping capacity of approximately 30 mgd, without significant modifications to the existing intake facility and electrical support system.

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Plant flow rates are currently defined by the number of raw water pumps on-line at any one time, and are therefore limited to increments of approximately 5 mgd. For increased flow control through the plant, installation of a variable frequency drives (VFD) on a minimum of one existing raw water pumps is recommended, two is preferred for reliability. Improved plant control will allow for greater operator flexibility and treatment optimization in the future.

4.1.3 Raw Water Pipeline/Channel Capacity to the Basins

The raw water pumps discharge into an underground 36-inch raw water pipeline which exits the intake facility to the north for approximately 5-feet, then bears east for approximately 20-feet, where water is introduced into a metering/chemical injection/static mix vault. Immediately following the vault, water flows through the 36-inch pipeline split between one 24-inch pipeline (delivering water to the slow-mixing basin and eventually to Basins #1 and #2), and one 30-inch pipeline. This 30-inch pipeline also splits into two 24-inch pipelines, one providing water to Basin #3, the second is currently blind flanged, and was included for plant expansion, presumably to carry approximately 10 mgd to a 4th basin. A small vault containing a flow control valve and meter (not currently in use) is located along the 24-inch pipeline prior to Basin #3. A combination of manually actuated valves is used to control the flow split between the three existing basins.

Pertinent design factors for the existing raw water pipelines are presented in ***Table 4-1***.

TABLE 4-1: RAW WATER PIPELINE VELOCITIES AND HEADLOSS

Plant Flow (mgd)	Velocity (fps)	$V^2/2g$ (ft)	Headloss (ft/100 ft)
36-inch			
15	3.28	0.17	0.08
20	4.38	0.30	0.14
25	5.47	0.46	0.22
30	6.57	0.67	0.29
30-inch			
10	3.15	0.15	0.09
15	4.73	0.35	0.20
20	6.30	0.62	0.36
25	7.88	0.96	0.57
24-inch			
6	2.95	0.14	0.10
8	3.94	0.24	0.17
10	4.92	0.38	0.29
12	5.91	0.54	0.41
15	7.39	0.85	0.60

As shown in the table, velocities through the 36-inch raw water pipeline exceed 6.0 fps at flows of approximately 27.5 mgd. Normally, for raw water pipelines exiting a pumping station, the maximum recommended velocity is 6.0 fps due to surge control concerns (water hammer) and pipe protection constraints. The specific piping network and operating conditions would need to be modeled to determine exact conditions and concerns. However, it is possible to tolerate higher flows given the relatively short segment of 36-inch pipe prior to the static mix, but a hydraulic modeling effort is required to confirm these scenarios and to determine pumping requirements. Also, the headloss associated with additional flow through the existing pipelines will ultimately raise the system TDH, reducing the capacity of the raw water pump station. To account for this, the City should consider slightly over-sizing any future raw water pumps to compensate for this increased headloss.

Velocities through the two 24-inch pipelines are a function of the flow split to each of the Basins. Basin #3 was designed to handle 8 mgd, or 40-percent of the plant flow at 20 mgd; the remaining 12 mgd is diverted to Basins #1 and #2. As shown in ***Table 4-1***, at approximately 12 mgd, the velocities through these pipelines approach 6 fps, the

maximum recommended velocity for a raw water pipeline. Therefore, the 24-inch pipeline leading to Basins #1 and #2 is currently "at capacity"; the 24-inch pipeline leading to Basin #3 may have an additional 4 mgd capacity before velocity criteria is exceeded. The 30-inch pipeline can tolerate flows of approximately 19 mgd before the 6 fps velocity criteria is exceeded.

As previously mentioned, flow splitting between the Basins is currently achieved by manually "throttling" a combination of valves along the raw water pipeline. Since adjustments to these valves are difficult to make, the valve settings were determined during maximum flow (i.e. 20 mgd) where they normally remain during all plant flow conditions. Therefore, the flow split is variable at flows less than 20 mgd. For increased operator control and flexibility, it is recommended that the existing Basin #3 flowmeter (currently out of service) be replaced with magnetic type flowmeter, less vulnerable to interference resulting from suspended solids (coagulated particles, sand) and more appropriate for "buried" application. Once a meter that operates properly is installed on this pipeline, manual valve adjustments to account for flow are more practical.

In general, in-line static mixers should be designed to provide between 1 to 3 seconds of mixing and a maximum headloss of 2 to 3 feet across the unit (imparting a mixing energy, or "GxT" = $3x10^4$ to $2x10^5$). The degree of mixing and the mixing time are directly related to the raw water flow rate through the static mixer. There is limited information on the type and design criteria for the existing static mixer on record; it is assumed that the static mixer was designed to provide optimal mixing between 5 and 20 mgd (the current range of plant flows). Record drawings indicate the existing static mixer is 36-inches in diameter and approximately 8-feet long. At 20 mgd (or 4.38 fps through a 36-inch pipe), the current mixing time is approximately 1.8 seconds. At 30 mgd (or 6.57 fps through a 36-inch pipe), mixing time will be reduced to approximately 1.2 seconds, slightly higher than the minimum recommended mixing time of 1 second. However, at these higher flowrates, headloss through the static mixer will increase significantly, raising the overall TDH and decreasing the capacity of the raw water pump station. Efforts to better understand the mixing energies imparted at various flow rates

should be taken prior to plant expansion above 20 mgd (the assumed design point for the existing static mixer).

The slow-mixing basin upstream of Basins #1 and #2 is currently not in use. However, two "flow through" baffle walls originally installed to ensure equal flow distribution between basins are still intact. These baffle walls create 3 to 5 inches of additional headloss through the basins that, if removed, may allow for a minor increase in hydraulic capacity to Basins #1 and #2. However, adjustments to the basin influent mud valves will need to be made to ensure proper flow split following baffle removal.

4.1.4 Basins and Filter Influent Channel

Section 2.4.2 discussed the design features of the three existing contact basins, each of slightly different size and shape. These basins were designed for a total combined hydraulic capacity of approximately 20 mgd. As discussed, these basins provide chlorine contact time for disinfection and efficient solids settling during most of the year. There are no provisions for continuous solids removal; the basins need to be manually cleaned periodically when the plant can afford to take a basin out of service. Settled water flows from all three contact basins via the launders into a filter influent channel located at the north end of the basins. This channel is continuous at the effluent of Basins #1 and #2; a 30-inch pipe connects the channel at the effluent of Basin #3 with that of Basin #1 and #2.

The normal water elevation in the basins is approximately 935.38 feet at 20 mgd with a triangular launder weir invert elevation of approximately 935.17 feet, according to field measurements taken during the plant survey (July 29th, 2003). The bottoms of the launder troughs are approximately elevation 932.67 feet.

The current water level in the basins is relatively high (i.e., little freeboard, particularly in Basins #1 and #2), leaving little room for additional flow in the Basins. The contact basins may be able to handle combined flows up to 30 mgd (approximately 12 to 15 mgd for Basins #1 and #2, and 15 to 18 mgd for Basin #3), at least hydraulically. This

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approach could eliminate the need to add any more pretreatment basins if they can be properly designed for good pretreatment performance. However, this would require significant improvement to the process and hydraulics within each basin. Alternatives for basin expansion and improvements are discussed in detail in **Section 6** of this report. However, a detailed hydraulic analysis is recommended in the future if/when the City is interested in "pushing" flows in excess of 20 mgd through the existing basins.

Water flows from all three basins via the launders into the filter influent channel at the north end of the basins. This channel is 2-feet wide by 5-feet high for Basins #2 and #3 (equivalent to a 36-inch diameter pipe), and transitions to a 1 ½-feet wide by 5-feet high channel in front of Basin #1 (i.e. south of Filters 1 through 3). As previously mentioned, the channel is continuous north of Basins #1 and #2; a 30-inch pipeline connects the channel from Basin #3 to that of Basins #1 and #2. The channel/pipeline adequately distributes water from the basins to the filters at flows up to 20 mgd. No hydraulic deficiencies were reported when one basin was taken off-line for cleaning. However, hydraulic limitations may exist in channel and/or the "hard-pipe" portion connecting the two portions of the filter influent channel at flows in excess of 20 mgd. The hydraulics associated with the filter influent channel/pipeline should be considered if/when the City performs a detailed hydraulic analysis to "push" flows in excess of 20 mgd through the basins.

Water from the filter influent channel is conveyed into the filters via a submerged pipe and gate valve (one for each filter). These valves are 16-inch for Filters 1 through 3, and 18-inch for Filters 4 through 8. These valves do not appear to create excessive headloss at flows of 20 mgd, based on water levels measured during the plant tour conducted on July 28th, 2003.

4.1.5 Filters and Filter Effluent Piping

Section 2.4.3 provides basic design information for the filters. Water typically enters the filter area via the troughs to achieve a normal filter operating level of 934.1 to 934.3 feet. A filter effluent modulating valve is used to maintain this water level; as headloss

increases, the valve opens. This water level provides 3.9 to 4.3 feet of submergence over the top of the filter media. The water flows down through the media, gravel and underdrains, and then out an effluent pipe into the filter gallery. The filtered water flows through the effluent pipe, an orifice plate flowmeter, a modulating butterfly valve and then into the filter effluent channel below the pipe gallery. Filter effluent pipeline diameters are 16-inch for Filters 1 through 3 and 6 through 8, and 18-inch for Filters 5 and 6; this pipe also delivers backwash water into the filter. The normal water level in the filter effluent channel is between 920.96 and 922.93 feet (per 1983 plant expansion drawings, CH2M Hill), which provides a total filter driving head of approximately 13 feet. Terminal headloss is currently set at 7.5 feet. As discussed in **Section 2.4.3**, the filters are currently operated at a relatively high filtration rate, particularly when one filter is out of service. Additional filters will need to be added to provide a plant capacity > 20 mgd future demands. Minor filtration rate and filter flow increases may be tolerated with deeper media.

The location of the existing filter effluent meters currently prohibits the metering of filter-to-waste, and may contribute to particulate "surge" when transitioning from FTW to production mode. Also, requirements for straight-pipe both upstream and downstream of the meter are not met, reducing the accuracy of the meter. During the plant tour on July 28th, 2003, the sum of the individual filter effluent meters was approximately 20% less than the flow determined by the raw water flowmeter. The filter effluent meters also rely on approximately 9 to 12 inches of headloss across the orifice plate to measure flow. If this head was available for filtration, filter run lengths could be increased approximately 10 to 15 percent longer than those currently achieved at the plant. Therefore, it is recommended that the meters be eventually relocated and replaced with meters that don't induce headloss (magnetic or ultrasonic meters, for example).

4.1.6 Clearwell

The clearwell at the WTP is comprised of three interconnected clearwells, one located under each group of filters and built at different times. A common filtered water channel currently routes all filtered water to the east clearwell (located beneath Filters 1 through

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3), where it is chlorinated, then directed through a series of serpentine baffles through the center and west clearwells and finally to the finished water pump area in the west clearwell. The clearwell provides finished water storage, disinfection contact time and stored water for filter backwashing in addition to serving as the wetwell for the high service and backwash pumps. The clearwell overflow weir is in the north west corner of the west clearwell, beneath Filter 6 through 8. The overflow water is discharged into a square concrete structure located north of the new filter pipe gallery (near Filter 8), before it flows via a 36-inch pipeline connected to the 36-inch plant drain (which leads to the washwater and solids equalization basin). This drain pipe should be sufficient to handle clearwell overflows up to 20 mgd; improvements to the clearwell overflow and drain pipe may be required if/when the capacity at the plant is expanded.

The total volume of the clearwell is estimated at 433,000 at the overflow weir (*Water Filtration Plant O&M Manual*, CH2MHill, 1983). Based on limited construction drawings, the minimum floor elevation is 907.54 feet, but drops to an elevation of approximately 906.0 feet in the pumping area to allow greater use of the entire clearwell volume. The elevation of the overflow weir is 923.04 feet. According to plant staff, the current minimum operating water elevation is 920.5 feet to ensure adequate detention time for disinfection and to provide minimum pump bowl submergence.

During normal operating conditions, the high service pumps operate to maintain a relatively constant clearwell level (approximately 922.9 feet); two of the high service pumps are equipped with VFDs to account for this flow variability. When a filter is backwashing, the high service pumping rate can be reduced to maintain the clearwell level above 920.0 feet. Typically, a maximum of 70,000 gallons of water is used for backwashing the largest filters (Filters 5 and 6), representing approximately 1.5 feet decrease in clearwell level.

The nominal clearwell volume at maximum operating level (921.14 feet) is 362,000 gallons and the average detention time is 21.8 minutes at 20 mgd. The clearwell has limited storage for consecutive filter backwashes. Flows from the High Service Pump

Station must be adjusted to minimize clearwell drawdown during periods of consecutive filter backwashes to ensure reliable high service pumping and to meet CT requirements.

MWH typically recommends a minimum clearwell volume of 60 minutes of detention time at peak flow rate, assuming the distribution system has an abundance of storage. At the existing 20 mgd peak rate, this 1-hour criteria would result in a minimum clearwell volume of 830,000 gallons; the existing 433,000 gallon clearwell represents 52 percent of this recommended minimum volume. ,If the plant's capacity is expanded to greater than 20 mgd, it is recommended that the clearwell capacity also be increased to meet CT . At 30 mgd using the 1-hour criteria, the suggested minimum clearwell volume is 1,250,000 gallons. Detailed discussion regarding clearwell improvements for future expansion are presented in **Section 6**.

4.1.7 High Service Pump Station

The WTP is equipped with 5 vertical turbine, high service pumps including:

- Two large pumps, each rated at 4,000 gpm (5.8 mgd) at 210-feet TDH, with 300 Hp motors
- Two medium pumps, each rated at 3,500 gpm (5.0 mgd) at 210-feet TDH, with 250 Hp motors (one with VFD installed in 2003)
- One small pump, rated at 2,600 gpm (3.7 mgd) at 210-feet TDH, with 250 Hp VFD motor installed in 2002

The four larger pumps were installed as part of the 1983 plant expansion project. The original pump station layout provided for seven high service pumps total (with space for one backwash pump). So, there is room for two additional high service and/or backwash pumps. The pumps can be turned on and off from the SCADA system, based on the distribution system demands and storage conditions. Operators use the VFDs to control plant output to maintain relatively "constant" clearwell level. The existing high service pump conditions are as follows:

- Static water pressure is approximately 70 to 80 psi
- Operating pressures range from 70 psi to 102 psi, according to plant staff.
- Actual pump TDH ranges from 160 to 240 feet, including the lift out of the clearwell.

According to current planning and operating conventions within the water industry, we would define the firm, reliable pumping capacity to be approximately 16.7 mgd assuming one of the largest installed pumps is out of service. The plant will have to increase its pumping capacity to reliably deliver treated water from the WTP to the distribution system as peak day demands approach 16 mgd. There should be at least one more pump added to increase the reliable pumping capacity to approximately 20 mgd. At least two pumps should have VFDs for increased reliability. Increased flow control will allow for greater operator flexibility and disinfection optimization in the future.

Assuming both available pump spaces are filled with new high service pumps, all of the plant's total pumping capacity can probably remain located in the existing High Service Pump Room at plant flows up to 30 mgd. There are a number of options for increasing the pumping capacity to meet these future demands; alternatives for pumping expansion are discussed further in **Section 6**.

4.1.8 Finished Water Pipeline

The high service pumps discharge into a 36-inch finished water pipeline which exits the building to the north, then bears north north-east before connecting to the distribution system south of "M" Street. An 18-inch connection links this transmission pipeline to the on-site surge tank, buried underground, north of Filter 6. The 36-inch pipeline splits to two 30-inch pipes, one continues north north-east, then bears east along "M" Street, the second bears west, then south, crossing the Rogue River. Pertinent design factors for the existing 36-inch pipe are presented in ***Table 4-2***.

TABLE 4-2: FINISHED WATER PIPELINE VELOCITIES AND HEADLOSS

Plant Flow (mgd)	Velocity (fps)	$V^2/2g$ (ft)	Headloss (ft/100 ft)
36-inch			
15	3.28	0.17	0.08
20	4.38	0.30	0.14
25	5.47	0.46	0.22
30	6.57	0.67	0.29

Normally, for finished water pipelines exiting a pumping station, the maximum recommended velocity is 6.0 fps due to surge control concerns. However, the existing 11,300 gallon surge tank significantly reduces risks associated with system surge (water hammer). Therefore, the existing finished water pipeline should be sufficient to transmit flows of 30 mgd. However, at these higher flows, the surge tank will likely need to be replaced with a larger tank to provide adequate protection. A detailed hydraulic analysis of the down-stream distribution system is recommended before the City considers "pushing" flows in excess of 20 mgd out of the plant. Depending on the location of future system demands, distribution system improvements may be required for the system to receive flows in excess of 20 mgd

4.1.9 Backwash Piping and Pumping

The WTP is currently equipped with one vertical turbine backwash pump with a 200 Hp motor, rated at 7,000 gpm with 62-feet TDH. A VFD was installed on the backwash pump in 1999. Emergency backup to the backwash pump is provided via a connection with the high service pump station discharge pipeline. However, adequate pressure reducing and flow control valves were not installed, raising concerns about potential excessive pressures in the backwash header. Therefore, there is currently no reliable backup for the backwash pump. A replacement motor is available in the event the backwash pump motor fails; replacement time is estimated at approximately 7 hours. At current system demands/operating durations, the plant can rely on replacement of the backwash motor as a feasible "backup" strategy. However, as system demands (and corresponding operational durations) increase, installed backup capacity for the backwash system is recommended.

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Options for backup backwash capacity include having an entire replacement pump ready for installation, the installation of a new backwash pump, or improvements to the existing backup system (i.e. connection to the existing high service pump station discharge header). Since the installation of additional finished water pumps will be required to reliably deliver flows in excess of 16.7 mgd, preserving the two additional spaces for future high service pumps is advised. Therefore, the purchase of a complete new pump, ready for installation and/or improvements to the existing backup system (including installation of appropriate flow control and pressure reducing valves) is recommended.

The backwash pump discharges into a 16-inch diameter header that feeds backwash water to the individual filters. This pipeline could conceivably accept flows up to 7,500 gpm and still meet velocity/headloss design criteria. However, there is currently inadequate surge protection along the pipeline. One such surge event caused by the premature closure of a backwash valve disrupted the "push-on" joints along this pipeline, resulting in continuous leaking from the pipeline located in the Filter 4 and 5 pipe gallery. As a result, backwash pumping capacity is currently limited by the operators to less than 7,000 gpm to prevent further damage; 7,000 gpm is required to clean Filters 5 and 6.

Waste washwater discharges through a backwash drain pipeline (14-inch for Filters 1 through 3, 18-inch for Filters 5 and 6, and 18-inch for Filters 6 through 8) which eventually connects to a 36-inch drain line leading to the washwater and solids equalization basin. It is reported that Filters 1 through 3 currently experience "choking" in the washwater channels/piping at flows in excess of 4,500 gpm. Improvements to these facilities will be necessary if backwash rates in excess of 4,500 gpm are required (based on installed media specifications).

4.1.10 Solids and Washwater Handling

The Washwater and Solids Equalization Basin was designed to receive large flows of waste washwater, filter-to-waste water and Basin cleaning/drain water. The total basin volume to the overflow weir elevation is approximately 116,000 gallons; water is diverted to the raw water intake in the event of an overflow. This basin was originally

sized to allow for two consecutive backwashes of the large filters, assuming 10 minutes are required for each backwash. However, the current backwash regimen (i.e. 15 minutes of backwash) produces more washwater than was designed for, thereby requiring a faster pumping rate to the lagoon and limiting operator flexibility. Improvements to minimize the amount of washwater created during backwash were discussed in **Section 2**.

Two transfer pumps were installed in the Washwater and Solids Equalization Basin as part of the 1983 plant expansion that deliver water/solids to the sludge pond/lagoon. Both older pumps have 30 Hp motors, each rated at 1,500 gpm at 36-feet TDH. At this pumping rate with one pump on, it takes approximately 46 minutes to deliver one large filter backwash volume to the lagoon. The pumps operate automatically from level controls that turn the pumps on/off; the pumps are operated in a "lead/lag" configuration; the "lead" pump turns on and off according to basin water level during normal operation, the "lag" pump will turn on if a "high" water level is reached. A third pump was installed in 2000. This pump has a 60 Hp motor, and is rated at 1,750 gpm at 60-feet TDH. This pump was intended to eventually replace one of the original pumps.

The transfer pipeline is 8-inches in diameter. At current single-pump flows of 1,500 gpm, velocities in this pipeline approach 9.8 fps. With both pumps on line, velocities in this pipeline approaches 12 fps. The City may consider improvements to this pipeline to reduce the velocities in the pipeline in order to increase the pumping rate.

4.1.11 Summary of Hydraulic Capacity Evaluation

- The plant appears capable of handling approximately 30 mgd "into and out of"
- Improvements to the intake are required to meet the current minimum approach velocity requirements to protect juvenile salmonid species at pump rates in excess of 10 mgd. Consider making the improvements suitable for 30 mgd.
- Install 5.0 mgd additional raw water pumping capacity to increase the reliable (firm) pumping capacity to 20 mgd, with a maximum pumping capacity of 25 mgd at the time when plant demands reach 15 mgd. The intake can be equipped with 2 more pumps to provide approximately 30 mgd total pumping capacity.

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- To increase operator control and optimize plant performance, installation of a VFD on at least one existing raw water pump is recommended; installation of VFDs on two pumps is preferred for equipment reliability.
- For increased operator control and flexibility, it is recommended that the existing Basin #3 flowmeter (currently out of service) be replaced with magnetic type flowmeter, less vulnerable to interference resulting from suspended solids (coagulated particles, sand) and more appropriate for "buried" application. Once a meter that operates properly is installed on this pipeline, manual valve adjustments to account for flow are more predictable.
- The City should consider removing the existing "flow through" baffle walls originally installed as part of the slow mixing basin (not currently in use) to recuperate the headloss through the slow mix basin if saving headloss is important to increase capacity through Basins #1 and #2.
- Filter flow meters should be relocated to measure filter-to-waste flows. The City should consider installation of new meters which require minimal upstream and downstream "straight pipe" for increased meter accuracy and decreased headloss through the meter.
- The clearwell is currently undersized. CT has been met through the plant by carefully monitoring and maintaining chlorine residual through the basins, limiting operator flexibility. The clearwell volume could be increased to add operational flexibility.
- The current reliable (firm) capacity of the High Service Pump Station is 16.7 mgd. The plant will need to install additional pump(s) to increase the firm capacity when plant demands reach 15 mgd (same time when an additional raw water pump is added).
- The City should consider improvements to provide reliability to the backwash pump in case the existing pump fails. Options for correcting this deficiency include installation of a new back-up backwash pump, improving the design and control of the inter-connect with the high service header to ensure that overpressurizing the underdrains does not occur, or purchasing a new spare pump and motor (un-installed).

- Replacement of portions of the backwash discharge header through the Filter 4 and 6 pipe gallery are necessary to eliminate leaking and remove operator-imposed limitations on capacity and pressure in the backwash header.
- Hydraulic improvements to the waste washwater piping for Filters 1 through 3 will need to be considered if required backwash flows exceed 4,500 gpm (based on installed media design).

4.2 PROCESS CAPACITY EVALUATION

Each of the key plant processes was evaluated for its ability to meet current and possible future conditions, based on past proven performance and also on MWH's experience and opinions based on design of new plants and plant expansions observations made at other operating plants.

4.2.1 Chemical Feed systems

The primary chemical storage, metering and feed systems at the plant include:

- Liquid alum (50%) for primary coagulation
- Liquid sodium hypochlorite (12.5%) for disinfection (pre- and post-chlorination)
- Hydrated lime for pH adjustment
- Dry polymer for filter aid
- Dry potassium permanganate ($KMnO_4$) for taste and odor control

All five systems are typically used continuously whenever the plant is in operation; lime addition may not be needed during parts of the year. The doses of each chemical vary depending on plant flow and raw water quality.

4.2.1.1 Alum

Alum is stored in two 6,000 gallon fiberglass tanks (12,000 gallons total) inside the chemical storage room. The plant currently adds alum to the raw water for primary coagulation prior to static-mix. The chemical metering system consists of two positive displacement diaphragm pumps, both rated at 24 gph (at 125 psi). The alum feed is continuous using carrier water; carrier water flow rates are estimated at 15 gpm. On

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average, alum is diluted approximately 40:1 with carrier water, resulting in an alum concentration of approximately 1.25% in the chemical injection stream.

Table 4-3 presents pertinent alum pumping rates and storage capacities for the existing system.

TABLE 4-3: ALUM PUMPING AND STORAGE CAPACITY AT VARIOUS FLOWS

Peak Day Demand (mgd)	Range of Dosages (mg/L)	Pumping Rate[1] (gph)	Storage Capacity[2] (days)
10	15 - 50	9.7	33.3
15	15 - 50	14.5	22.2
20	15 - 50	19.3	16.6
25	15 - 50	24.1	13.3
30	15 - 50	29.0	11.1

[1]Based on minimum alum dosage at PDD
[2]Based on maximum alum dosage at ADD (calculated as PDD/2.14)

At the current maximum instantaneous plant flow of 20 mgd, the estimated maximum alum usage rate is 2,500 pounds per day (ppd) at an alum dose of 15 mg/L. This equates to a maximum chemical pumping rate of 19.3 gallons per hour (gph) using 5.4 pounds of alum per gallon of solution. 19 gph is below the current rated pumping capacity of the alum feed pumps. Assuming a dose of 15 mg/L, the existing pumping system should be capable of reliably meeting plant demands up to 25 mgd if maximum alum doses remain similar. Replacement of existing metering pumps with larger capacity pumps will be required to achieve reliable alum feed capacity at flows in excess of 25 mgd. The City may be able to avoid pump replacement if alum doses can be reduced via chemical optimization. However, by the time the City is ready to expand to 30 mgd, the existing pumps will likely have reached the end of their useful life, and will require replacement.

MWH typically recommends 15 to 30 days of chemical storage (depending on location, access to deliveries, potential winter delivery outages, etc.), calculated at a maximum dosage and average day demand. Alum storage requirements for the plant's existing flow conditions (i.e. 4.9 mgd ADD and a maximum alum dose of 50 mg/L) are approximately 5,800 gallons for 15 days or 10,600 gallons for 30 days. Thus, storage capacities at the

plant are sufficient for the near-term, as alum is readily available for delivery. The City may consider incorporating additional alum storage as peak day demands increase beyond 20 mgd. However, optimization of the current coagulation scheme may considerably decrease alum dosage in the future. Depending on the availability of alum, the existing alum storage tanks might be able to provide adequate storage up to 30 mgd.

Additionally, the alum carrier water flow rate is probably too high, resulting in over-dilution of the alum prior to injection into the process stream. Alum can be diluted up to 5-percent solution without serious impacts on the "reactivity" of the alum. However, at concentrations below 5-percent, the alum can potentially start to coagulate within the chemical feed lines, clogging the chemical feed line and/or elevating alum demands and increasing solids production. A flow control device should be installed on the alum carrier water line to ensure feed concentrations remain above 5-percent under all dosage and plant flow conditions.

4.2.1.2 Sodium Hypochlorite

Liquid sodium hypochlorite (12.5% solution = 1 pound of chlorine per gallon of solution) is delivered and stored in three fiberglass reinforced plastic tanks, each with a capacity of 2,120 gallons (total storage capacity = 6,360 gallons), located inside the hypochlorite feed room (adjacent to the chemical feed room). The storage tanks and metering pumps are located within a concrete containment area to contain a major leak. There are three positive displacement mechanical diaphragm-metering pumps, each rated at 17.0 gph. Under normal operating conditions, one pump is dedicated for pre-disinfection (with injection into the static mixing vault), the second for post-disinfection (with injection into the clearwell), and the third pump serves as backup. Space and a piping connection has been included for future pump addition.

Table 4-4 presents pertinent hypochlorite pumping rates and storage capacities for the existing system.

TABLE 4-4: HYPOCHLORITE PUMPING AND STORAGE CAPACITY AT VARIOUS FLOWS

Peak Day Demand (mgd)	Dosage (mg/L)	Pumping Rate[1] (gph)	Storage Capacity[2] (days)
10	0.8 - 3	10.4	54.4
15	0.8 - 3	15.6	36.3
20	0.8 - 3	20.9	27.2
25	0.8 - 3	26.1	21.8
30	0.8 - 3	31.3	18.1

[1]Based on maximum hypochlorite dosage at PDD
[2]Based on maximum hypochlorite dosage at ADD (calculated as PDD/2.14)

At the current maximum instantaneous plant flow of 20 mgd, the estimated hypochlorite usage is 500 ppd at a combined (i.e. pre- and post-chlorination) dose of 3.0 mg/L (per the plant O&M Manual). *Please note: maximum dosage was used in this calculation as it more conservatively estimates hypochlorite usage during "peak" season (i.e. summer) demands.* This equates to a total chemical pumping rate of 20.9 gph total, or 10.5 gph per on-line pump, well below 17.0 gph, the current rated pumping capacity of each of the feed pumps. Assuming a dose of 3.0 mg/L, the existing pumping system should be capable of reliably meeting plant demands up to 30 mgd.

Hypochlorite storage requirements for the plant's existing flow conditions (i.e. 4.9 mgd ADD and a maximum hypochlorite dose of 3.0 mg/L) are approximately 1,800 gallons at 15 days and 3,600 gallons at 30 days. During periods of low demands, the City should consider dilution of hypochlorite to a concentration of 10-percent (or less, depending on demands) to reduce degradation of the chemical associated with longer holding times . Existing on-site storage capacity is sufficient for peak demand flows in excess of 30 mgd, while still providing more than 15 days of storage. Thus, no additional hypochlorite storage will be required in the foreseeable future.

4.2.1.3 Lime

Hydrated lime is shipped in bulk and stored in a lime bin/hopper with a total storage capacity of 1,900 cf, or approximately 30 tons. The lime feed system consists of a 6-foot diameter Vibra Screw bin activator, a BIF volumetric feeder, a 50 gallon solution tank and two constant speed slurry pumps rated at 40 gpm at 16 feet TDH. Lime solution is

mixed with carrier water and directed to one of several application points. During the July 23rd plant visit, all lime required for pH adjustment was being fed into Basin #2, near the settled water launders. This has been the typical feed location for several years.

At the possible maximum future peak day flow of 30 mgd, the estimated maximum lime usage is 2500 ppd (= 3.5 cf/hour @ 30 lb/cf) at a conservative summer dose of 10 mg/L. The existing lime feed system appears capable of feeding this higher rate if desired. Lime storage requirements for the plant's existing flow conditions (i.e. 4.9 mgd ADD and a maximum lime dose of 15.0 mg/L) are approximately 4.6 tons, or 15-percent of the storage currently available at the plant. Existing on-site storage is sufficient to meet plant demands in excess of 30 mgd. Therefore, no improvements will be required through the 20 year planning window considered for this analysis. Additionally, lime usage may decrease in future if alum dosages are decreased.

Though lime storage capacity at the plant appears more than adequate, issues associated with delivery may increase the desirable on-site storage capacity. There is currently no local vendor capable of delivering NSF certified lime; the closest vendor is located in the Bay Area. Therefore, the excess storage capacity will add flexibility to lime delivery schedules.

The current point of lime addition at the plant may be creating water quality issues in the clearwell and distribution system, including manganese oxide deposits and alum "after-floccing" in the distribution system, by raising the pH of the water leaving Basin #2. Adding the entire plant flow's lime dose in Basin #2 effluent is creating a "local" high pH (>9.0) in Filters 4 and 5, potentially re-dissolving alum floc and permanganate. These dissolved constituents equilibrate with the blended water pH and precipitate in the distribution system.

In general, pH adjustment should be delayed as long as possible through a water treatment process (often in the clearwell effluent) to optimize coagulation/filtration and to maximize the disinfection efficiency through the clearwell. pH adjustment with lime

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may be one exception as insoluble particulates inherent in the lime will naturally increase the turbidity in the finished water, potentially impacting regulatory compliance. For example, if a lime slurry (typically 20 NTU) is dosed at 20 gpm into a total plant flow of 20 mgd, turbidity in the finished water will increase by approximately 0.03 NTU. Based on these concerns, pH adjustment was moved upstream of the filters many years ago.

Some state regulatory agencies have "forgiven" this incremental increase in turbidity, as the rules were intended to minimize pathogen survival through a plant. Since it was shown that the alkaline nature of pure lime is a prohibitive environment for pathogens, some WTPs have been able to sample for turbidity prior to lime addition for regulatory reporting. Since lime is the lowest-cost pH adjustment chemical and the existing feed system is already in place, the City should consider alternatives to the current dosing location, and engage DHS regarding impacts of lime dosage on finished water turbidities. Lime doses may decrease in the future if less alum is used, meaning lower solids to clearwell and less impact on turbidities in the finished water. If it's decided that adding lime to the clearwell isn't feasible or acceptable, considering switching to NaOH or soda ash, which will require a new feed/storage system and chemical costs will increase

4.2.1.4 Polymer

The plant currently adds non-ionic polymer to the filter influent pipelines as a filter aid to improve filter performance. A dry feed system, including two 290-gallon mix/aging and feed tanks and one diaphragm positive displacement metering pump rated at 16.7 gph, are used to make and feed the solution. Dry polymer is shipped in 55-pound bags and stored adjacent to the mixing tanks in the chemical room.

At the possible maximum future plant flow of 30 mgd, the estimated maximum polymer usage is 12.5 ppd, assuming a polymer dose of 0.05 mg/L. The existing polymer feed system and storage capacity appears capable of accommodating this higher rate if desired. Improvements associated with the filters and basins will likely reduce the filter aid polymer doses in the future.

4.2.1.5 Potassium Permanganate

The plant currently adds potassium permanganate to the raw water pipeline and slow-mixing basin for taste and odor control. The permanganate feeder is a volumetric BIF type with hopper that discharges to a flushing funnel and eductor which discharges the resulting solution to the application point. Prior to application, the permanganate solution is further diluted; dilution water is controlled by a solenoid valve. Dry $KMnO_4$ is shipped in 110-pound steel drums and stored between the permanganate feeder and the polymer metering pumps.

At the possible maximum future plant flow of 30 mgd, the estimated maximum permanganate usage is 62.5 ppd, assuming an average dose of 0.25 mg/L. The existing permanganate feed system and storage capacity appears capable of accommodating this higher rate if desired.

Current dosages of permanganate are relatively high for background taste and odor control. Also, to avoid permanganate "breakthrough" (i.e. pink color reaching the filter influent channel) caused by short-circuiting through Basin #3, permanganate is not dosed equally between the basins; the majority of permanganate is dosed in Basins #1 and #2. The elevated pH in Basin #2 may be preventing precipitation of permanganate, resulting in manganese oxide carry-over through the filters and eventual deposit in the distribution system. In addition to previously recommended adjustments to the pH adjustment at the plant, the City should consider reducing the permanganate dose through the plant. A series of "trial and error" experiments are recommended to determine an appropriate dose.

4.2.2 Coagulation Performance

Rogue River water is generally considered a low turbidity/ good quality supply, but some treatment challenges exist at the WTP, resulting from wide swings in pH (seasonal as well as diurnal during the warmer months), seasonally variable turbidity, temperature, and color, as well as occasional taste and odor events. Excepting taste and odor, this variable raw water quality can significantly impact coagulation performance at the plant.

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Historically, these treatment challenges have been met using a relatively high dosage of alum. This strategy has resulted in relatively high solids production (putting a "stress" on the existing solids handling facilities by filling up the pond faster than expected after cleaning), depressed pH (corresponding to an increase in pH adjustment chemical usage/costs), and decreased overall plant efficiencies. Each of these issues is discussed in detail later in this report. Improvements to the filters and/or basins may serve to improve overall plant efficiencies. However, without these improvements, continued use of alum as the sole, primary coagulant may not be sufficient to meet performance expectations as the plant production demands increase. This section discusses some alternative coagulation strategies for the City's WTP.

Table 4-5 presents potential alternative coagulation schemes for the City's WTP.

TABLE 4-5: SUMMARY OF COAGULATION ALTERNATIVES

Coagulant Scheme	Remarks
Single Chemical	
Aluminum Chlorohydrate (ACH)/ Poly-aluminum Chloride (PACl)	• ACH may be ineffective at higher temperatures based on plant tests
Ferric Chloride/Sulfate	• Performance similar to alum • Sludge more "dewaterable" • Out-performs alum in cold water • Solids production similar to alum
Alum/Poly or Ferric/Poly Blend	• Relatively expensive vs. purchasing separately
Multiple Chemicals	
Alum + ACH/PACl	• Not as much pH depression versus alum • Sludge production similar to alum
Alum + Cationic Polymer	• Depressed pH • Significantly reduces overall alum dose • Minimizes impacts on pH • Relatively low sludge production
Ferric + Cationic Polymer	• Performance similar to alum + Cat Poly • Relatively low sludge production • May see lower settled water turbidities in winter
ACH/PACl + Cationic Polymer	• Less impact on pH than alum + Cat Poly

There are many plants in the Pacific Northwest treating river supplies similar to the Rogue, who have been successful in reducing their alum dosages by as much as 50%

using alternative coagulation chemicals. For example, the South Fork Water Board WTP (on the Clackamas River) converted from alum alone to alum plus cationic polymer in the mid-1990's, reducing alum dosage from 15-25 mg/L to an average of 6 mg/L during low turbidity events; soda ash usage was also decreased. This resulted in a net chemical cost reduction as well as minimized sludge production and increased production efficiencies. The Lake Oswego WTP and Clackamas River Water WTP both employ a combination of ACH + alum to decrease alum demands. (*NOTE: The Lake Oswego WTP also uses pH adjustment with carbon dioxide to maintain optimal pH during coagulation.*) Similarly, the Medford WTP (Rogue River supply) is currently using alum plus cationic polymer, but is considering the use of PACl alone or PACl plus cationic polymer to avoid impacts of high alum doses on pH and reduce sludge production. The City of Roseburg recently converted its Umpqua River plant to ACH from alum and uses it as a single coagulant much of the year

Though there is potential to optimize the current coagulation strategy at the WTP, these efforts must be carefully balanced with the solids loading rates placed on the filters. Historically, the relatively high alum doses have been successful in forming large, settleable floc (evident by the cleaning frequency required in the sedimentation basins). Though some alternative coagulation strategies may produce a smaller, more filterable floc at lower coagulant doses, this floc may be unable to settle in the basins, leading to an overall increase in the solids loading rate on the filters and shorter filter runs.

In addition, coagulation performance can be quite seasonal. The City experienced this seasonal performance variability during recent full-scale testing of the alternative coagulant ACH (Pelican Chemicals 801B). Preliminary results from tests conducted during the period April 10 through 19, 2002 (with an average raw water temperature of 50°F) indicated that settled water turbidity was lower and filter runs were longer compared to the use of alum alone. However, similar testing performed in July 2003 (with an average raw water temperature 67°F) resulted in poorer settled water quality, premature turbidity breakthrough and short filter runs compared alum alone. The reason(s) for the differences in performance of ACH during the two brief tests is unclear.

To fully understand the possible benefits and costs of using alternative coagulants, pilot and/or full-scale tests should be conducted seasonally under different water quality conditions using a variety of chemicals/combinations to ensure that treatment requirements and performance are well understood. An "optimal" coagulation strategy will balance plant efficiency with coagulation chemical costs, disinfection requirements, sludge production and pH adjustment requirements. See ***Appendix E*** for a summary of jar tests conducted in November 2003 using alternative coagulants.

4.2.3 Basins

A summary of historical performance from the Basins is summarized in **Section 2.4.2**. The basins currently provide contact time for disinfection and some solids removal, prior to filtration; no formal flocculation (mixing) is provided in the basins other than "mild" hydraulic turbulence. Basins #1 and #2 have a combined rated capacity of 12 mgd; Basin #3 is rated at 8 mgd, for a combined process capacity of 20 mgd. The basins provide satisfactory water for filtration most of the year. However, all basins experience challenges with regard to short-circuiting (Basin #3 is particularly vulnerable to short-circuiting), high solids loading, sub-optimal flocculation and seasonal turbidity spikes. In addition, there is no continuous solids removal system; as solids accumulate in the basins, effective volume is reduced, compromising CT compliance and settling efficiencies.

Selected existing design criteria for the existing basins are summarized in ***Table 2-2***. Design criteria considered "optimal" for pretreatment are summarized below in ***Table 4-6***. These "optimal" parameters serve as a useful comparison when considering basin improvement priorities.

TABLE 4-6: "OPTIMAL" FLOCCULATION/SEDIMENTATION DESIGN CRITERIA

Parameter	Units	Value
Settled Water Quality	NTU	< 2.0
Mixing (Flocculation)		
Mixing Time	min	20 - 30
Mixing Energy ("G x T")	-	$3x10^{-4}$ - $2x10^{-5}$
Sedimentation		
Settling Time	min	90 - 120
Length:Width Ratio	-	4:1
Length:Depth Ratio	-	1:15
Hydraulic Loading Rate	gpm/sf	0.34 – 1.0
Sludge Collection System		Continuous

Based on a comparison between "optimal" and existing basin design criteria, several improvements to the basins are recommended to ensure reliable performance at the current plant capacity.

- Incorporation of formal flocculation (either mechanical or hydraulic) for improved settled water quality
- Installation of a continuous sludge removal system to minimize short-circuiting associated with solids accumulation and to equalize sludge loading to the solids handling system
- Installation of internal baffling in Basin #3, in addition to flocculation, to minimize short-circuiting resulting from the geometric limitations of the basin

Alternatives to address these process limitations are discussed in detail in **Section 6**. The suggested improvements are intended to optimize the treatment process, and may not increase the process capacity of the basins. To meet demands in excess of 20 mgd, additional flocculation/sedimentation capacity or incorporation of "high-rate" processes (such as plate or tube settlers) is required. To avoid investments in facilities that may no longer be a part of the future treatment train, the selected strategy for meeting future demands will need to be considered prior to recommending near-term basin improvements.

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4.2.4 Filtration

Section 2 presents a detailed evaluation of historical filter performance and a discussion of possible capacity limitations. A summary of deficiencies identified as part of the historical performance analysis and filter investigation is presented below:

- Filter production efficiencies are currently 80 to 90 percent; 97 percent is considered the minimum desirable filter production efficiency.
- All filters have lost media over the years due to media carry-over during backwash; Filters 6 through 8 have lost most of the originally installed sand (either via carryover or through the underdrains); current media depths are 18 to 20-inches compared to the original design of 24-inches.
- Filters are not and can not be properly cleaned given the current, improperly "matched" media sizes and backwash pumping limitations.
- The surface wash system is ineffective due to lack of media expansion during backwash.
- Short filter runs result from relatively high filtration rates through a relatively shallow, dirty media.

With the filters' existing condition, it would be very difficult to operate the plant at the 20 mgd rate on a continuous, 24 hour per day basis, due to the short filter runs and frequent backwashes. A discussion of alternative filtration improvements to address these deficiencies is presented in **Section 6**.

4.2.5 Clearwell

The current clearwell is relatively small for a 20 mgd plant; CT compliance is only possible through the plant by carefully monitoring and controlling the chlorine residual through the Basins. The recent incorporation of VFDs on two High Service pumps helps maintain a relatively high water level in the clearwell, however, multiple "back-to-back" backwashes can create challenges to CT compliance.

Process changes, including longer filter runs, higher overall plant efficiencies and relocation of lime addition, will help ensure continued CT compliance in the near-term. However, if the Rogue River supply is determined to have excessive concentrations of *Cryptosporidium*, the LT2ESWTR may require other, non-chlorine based forms of disinfection that would result in significant plant modifications.

Clearwell volume will need to be expanded in the future when plant demands exceed 20 mgd. Ideally, the clearwell should provide at least 60 minutes at 30 mgd, or 1.25 MG of storage. Alternatives to integrate additional clearwell volume with the existing clearwell and HSPS are discussed in **Section 6**.

4.2.6 Disinfection/DBP Formation

The plant is currently capable of meeting CT within the existing basins and clearwell by using higher pre-chlorination residual and maximizing the operating level in the clearwell. However, the dependence of disinfection compliance on the contact time achieved through the basins significantly limits operational flexibility at the plant; free chlorine residual must be carefully monitored and maintained through the basins to meet CT requirements. In addition, efforts to increase the pre- and post-chlorination residual must be balanced with DBP control.

DBP and/or *Cryptosporidium* requirements may "drive" the disinfection improvements at the plant in the coming years, if on-going monitoring indicates elevated concentrations. If Grants Pass is required to inactivate for *Cryptosporidium* in the future (depending on levels in the Rogue River), installation of a disinfectant stronger than chlorine (e.g. ozone, chlorine dioxide, or ultraviolet (UV) irradiation) would be necessary, as chlorine is a relatively ineffective disinfectant for *Cryptosporidium*. Similar disinfection process modifications would need to be incorporated if results from on-going DBP tests indicate excessive concentrations of HAAs or THMs per the proposed D/DBP Rule. Discussion of improvement alternatives for each case are presented in **Section 6**.

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4.2.7 Washwater and Solids Handling System

As previously stated, the existing sludge lagoon is full and needs to be cleaned. In addition, the existing lagoon is not capable of successfully "drying" the sludge. At least a portion, if not all, of the liquid (non-dried) sludge from existing pond needs to be removed and hauled off-site immediately. Since the sludge is less than 15% solids, disposal at a landfill is not an option; an alternative site for disposal will need to be identified in the near-term. In addition to this immediate cleaning requirement, a long-term strategy for solids handling and disposal needs to be developed. The type of solids handling process appropriate for consideration depends largely on the methods available for disposal.

For preliminary analysis of sludge handling alternatives, an estimate of sludge production (both today, as well as future production) is required. Sludge production rate can be estimated using the following equation (Kawamura, 2001):

1. **Sludge (dry lb/MG) = 8.34x[(Alum dosage (mg/L)x0.26)+(Turbidity (NTU)x1.3)]**

Based on Equation 1, ***Table 4-7*** summarizes annual as well as seasonal average sludge production at the WTP for various peak day demands.

TABLE 4-7: SLUDGE PRODUCTION ESTIMATE BASED ON CURRENT ALUM USAGE

Peak Day Flow (mgd)	Sludge Production (dry weight)					
	Annual Average[1]		Peak Season Average[2]		Off-Season Average[3]	
	lb/day	ton/year	lb/day	ton/season	lb/day	ton/season
10 (current)	385	69	339	26	430	44
15	578	104	508	39	645	65
20	770	139	678	51	859	87
25	963	173	847	64	1074	109
30	1155	208	1016	77	1289	131

[1]Based on a peaking factor of 2.14
[2]Based on a peak day:peak season ratio of 1.44; Peak season is defined as June – October
[3]Based on a peak day:off-season ratio of 3.28; Off-season is defined as November - May

A detailed discussion of alternative solids handling and disposal methods is presented in **Section 6**.

4.2.8 Summary of Process Capacity Evaluation

- All chemical systems appear to be adequate to serve the next 10 to 20 years except for periodic maintenance and replacement. This equipment may need replacement when plant is expanded to 30 mgd
- Adjust the alum carrier water to ensure alum dilution remains above 5 percent prior to injection at the static mix vault.
- Keep lime as primary pH adjustment chemical (less costly alternative), but relocate the point of addition near end of clearwell to avoid interference with filter performance and disinfection efficiencies. This will likely require construction of new chemical feed pipelines. The City should discuss impacts of lime addition on plant effluent turbidity with DHS to ensure continued compliance with finished water turbidity requirements. If not successful, addition of a new NaOH or soda ash system to adjust pH in clearwell will be required.
- The City should try and reduce the potassium permanganate dosages and study the impacts on taste and odor control. The current permanganate dose is relatively high compared to similar plants with "background" taste and odor issues.
- To fully understand the possible benefits and costs of using alternative coagulants, pilot and/or full-scale tests should be conducted seasonally under different water quality conditions using a variety of chemicals/combinations to ensure that treatment requirements and performance are well understood. An "optimal" coagulation strategy will balance plant efficiency with coagulation chemical costs, disinfection requirements, sludge production and pH adjustment requirements.
- Incorporation of formal flocculation prior to sedimentation in all Basins is recommended for improved settled water quality during "challenging" water treatment conditions.
- Installation of continuous sludge removal systems in the basins is recommended to equalize solids loading to the solids handling system, to maximize the contact time by

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minimizing solids accumulation, and to eliminate the need for taking basins "off-line" for cleaning.

- The City should make upgrades to the filters (media and underdrains) to increase plant efficiencies and to ensure continued compliance with water quality regulations. Modifications should include a deeper filter media to improve production efficiencies and provide for better cleaning.
- The existing surface wash system is currently ineffective. Improvements to the existing system are recommended to ensure proper media cleaning during backwash.
- The City should experiment with the current backwash rates and durations to better optimize cleaning of the existing media, and to potentially reduce backwash water usage.
- The plant is currently capable of meeting CT requirements. The clearwell will need to be expanded as plant demands increase; these needs should be addressed during expansion, or if future regulations require a change in disinfection strategy at the plant.
- The City should continue to monitor the impacts of increased pre- and post-chlorination residuals on the formation of DBPs in the distribution system. Planning for future improvements is recommended to better prepare for impacts of future regulatory requirements.
- The existing sludge lagoon is full and needs to be cleaned. In addition, a long-term strategy for solids handling and disposal should be developed.

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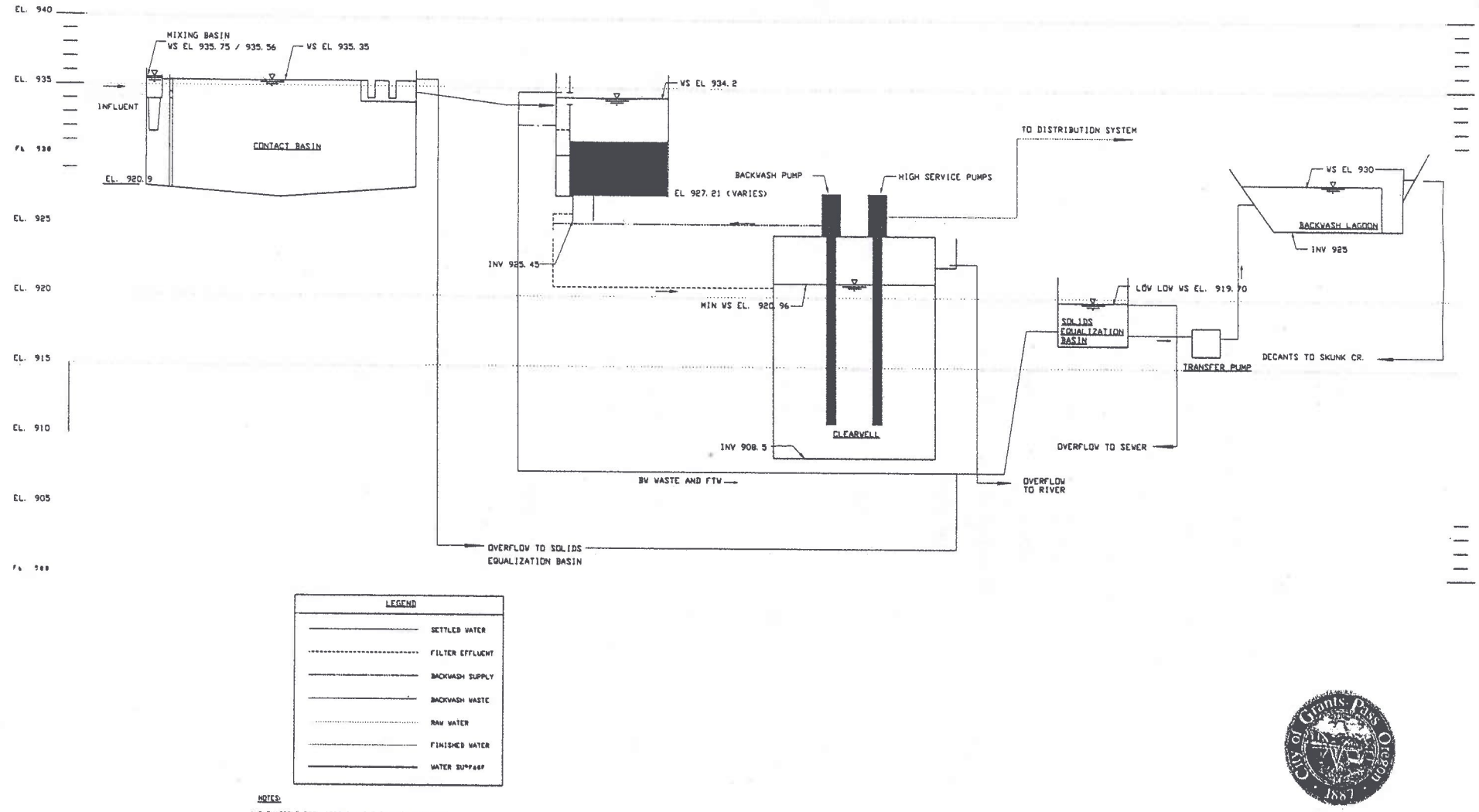

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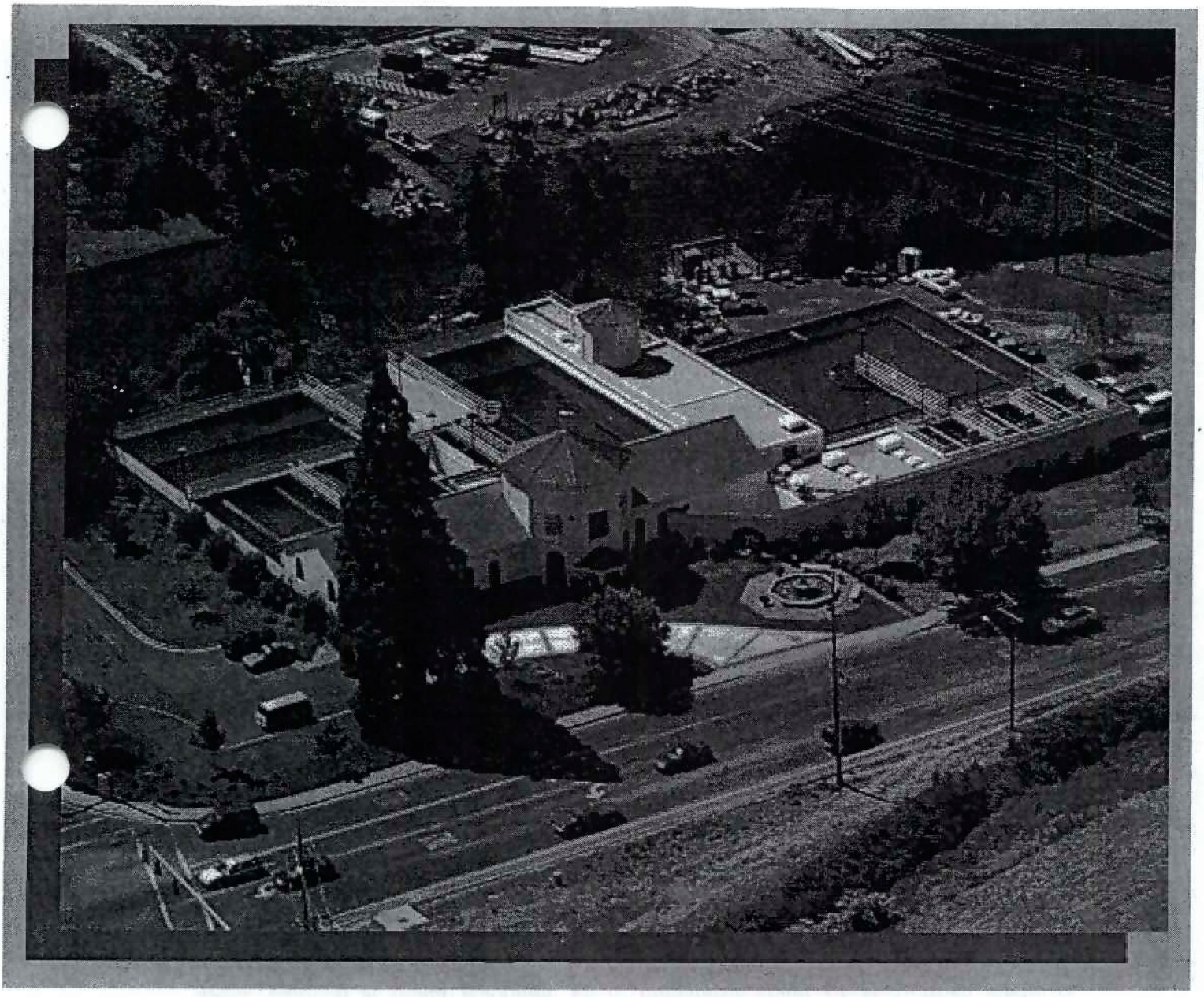
Figure 4-1
City of Grants Pass
Water Treatment Plant Facility Plan
Existing WTP Hydraulic Profile



Facilities Review

5 FACILITIES REVIEW

The final element of the WTP Evaluation is the Facilities Review. Each of the existing plant's major systems and structures were reviewed to determine if capital improvements are required, and to estimate remaining useful life. The results of this review are integrated with the Regulatory and Capacity Reviews to develop a Capital Improvement Program to maintain existing capacity and to increase capacity if so desired.

5.1 PLANT EQUIPMENT INVENTORY

Table 5-1 contains an inventory of major plant equipment. The following is a discussion of each major system, including pertinent information and observations used to determine remaining useful life as well as suggested capital improvements associated with the equipment.

5.1.1 Raw Water Intake and Pump Station

The intake and pump station were constructed in the early 1980's as part of the last major plant expansion. The intake is equipped with one travelling screen and a wetwell "de-silting" system. The four raw water pumps were installed in 1983 when the new intake facility was constructed with space available to add two more pumps. Since installation, the pumps have been re-built, and the pump impellers replaced. The pumps appear to be functioning appropriately and with continued maintenance and repair, should have significant remaining useful life. As described in previous sections, the "firm" raw water pumping capacity is 15 mgd, installation of an additional pump is required, when demands approach 15 mgd, to reliably deliver 20 mgd.

The Technical Memorandum in ***Appendix D*** reviews the status and compliance of the intake and pump station. As discussed in the TM, the intake does not comply with current fish protection screening criteria and significant modifications are required to bring it into compliance. Until the intake is modified with a different type of screening system, the City should make limited investments in the existing travelling screen. It is not likely to be used with the modified intake, but it requires some maintenance and repair to keep it operational over the next few years.

The raw water pumps have performed well and are in no need of immediate attention. The City is contemplating the addition of a new VFD on one of the raw water pumps to provide better flow control of the plant, and MWH supports this proposed improvement.

5.1.2 Chemical Systems

The plant has five chemical storage and feed systems, including:

- Liquid alum
- Liquid sodium hypochlorite
- Hydrated lime
- Dry polymer
- Dry potassium permanganate ($KMnO_4$)

In general, all chemical feed systems are in good condition, and can reliably meet the City's needs for many years. However, this equipment has a finite useful life, and will likely need to be replaced once within the 20 year planning horizon considered for this report. The replacement schedule will depend on when the equipment was installed, and is hard to predict. The City should also consider chemical feed system replacements when the plant capacity is expanded.

The liquid alum storage tanks are not currently protected from leaks should the tank become damaged. Construction of a wall around the base of the alum tanks is recommended to contain potential leaks. The containment system should be designed to hold the maximum volume of alum (12,000 gallons), in addition to 2-hours of fire-sprinkler per building code requirements. However, the chemical storage area is not currently protected by fire sprinklers, so the containment volume could possibly be reduced. Including the sprinkler volume, an approximate 3-feet high containment wall around both tanks is required. A step-ladder should also be provided for tank access.

5.1.3 Sedimentation Basins

Basin #1 was built as part of the original plant construction in 1931 and is therefore over 70 years old. Basins #2 and #3 were added to increase plant capacity in 1950 and 1983, respectively. The concrete in all basins appears to be structurally sound and have many years of remaining useful life; few cracks in the exterior walls were observed. The launders in all basins show little sign of deterioration and are in fair condition.

In order to improve the basins' solids removal capabilities, all of them require formal flocculation. Basin #3 also requires the installation of internal baffles to minimize short circuting. Once the decision is made to make improvements to the basins, the City should take a more serious look at the structural integrity of the basins and launders, and repair any cracks in the basin walls. In addition, the launders in Basin #2 oscillate during high flows, and should be reinforced. Similar improvements have been performed on Basin #1.

5.1.4 Filters

Filters 1 through 3 were built as part of the original plant construction in 1931 and are over 70 years old. Filters 4 and 5 were added in 1950. Filters 6 through 8 were added as part of the most-recent plant expansion project in 1983. Structurally, the filters appear to have many years of remaining useful life.

As discussed in **Section 2** and **Section 4**, improvements to the existing filter media, underdrains and surface wash system are recommended to increase plant production efficiency and to ensure continued compliance with water quality regulations. Alternatives for these filter improvements are discussed in detail in **Section 6**.

The washwater troughs in several of the filters have significant cracks and leak during backwash; several have 2-inch holes associated surface wash pipes that have since been relocated. To ensure optimal flow distribution and minimize media carry-over during backwash, these leaks should be repaired.

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The location of the filter effluent flowmeters prevent the measurement of filter-to-waste flows, preventing the ability to monitor the filter flow during initial startup and to assist with "seamless" transition from filter-to-waste to filter production, thereby potentially compromising filtered water quality. The existing filter effluent flowmeters lack adequate lengths of upstream and downstream "straight-pipe", significantly reducing the accuracy of the meters. Replacement of these meters with a type that have less upstream/downstream "straight-pipe" requirements, such as magnetic-type flowmeters, is recommended. It is therefore recommended to install new filter flowmeters that can also measure filter-to-waste flows.

All of the suggested filter improvements, including valve/actuator replacements discussed later in this Section, should ideally be completed as part of one construction effort for economies of scale and for ease of sequencing filter outages during construction. This work can not be done during the peak summer demands season as all eight filters are required to meet demands, but any seven of the existing eight filters can provide adequate treatment and capacity during other times of the year when the plant operates at lower rates. The City should consider making filter gallery improvements in unison with filter media/underdrain improvements for economy-of-scale reasons and to minimize plant disruption.

5.1.5 Clearwell

The 433,000 gallon clearwell, which serves as a wetwell for the high service and backwash pumps, and a contact basin for disinfection, appears to be structurally sound and has significant remaining useful life. The clearwell is actually comprised of three interconnected clearwells, one located under each group of filters and built at different times. A common filtered water channel currently routes all filtered water to the east clearwell (located beneath Filters 1 through 3), where it is chlorinated, then directed through a series of serpentine baffles through the center and west clearwells and finally to the finished water pump area in the west clearwell.

A recent inspection of the clearwell(s) by the City indicated no major structural deficiencies, but did identify that the inter-connecting pipe (actually a piece of culvert pipe) between the area underneath Filters 4 and 5 appears to be leaking. This section of pipe should be replaced.

Additional clearwell volume should be added when the plant's capacity is increased, preferably to provide a minimum of 1-hour of detention time at peak flow, but with enough volume to provide for successive filter backwashes at 70,000 gallons each without compromising disinfection performance.

5.1.6 High Service Pump Station

The high service pump station consists of two large pumps, two medium pumps and one small pump, installed in 1961, 1983 and 1983, respectively. The high service pump station is currently rated for a firm capacity of 16.7 mgd, with a maximum pumping capacity of 21 mgd with all five pumps operating. All of the pumps and motors have been re-built within the last 15 years and the City has budgeted for at least one pump/motor re-build over the next 5 years. With continued maintenance and repair, the pumps appear to be capable of continued service throughout the 20-year planning horizon considered for this report.

The original backwash pump was installed in 1983 as part of the plant expansion project. A back-up backwash line, connected to the high service discharge header, was also installed but has never been used due to a lack of pressure/flow control. The backwash pump has required little maintenance according to plant staff, and appears to be functioning appropriately. The pump should have significant remaining useful life. Since there is currently no back-up backwash supply, increased inspections and service are recommended on a semi-annual basis to ensure that major repairs are minimized. As demands increase, improvements to the back-up supply are recommended to avoid extended backwash down-time. The City's preferred option is to purchase a complete pump and motor to have it available at the plant in case the existing pump fails unexpectedly.

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5.1.7 Flowmeters

Both the raw water and finished water pipelines are equipped with Venturi-type flow meters; the backwash flow also used to be measured with a similar type of meter. The pressure sensing tubing associated with these meters are prone to collecting air bubbles, significantly decreasing the accuracy of the meter. The City recently replaced the backwash flowmeter with a magnetic-type flow meter. Replacement of the raw and finished water flow meters with similar meters is recommended when the budget will allow. Replacement of the Basin #3 influent flowmeter is also recommended to better monitor and control flow-split between basins.

5.1.8 Major Valves and Actuators

Most pneumatic actuators were installed prior to 1980 except those installed in Filters 6 through 8, during the most-recent plant expansion. All pneumatically-operated filter valve actuators are old and in need of repair; replacement parts for these actuators are becoming increasingly difficult to obtain. Replacement of these actuators with modern electric valve actuators for ease of control and maintenance is recommended. All air piping in the filter galleries should be removed as part of the actuator replacement project. Several valves, including the filter influent valves and the backwash valves, leak and are in need of replacement/repair. The City should also consider installing new valves with the actuator replacements since the valves are relatively inexpensive compared to the electric actuators and it will benefit installation and warranties if new valves are provided along with new actuators. These improvements should be made in conjunction with other filter gallery piping and flowmeter improvements.

5.1.9 Air Compressor System

The plant is equipped with two compressor/air receiver systems located in the High Service Pump Room. Both systems provide plant air to operate the pneumatic valve actuators for the filters, as well as providing air to keep the surge tank pressurized. Both systems have required little maintenance, and appear to be functioning properly. It is expected that these systems have many years of useful life remaining, although they may

not be required in the future, once all the pneumatic valve actuators are replaced with electric actuators.

5.1.10 Washwater and Solids Handling

Depending on the long-term strategy for solids handling at the plant, significant improvements to the solids and washwater lagoon may be required. Improvement alternatives for solids and waste washwater handling are presented in **Section 6**.

The equalization basin contains 3 transfer pumps which deliver washwater and solids to the lagoon. The two smaller pumps were installed as part of the 1983 expansion and the larger pump was installed a few years ago. The City intends to remove one of the original smaller pumps and replace it with a higher capacity pump to increase pumping capacity and reliability. With continued maintenance, the washwater piping and pumps have significant useful remaining life and require no major capital investments.

5.1.11 Water Quality Testing and Monitoring Facilities

The plant utilizes on-line water quality instrumentation and bench-top equipment to monitor and control plant performance. Raw water turbidity is continuously monitored using a HACH Surface Scatter on-line analyzer. Settled water turbidity from each basin is also continuously monitored using individual HACH 1720D turbidimeters for process optimization. Each filter is equipped with an on-line turbidimeter (HACH 1720D) to monitor filter performance and ensure regulatory compliance. If the turbidity from a filter rises above 0.12 NTU, then the filter is backwashed. A similar on-line turbidimeter is installed on the HSPS discharge header pipe to continuously monitor the combined filtered water quality exiting the plant. All turbidimeter signals are integrated into the SCADA system. Installation of individual particle counters on the filter effluent is recommended to better predict turbidity breakthrough in the future and ensure continued regulatory compliance.

The plant is equipped with on-line finished water pH analyzer (HACH EC 310) to continuously monitor the plant effluent pH to monitor for corrosion control compliance. Raw water and settled water pH are measured periodically each day via grab samples.

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One on-line chlorine residual analyzer (HACH CL-17) is used to monitor the plant effluent residual from the HSPS discharge header. Pre-basin and settled water chlorine residuals are measured periodically each day via grab samples.

The plant's laboratory appears to be equipped with adequate bench-top analytical equipment to perform routine daily testing for monitoring and control. It is recommended that the plant invest in a UV_{254} spectrophotometer to better monitor TOC removal through various stages in the treatment process.

5.1.12 Instrumentation & Control Systems

The plant has a Windows-based SCADA and control system that is operated via a central computer station. The existing control systems were installed as part of the SCADA improvements in 2002, and should have significant remaining useful life. As new systems and equipment are added to the plant, the SCADA system will need to be modified and integrated accordingly.

As technology evolves, the SCADA system at the plant will likely require additional upgrading. During the 20-year planning horizon considered for this report, replacement hardware and software may be needed to stay current with developing technology. These improvements and upgrades should be made via operating budget investments at the appropriate time and there are no capital investments included in this Plan.

5.1.13 Electrical Systems

The plant's electrical power is provided via a 1,500 kVA main transformer located on the plant site. The electrical service and transformer were upgraded during the 1983 plant expansion project. The existing plant electrical service and transformer appear to be adequate over the next 20 years as demands increase to 20 mgd. Improvements to the electrical system capacity and service need be addressed as part of future expansion projects or if major new electrical loads are added prior to the expansion.

The plant has not experienced any prolonged or severe power outages over the past 20 years. During "normal" power outages, service has been restored within 1 to 2 hours. This historical level of power service is expected to continue, but there is no guarantee that the City will not face an extended power outage during critical periods in the future when water production would be prohibited.

Some water treatment facilities are equipped with backup/emergency power sources, such as generators, which can allow a minimum level of water production in case of an extended power outage by the service provider. Some water providers also have dual electrical feeds from different parts of the power grid to reduce the risk of an extended outage. The City will have to decide if investments in backup power supply is warranted considering the risk of an extended outage. Addition of a backup generator is included in the 20-year CIP.

5.1.14 Control Building

The City should consider improvements to the HVAC system to provide efficient climate control; temperatures are often too hot in the summer and too cold in the winter. Improvements to update heating and cooling systems in the Control and Break Rooms located within the Control Building are recommended.

There is currently limited space available for storage and maintenance/repair within the Control Building. As demands increase, storage requirements for dry chemicals will increase, exacerbating the storage limitations. Improvements to increase the available storage and working space at the plant site are recommended.

5.1.15 Other Code Compliance Issues

The WTP was cursorily reviewed for its conformance to current regulatory codes and standards, including seismic and structural integrity, building code conformance, OSHA and ADA compliance. This information will help identify further needs and planning-level costs associated with future efforts.

The general construction of the Control Building and older Basin and Filter structures probably do not meet current building code requirements for seismic-resistant structures. There have been several earthquakes in the Pacific Northwest over the past 10-years that could have severely damaged the plant had they occurred in proximity to Grants Pass. A system vulnerability study is recommended to define the plant's and entire water system's vulnerability to seismic events. Anticipated improvements as part of this project include installation of pipeline restraints and reinforcement of concrete structures, especially the older basins and filters.

The walkways around the filters and basins are protected by guardrail. The spacing between horizontal railing may be too large to meet current OSHA requirements. No improvements are recommended at this time.

The plant access and pathways does not meet current ADA compliance requirements. The City should formally decide whether it desires to make the WTP ADA-compliant or provide a statement of non-compliance.

5.1.16 Integration of Vulnerability Assessment

The City has recently completed a Vulnerability Assessment (VA) of its water system per EPA requirements. It was decided to keep the recommendations of the VA Study separate from this WTPFP document. There may be some capital improvements recommended from the VA Study which could be integrated with improvements recommended by this Plan.

5.1.17 Summary of Facilities Review

- All chemical feed systems are in good condition, and can reliably meet the City's needs for many years. However, this equipment has a finite useful life, and will need to be replaced once within the 20 year planning horizon considered for this report. The replacement schedule will depend on when the equipment was installed, and is hard to predict, so is shown as a longer-term CIP item within the 20-year CIP. The

City may also need to replace/upsize chemical feed systems when the plant capacity is expanded.

- The liquid alum storage tanks are not currently protected from leaks should the tank become damaged. Construction of a wall around the base of the alum tank is recommended to contain potential leaks.
- The launders in Basin #2 oscillate during high flows, potentially compromising process performance. Installation of lateral supports, similar to those installed in Basin #1, are recommended during basin modifications.
- The washwater troughs in several of the filters have significant cracks and leak during backwash; several have 2-inch holes associated surface wash pipes that have since been relocated. To ensure optimal flow distribution and minimize media carry-over during backwash, these leaks should be repaired during filter modifications.
- All pneumatically-operated filter valve actuators are old and in need of repair; replacement parts for these actuators are becoming increasingly difficult to obtain. Replacement of these actuators with modern electric valve actuators for ease of control and maintenance is recommended. The City should also install new valves with the actuator replacement since the valves are relatively inexpensive compared to the electric actuators and it will benefit installation and warranties if new valves are provided along with new actuators.
- The location of the filter effluent flowmeters prevents the measurement of filter-to-waste flows, resulting in potential operations and water quality problems. The existing flowmeters lack adequate lengths of upstream and downstream "straight-pipe", significantly reducing the accuracy of the meters. Therefore, replacement of the filter effluent flowmeters is recommended along with piping changes to integrate filter-to-waste flow measurement.
- All of the suggested filter improvements, including valve/actuator replacements discussed later in this Section, should ideally be completed as part of one construction effort for economies of scale and for ease of sequencing filter outages during construction. This work can not be done during the peak summer demands season as all eight filters are required to meet demands, but any seven existing filters can provide adequate treatment and capacity during other times of the year. This

construction may be integrated with the filter media/underdrain rehabilitation effort for further economies of scale and to minimize plant disruptions.

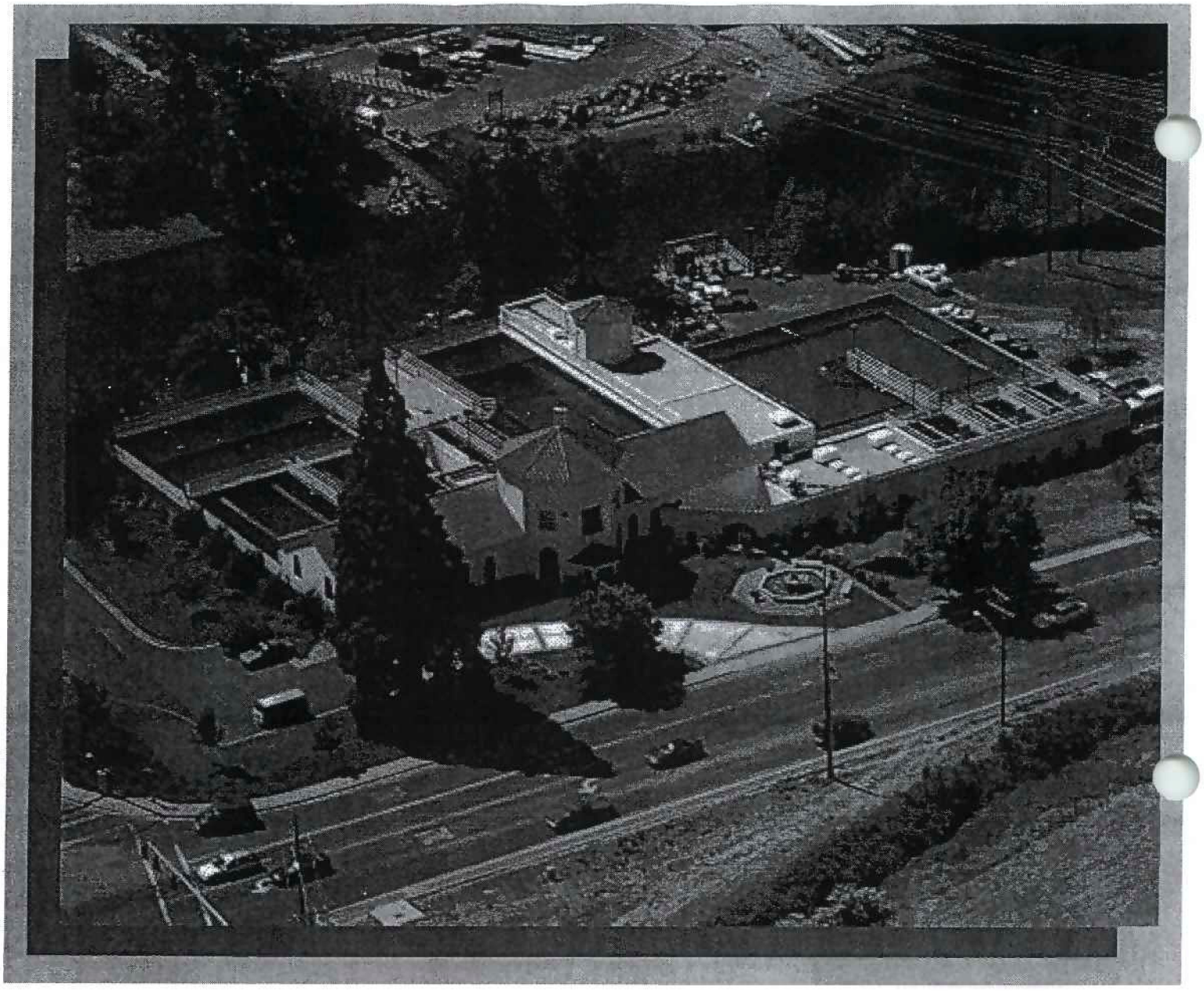
- Replacement of the raw and finished water flowmeters with magnetic meters is recommended for consistency with the new backwash flowmeter. Replacement of the Basin #3 influent flowmeter is also recommended to better monitor and control flow-split between basins.
- It is recommended that the plant invest in a UV_{254} spectrophotometer to better monitor TOC removal through various stages in the treatment process.
- As technology evolves, the SCADA system at the plant will likely require additional upgrading. During the 20 year planning horizon considered for this report, replacement hardware and software will be needed to stay current with developing technology.
- The City should consider improvements to the HVAC system to provide efficient climate control; temperatures are often too hot in the summer and too cold in the winter. Improvements to update heating and cooling systems in the Control and Break Room in the Control Building are recommended.
- There is currently limited space available for storage and maintenance within the Control Building. As demands increase, storage requirements for dry chemicals will increase, exacerbating the storage limitations. Improvements to increase the available storage space at the plant site are recommended.

Table 5-1
Inventory of Existing Grants Pass WTP System

Unit Process/Components	No.	Type	Manufacturer/Model	Capacity/Size
Screening				
Raw Water Intake Screen	2	Stationary Bar		
Wash System		Travelling Screen		
Raw Water Pumping				
Raw Water Pumps				
Pump #1	1	Vertical Turbine	Worthington/ 15HH-340	75-hp/3200gpm/65-ft
Pump #2	1	Vertical Turbine	Worthington/ 15HH-340	75-hp/3200gpm/65-ft
Pump #3	1	Vertical Turbine	Worthington/ 15HH-340	75-hp/3200gpm/65-ft
Pump #4	1	Vertical Turbine	Worthington/ 15HH-340	75-hp/3200gpm/65-ft
Chemical Feed				
Alum				
Storage	2	Fiberglass Cylindrical		6000 gal
Metering Pumps	2	PD Diphragm	JAC/Model 1212-21-9612	24 gph/125 psi
Lime				
Hopper	1			1900 cf
Volumetric Feeder	1	Volumetric Screw Auger	BIF/Model 25-12	
Mixing Tank	1	Stainless Steel		50 gal
Mixer	1	Propeller	GE/Model C242	1/4 hp / 1725 rpm
Slurry Pump	1	Constant Speed	Goulds/Model 3196	1.5"X6"/40gpm/16ft/1150rpm
Air				
Compressor #1	1	Twin Units	Quincy/Model 325L	5hp/19scfm/130 gal receiver tank
After Drier #1	1		Zum/Air & Gas Drier	
Compressor #2	1	Twin Units	Baldor/Model M3104	1/5hp/10scfm
After Drier #2	1		Honeywell/Model 8010	1/6hp
Permanganate				
Storage		Stored in Metal Buckets		
Feed Unit	1	Hopper/ Feeder/ Mixer	BIF/Model 25-06	
Polymer				
Storage	2	Stainless Cyl, Open-top		290 gal
Mixing	2	Propellor	Neptune Model D-4.00	480 rpm
Volumetric Feeder -- Pump	1	Positive Displacement	BIF/Proportioneer/Chemofeeder	
Hypochlorite				
Storage	3	Cyl FRP	RTP, Inc	2,120 gal
Pre-chlor Metering	1	PD Diphragm	Wallace and Teiman/ Encore 700	0.75 hp/16.7gph
Post-chlor Metering	1		Wallace and Teiman/ Encore 700	0.75 hp/16.7gph
Back-up Metering	1		Wallace and Teiman/ Encore 700	0.75 hp/16.7gph
Transfer	1	Seal-less Magnetic	Iwaki Seal-less Magnetic Drive	1 hp/50 gpm
Filtration				
Backwash Pump	1	Vertical Turbine w/VFD	Peabody Floway/ Model 22-BLK	200 hp/7000 gpm
Surface Wash System				
Filters 1,2,3				
Filters 4,5				
Filters 6,7,8				
On-line Monitoring				
Turbidity				
Raw Water	1	Digital- Integrated in SCADA	HACH Surface Scatter	
Settled Water				
Contact Basin #1	1	Digital- Integrated in SCADA	HACH 1720D	
Contact Basin #2	1	Digital- Integrated in SCADA	HACH 1720D	
Contact Basin #3	1	Digital- Integrated in SCADA	HACH 1720D	
Filter Effluent				
Filter #1	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #2	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #3	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #4	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #5	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #6	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #7	1	Digital- Integrated in SCADA	HACH 1720D	
Filter #8	1	Digital- Integrated in SCADA	HACH 1720D	
Plant Effluent	1	Digital- Integrated in SCADA	HACH 1720D	
Chlorine Analyzer				
Mixed Water				
Clearwell		HACH CI-17		
Flow Meters				
Raw Water	1	Venturi Differential Pressure		
Filter Effluent				
Filter #1	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #2	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #3	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #4	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #5	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #6	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #7	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Filter #8	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-100 inches of water
Backwash	1	Orifice Differential Pressure		
Finished Water	1	Venturi Differential Pressure		

Table 5-1
Inventory of Existing Grants Pass WTP System

Unit Process/Components	No.	Type	Manufacturer/Model	Capacity/Size
Filter Headloss				
Filter #1	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #2	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #3	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #4	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #5	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #6	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #7	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
Filter #8	1	Orifice Differential Pressure	Bristol/ACCO Signature	0-700 inches of water
pH				
Raw Water				
Clearwell			HACH EC 310/PS1202	
Point of Entry				
HSPS				
Finished Water Pumps				
Pump #1	1	Vertical Turbine	Fairbanks Morse/ Model 18HC	300 hp/4000 gpm/ 210 ft
Pump #2	1	Vertical Turbine	Fairbanks Morse/ Model 18HC	300 hp/4000 gpm/ 210 ft
Pump #3	1	Vertical Turbine	Worthington/ Model 15HH-340	250 hp/3500 gpm/ 210 ft
Pump #4	1	Vertical Turbine	Worthington/ Model 15HH-340	250 hp/3500 gpm/ 210 ft w/ VFD
Pump #5	1	Vertical Turbine	Worthington/ Model 15HH-277	250 hp/2600 gpm/ 210 ft w/VFD
Sump Pump				
Waste Water				
Sewage Pumping				
Pumps	2	Submersible	Peabody Barnes/ Model 45E154E	1.5 hp/ 100 gpm/ 22 ft
WWW and Solids Equalization Basin				116,000 gal
Pumps	2	Quick-disconnect Submersible	Peabody Barnes/ Model 6GSEH2004	30 hp/ 1,500 gpm/ 36 ft
Pumps	1	Quick-disconnect Submersible		60 hp/ 1,750 gpm/ 60 ft
Plant Sump				
Pumps	2	Quick-disconnect Submersible	Peabody Barnes/ Model 65E1003	10 hp/ 830 gpm/ 15 ft



Facilities Planning for Grants Pass WTP

6 FACILITIES PLANNING FOR THE GRANTS PASS WTP

Based on the findings and information presented in Sections 2 through 5, the City's existing Water Treatment Plant (WTP) is capable of treating and delivering potable water for the 20-year planning horizon and beyond. Significant improvements are required to maintain the existing 20 mgd rated capacity, to ensure continued compliance with increasingly-stringent drinking water quality and other regulations, and to improve operations and cost-effectiveness for the plant's remaining useful life. The WTP's capacity can also be expanded up to 30 mgd with significant improvements.

The WTP is now operating "at capacity" even though the current maximum day production is less than 11 mgd because the plant is not operated for 24 hours per day. The plant is operated for 12 to 15 hours per day during peak demand periods, often at the 20 mgd rated production capacity, to make the required daily volume of water. If water demands continue to increase as projected, the plant will have to be operated for longer durations each day until the maximum daily production capacity (20 mgd) is reached. At that time (currently projected for approximately year 2025), the plant will have to be expanded or an alternative source of supply needs to be implemented. To meet the longer operating periods as demands increase, the City will eventually require additional operations staff.

The recommended plant improvements to be implemented at the WTP, along with detailed analyses of key issues, are presented in this Section of the report. This Section concludes with prioritized capital improvements and costs for the WTP over the next 20 years and beyond.

6.1 CONCLUSIONS AND RECOMMENDATIONS FROM PLANT EVALUATION

This Section summarizes the major conclusions and recommendations based upon the evaluation of the City of Grants Pass WTP. Major topics addressed herein include plant capacity, treatment processes, regulatory compliance, support facilities, and monitoring/control issues. Addressing these topics in a prioritized and systematic fashion

will ensure that the WTP continues to serve the City for 20 years and beyond as the primary source of potable water.

6.1.1 Plant Capacity

- The current maximum daily production is 10.5 mgd with the plant operating 12 to 15 hours per day during the peak demand season, often at a 20 mgd production rate.
- The plant's existing hydraulic capacity is capable of supporting the existing 20 mgd design rate for a 24-hour period.
- The plant's main unit processes, including flocculation/sedimentation and filtration, require improvements to reliably provide 20 mgd capacity under all water quality and operating conditions.
- The plant can meet disinfection requirements under all flow and water quality conditions by carefully controlling the pre-chlorination process to achieve target residuals and by also maintaining the clearwell level as full as possible.
- The plant and site appear capable of supporting an ultimate maximum capacity of 30 mgd with significant improvements. A plant expansion will be required in the next 20 to 25 years if demands continue to increase as they currently are.
- The City should continue to develop and protect its water rights on the Rogue River.
- The existing raw water and finished water pumps have a firm, reliable capacity of 15 mgd and 16.7 mgd, respectively, and additional pumps should be added when the maximum daily demand reaches these production rates, in approximately 10 to 15 years.
- The existing intake is hydraulically capable of withdrawing the maximum flow, but current fish protection (screen) criteria are not being met. Improvements are required to meet fish screen criteria, and the City should seriously consider expanding the intake's capacity to 30 mgd as part of these improvements.
- A new flowmeter should be installed ahead of Basin #3 to accurately and reliably monitor and control the flow split between the 3 basins.
- Filter effluent flowmeters should be relocated to measure filter-to-waste flows for reliable filter control. As part of this process, the City should consider installation of

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new flowmeters which require minimal upstream and downstream "straight pipe" for increased meter accuracy and decreased headloss through the meter.

- Reliable plant production capacity is currently vulnerable to an extended outage if the existing single backwash pump fails. The City should invest in a reliable backwash backup system, either by purchase of a spare pump and motor (and perhaps installing it), or improving the existing inter-tie with the high service header.
- Replacement of the backwash discharge header pipe through the Filter 4 and 5 gallery is necessary to eliminate leaking and to remove operator-imposed limitations on capacity and pressure in the pipe. Depending on the type of pipe (asbestos lined), building codes may mandate replacement of the entire pipe system within the gallery.

6.1.2 Treatment Processes

- In general, the plant and filters have performed well with regard to finished water quality; the plant has consistently met regulatory requirements for filtered water turbidity.
- Plant production efficiencies are typically 80 to 90 percent throughout the year, and generally decrease in the winter when total production is lower and the water is colder and more turbid. Plant efficiencies should be improved to minimize costs associated with plant operations (longer operation time, pumping and chemical costs, sludge production). Efficiencies of 97 percent are considered the minimum desirable filter production efficiency. Plant efficiencies can be improved by increasing the filter run lengths, which can be via improvements to the filters and sedimentation basins, as well as possibly improving the coagulation process.
- Based on our analysis, short filter runs result from relatively high filtration rates through a relatively shallow, dirty media. Filter media should be replaced with a new design, maximizing the overall media depth.
- The filters are dirty and can not be properly cleaned with the current backwash regime. Poor cleaning leads to higher initial headloss, which reduces the available head for filtration, resulting in shorter filter runs and decreased plant efficiencies. Optimum cleaning can be accommodated with an optimum media and underdrain system design.

- The plant can modify its existing backwash sequence and volumes to slightly improve production efficiency until filter modifications are completed.
- There is potential to optimize the current coagulation strategy at the WTP to reduce chemical usage, reduce sludge production and increase plant production efficiencies. However, these efforts must be balanced with the overall solids loading on the filters and seasonal performance variability resulting from a change in coagulants. Jar testing conducted as part of this planning effort were inconclusive. Staff should continue experimenting with different coagulants.
- Operators report manganese oxide deposits throughout the distribution system. This may result from either: 1) overfeeding of potassium permanganate, 2) sub-optimal pH ranges for permanganate solubility in Basin #2, or 3) both. The continued use of potassium permanganate at the plant needs to be reviewed and optimized. It is possible that permanganate doses can be reduced compared to historic usage rates.
- Similarly, operators report alum "after-floccing" in the distribution system, likely resulting from sub-optimal pH characteristics in Basin #2. The City should consider relocating lime addition point to downstream of the filters. An increase in finished water turbidity may result from the "inert" particles associated with lime addition to the clearwell. The City should discuss impacts of lime addition on plant effluent turbidity with DHS to ensure continued compliance with the regulations.
- If lime is found to no longer be a viable option for pH adjustment at the plant, alternatives to lime, including soda ash and caustic soda, should be considered. Chemical costs associated with these alternatives may be substantially higher when compared to lime. Also, there are space limitations for a new chemical injection/storage system on site.
- A long-term plan for solids handling and disposal is needed for the plant. The sludge lagoon is full and requires immediate cleaning to support another operating season. The City needs to implement solids handling improvements at the plant to support the long-term disposal option.
- The basins currently need to be cleaned of accumulated solids twice per year and this cleaning cannot be performed during the summer when all basins are required for treatment. Solids accumulation in the basins reduces the plant's performance and

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reduces the contact time for disinfection. When the basins are cleaned, slug loads of solids overload the solids handling system. As plant demands and solids production increase, the basins will require more frequent cleaning. It is recommended that an automated, continuous sludge removal system be installed in the basins.

6.1.3 Regulatory Compliance

- A review of historical compliance records indicates that the Grants Pass WTP has met all primary and secondary drinking water standards since 1998. There are no immediate requirements to modify the plant to meet current primary drinking water regulations.
- Further optimization of chlorine disinfection through the plant to reliably meet CT requirements is needed, including moving the lime addition point, increasing the chlorine residual through the basins and increasing the minimum allowable clearwell water level.
- Increasing the chlorine dose for CT compliance may increase the concentrations of disinfection by-products (DBPs) in the distribution system. The City will have to closely monitor the DBP concentrations with respect to meeting the future Stage 2 DBP Rule.
- The City should develop new DBP sampling and monitoring protocols per the Stage 2 DBP Rule (using ISDE methodology) to better prepare for future DBP regulations.
- The City should update its plant Disinfection Profile based on modifications to the disinfection process.
- Frequent tracking of TOC removal through the treatment plant, using UV_{254} as a surrogate parameter, is recommended to better define seasonal water quality variations and organics removal, and to help understand the relationship between TOC and DBP formation.
- If DBP concentrations ultimately exceed the Stage 2 DBP requirements, the City may need to alter its disinfection process to reduce DBP formation. Options include conversion to chloramines as a residual disinfectant for the distribution system, and/or use of a stronger disinfectant such as ultraviolet light, ozone or chlorine dioxide.

- The Long-Term Enhanced Surface Water Treatment Rule (LT2ESWTR) will require two years of monitoring for *Cryptosporidium* and *Giardia* in the plant's raw water. If excessive concentrations of *Cryptosporidium* are detected, then the City may need to install a stronger and more-expensive disinfectant compared to chlorine, such as ultraviolet light, ozone or chlorine dioxide. The City began this monitoring in September 2003 and initial results indicate low levels of these pathogens, which would not require a change in the plant's disinfection scheme if these results continue for the next 18 months of sampling.
- The existing solids lagoon is currently full and needs to be cleaned. Potential short-circuiting through the lagoon is threatening the release of solids and/or chlorine into Skunk Creek, which would be in violation of the current NPDES permit. To ensure continued compliance, immediate removal of some or all of the accumulated solids is required. In addition to this immediate cleaning requirement, a long-term strategy for solids handling needs to be developed. The type of solids handling process appropriate for consideration depends largely on the methods available for disposal. The City should then make improvements to its solids handling system to accommodate the selected disposal option.
- Recent environmental regulations have been promulgated to protect threatened and endangered (T&E) species including several anadromous fish (salmon and steelhead) which populate the Rogue River. These new rules include specific requirements for river intakes and diversions to avoid the potential "take" of these species, especially juvenile fish. The City's existing intake does not meet specific requirements for screen type, approach velocity and sweeping velocity. Significant improvements are required to bring the intake into compliance. The City should consider making improvements to allow withdrawal of 30 mgd to support the ultimate WTP site capacity.

6.1.4 Support Facilities

- The intake, basins, filters, clearwell and plant buildings have many years of remaining useful structural life, but some of the structures are over 70 years old. These facilities should be reviewed with respect to their vulnerability to damage

during a severe earthquake. There have been several earthquakes in the Pacific Northwest over the past 10 years which could have severely damaged the plant if the event occurred closer to Grants Pass. A detailed seismic evaluation of the plant is recommended to determine improvements necessary to ensure that it can reliably produce water for the remaining useful life.

- The existing raw water, high service and backwash pumps, although 20 years old, appear to be functioning appropriately and should have significant remaining useful life. These pumps require routine inspections and maintenance.
- The plant electrical and I&C components are performing well and have significant remaining useful life. The plant's I&C/SCADA system was recently upgraded to replace older and outdated systems. The plant's primary electrical service will need to be upgraded if major new electromechanical facilities are constructed at the existing WTP site.
- The plant has never been out of service for an extended period of time due to unplanned power outages. The City should consider installation of an on-site emergency power generation system, to allow the plant to produce 3 to 5 mgd, if it feels vulnerable to severe power outages. Alternatively, emergency power could be provided from another grid if available.
- As discussed previously, the City should implement a plan to keep the plant in service if the existing backwash pump fails.
- The existing pneumatic controls for all filter valves and backwash valves are old and have little remaining useful life. Pneumatic control technology is being replaced with electric/electronic controls throughout the industry and replacement/repair parts are becoming more difficult to obtain. Replacement of all pneumatic control valves with electric-actuated valves is recommended. Due to the age of the valves (some leak now) and the relative low cost of the valves versus the electric actuators, the valves should all be replaced at the same time. During this replacement project, old and structurally-inadequate filter and backwash piping should be replaced.
- All filter flowmeters should be replaced with non-contact technology (magnetic or ultrasonic) due to reliability, age and potential fouling problems. The filter flowmeters should also be relocated to allow measurement of filter-to-waste flows.

- The City has decided that the existing plant, due to its age, use and location, does not need to comply with current ADA access requirements, both for potential employees and for the general public.
- There are a few locations within the plant that may not meet current employee protection against falls and accidents (OSHA standards). Various stairs, steps, ladders and handrails should be improved/modified to meet current codes.

6.1.5 Monitoring and Control

- The current I&C/SCADA system was upgraded recently and provides a good level of monitoring and control, including the ability to monitor and control the plant remotely. Various upgrades to the control system, including hardware and software, will be required to integrate any improvements made to the plant.
- The City should consider adding particle counters for each filter, even though not a regulatory/monitoring requirement, to further optimize plant and filter performance.
- The City should routinely monitor the total organic carbon (TOC) in its raw and filtered water. The City should measure UV absorbance at 254 nanometers (UV_{254}) as a surrogate for TOC measurements.

6.1.6 Integration of Vulnerability Assessment Recommendations

- The City has recently completed a Vulnerability Assessment (VA) of its water system per EPA requirements. It was decided to keep the recommendations of the VA Study separate from this WTPFP document. There may be some capital improvements recommended from the VA Study which could be integrated with recommended plant improvements from this Plan.

6.2 Alternative Analysis For Critical Process Issues

Based on the summary of recommendations presented above, the project team selected four potential improvements for more-detailed analysis, to provide better definition and to assist in prioritizing these improvements. These potential improvements were determined to have the highest priority requiring implementation over the next few years:

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- Filter Modifications
- Basin Modifications
- Solids Handling and Disposal
- Intake Modifications

Each of these topics is reviewed and discussed in the following sub-sections and a recommended course of action is presented.

6.2.1 Filter Modifications

Filter production efficiencies are typically between 80 and 90 percent throughout the year, and generally decrease in the winter when total production is lower and the water is colder and more turbid. Poor efficiencies contribute to increased operational costs, including longer operation time, increased pumping and chemical costs, and increased sludge production. The minimum desired filter production efficiency is 97 percent. Based on our analysis, low plant efficiencies result from short filter runs at relatively high filtration rates through a shallow, dirty media. To increase overall efficiency, the existing filter media should be replaced with a new design, maximizing the overall media depth.

There are three options to improve the filters and increase plant production efficiency including: 1) replace the existing media while keeping the existing underdrains, 2) install new underdrains to allow for a deeper media, and 3) replace the conventional media filters with membrane filtration. The potential benefits/drawbacks associated with each alternative are discussed below. A summary of capital costs is presented at the end of this sub-section.

6.2.1.1 Membrane Filtration

Membrane filtration has become an increasingly popular filtration alternative. As the technology comes of age, the costs for new construction are increasingly competitive with conventional filtration. However, the costs associated with converting existing media filters to membrane filtration, especially if no capacity expansion is desired, are still significantly higher than for other alternatives. During membrane filtration, suspended particles are rejected from the influent as the water flows through the pores of

the membrane. The pore size of the membrane determines which particles are rejected. For application at the Grants Pass WTP, microfiltration (possibly in conjunction with pre-chlorination and coagulation) would be recommended. These filters would provide an absolute barrier to *Giardia* and *Cryptosporidium*, ensuring continued compliance with future regulations.

There are several "submerged" microfiltration systems on the market today which may be appropriate for the Grants Pass WTP, including those systems manufactured by Zenon Environmental Inc. and USFilter/Memcor. The plant's existing filters or basins can be retrofitted to accommodate the "submerged" technology, better matching the plant's existing HGL and minimizing additional pumping requirements. These systems normally require minimal chemical addition for treatment and provide high quality drinking water and operational simplicity within a relatively small footprint. However, membranes do require periodic chemical cleaning.

A pilot study to determine the design constraints for full-scale performance would be required if the City decides to implement this technology. Significant engineering would be required to successfully integrate membrane technology into the existing plant's treatment process, as well as identify a site for all the ancillary equipment. As previously mentioned, these proprietary technologies generally require large capital investments and costly periodic membrane replacements. These additional costs make this alternative less attractive compared to other alternatives. A planning-level capital cost estimate for this alternative is presented in ***Table 6-2***.

6.2.1.2 Replace Existing Media and Gravel

The least expensive filter improvement alternative is to simply re-build the filter media and gravel while leaving the existing underdrains intact. This alternative limits the available depth of media to approximately 20 to 24 inches. If the top of media is any closer to bottoms of troughs, the plant will continuously lose media via "carry-over" during backwash as has occurred at the plant. Shallow media limits the filter run lengths and ultimately reduces plant efficiencies. There may be room to raise the troughs to

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allow for a deeper media, however, limitations on the media depth may still exist. A planning-level capital cost estimate for this alternative is presented in ***Table 6-2***. Costs associated with this alternative include improvements to the surface wash system, discussed later in this section.

6.2.1.3 Re-build Filter Media and Underdrains

The existing depth from the filter floor to the bottom of the filter media is approximately 2.07 feet (24.84-inches) including 11.84-inches of underdrain and grout and 13-inches of support gravel. It is possible to gain additional filter media depth in the existing filters by replacing the existing underdrain and support gravel with a gravel-less underdrain system. Profiles for these gravel-less underdrains range from as low as 6 inches to as high as 14 inches. This section describes the potential underdrain options for the plant.

The advent of gravel-less underdrains has allowed retrofits inside existing filter cells to deepen media. Essentially, the space previously used for gravel layers to support the filter media and to promote even backwash flow distribution can now be used for more filter media. Also, gravel-less underdrains eliminate the operational problems often encountered by migration and mounding of gravel, which can quickly upset a filter and require complete re-building. Basically, there are 3 types of gravel-less underdrains for consideration by the City including 1) false floor with plenum, 2) slotted screens, and 3) plastic blocks.

Plenum under false floor with nozzles. These types of systems have been successfully used for many years and are made by Infilco Degremont (IDI), General Filter (GF), Patterson Candy (PCI) and others. A false floor with proper structural design characteristics must be constructed above the filter floor to create the plenum where water and air can uniformly enter and leave the filters. Specially-designed nozzles are installed through the false floor to allow proper collection of filtered water as well as proper distribution of air and water during backwash, and to keep media from entering the plenum.

The height of the false floor off the filter bottom will determine the overall filter box configuration when designed with a specific filter media configuration. A minimum plenum depth of two feet is normally recommended, but up to three feet is often provided if access to the plenum is desired. Plenum depths less than two feet are possible, but this must be carefully evaluated on a case-by-case basis to ensure proper air and water distribution characteristics; less depth is required if air scour backwashing is not used. The manufacturers of these systems need to be consulted to determine the lowest-possible plenum depth. These systems offer the highest profile of the underdrain system alternatives, so they offer the lowest potential to maximize filter media depth in a retrofit situation.

The nozzle design and nozzle spacing must also be determined to meet the needs of the specific installation. The nozzle slit width must ensure controlled air and water distribution as well as retain the smallest media size. Nozzle materials must be carefully selected to avoid erosion of the slits over time, which can be caused by high water velocities during backwash. Some nozzle systems are designed with a shallow gravel layer over and around the nozzles to minimize slit erosion problems. The nozzle heights are adjustable, but each must be located within close tolerances to ensure uniform flow distribution during backwash. When used without a deep layer of gravel support under the filter media, there is some concern that the media between the nozzles can be cleaned adequately.

Low-profile laterals constructed of stainless steel or plastic. These types of systems have been in limited use for only the past five to ten years, and entered the marketplace as an alternative gravel-less underdrain for retrofit applications. EIMCO, AWI-Anthratech and CPC all market similar products, but the stainless steel products are typically of most interest due to their durability compared to plastic. These types of underdrains are generally reserved for small package plants and there are few larger installations in the U.S. to gather operating data and opinions of performance from operators.

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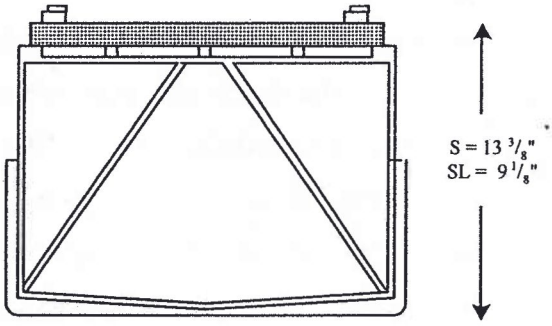
These systems offer the lowest profile of any underdrain system so they offer the greatest potential to maximize filter media depth in a retrofit situation. Air and water pass through specially designed slits in the underdrain and must be carefully designed to ensure even flow distribution along its length. Unlike the other underdrain systems, air usually enters from the top and the air piping must be installed inside the filter box and penetrate down through the media. MWH has concerns about these types of systems for two major reasons:

- The uniform distribution of air and water or water alone during backwash is suspect based on observations made at operating facilities. The longer the laterals are, the more concern about this problem.
- The durability of the materials during installation is a concern. It is possible for untrained workers to damage the laterals, or slightly displace the slits, by walking on them or kicking them, such that the integrity of the system as well as the backwashing performance is jeopardized.

Plastic block with porous plate cap. The plastic Universal Type S Underdrain system, made by Leopold, has been successfully used in many installations for years. Leopold then created its IMS Cap for use with the Universal Underdrain to eliminate the need for gravel. The IMS Cap is a porous plastic plate attached directly to the block. The IMS Cap system has been successfully used at a number of plants for many years also. Several years ago, Leopold introduced its Type SL system, which has a lower profile (4 inches lower) than the Type S system. The Type SL system should not be used for lateral lengths greater than 20 feet due to flow distribution concerns, and therefore is acceptable for the Grants Pass WTP (15 to 18-foot laterals).



Profile of Leopold Type S and SL Underdrains

Leopold had the patent on this type of system until a few years ago and now there are at least three other plastic-block underdrain manufacturers besides Leopold including TETRA, Roberts Filter and US Filter. MWH has less experience with these manufacturers than with Leopold, but they all have a number of representative installations. The major difference is in the width and length and cap type of the various products. Roberts Filter underdrains are made of PVC, while the others are made of HDPE.

MWH has designed many new filters, as well as many filter modifications, using this type of system. It offers a lower profile than the plenum/nozzle system, but not as low as the screened laterals described above. Designed and installed properly, plastic block underdrains offer a good choice for a gravel-less underdrain system for use with or without air as demonstrated in several recent Oregon installations (City of Newberg, McMinnville Water and Light, Joint Water Commission, City of Lake Oswego, City of Wilsonville and South Fork Water Board WTPs).

Underdrain Recommendation. To extend the life of the filters and to maximize the new filter media depth, the most reliable and shallowest underdrains available at a reasonable price should be selected. We feel these criteria are best achieved by the low profile plastic block underdrains with gravel-less caps represented by numerous manufacturers. The plastic-block type underdrains are more commonly installed in filter retrofits than the low profile laterals and are less expensive to purchase and install. Planning-level capital cost estimates for this alternative are presented in ***Table 6-2***. Costs associated with this alternative include improvements to the surface wash system, discussed in the following sub-section.

6.2.1.4 Surface Wash System Improvement Alternatives

If conventional media filters remain at the WTP, an auxiliary filter media cleaning system is necessary for effective cleaning of the filter media. Air-scour and surface water wash are the most common media cleaning methods. Air-scour has become popular during the past 10 to 15 years, as deeper filter media have become more common. As a result, older

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filters have rotating surface wash systems; most new filters have air-scour. Some filters have been designed with both systems for redundancy and superior cleaning. However, incorporation of air-scour at the Grants Pass WTP will require significant financial investment. In addition, even with the installation of gravel-less underdrains, the filter media will not be deep enough to warrant installation of air-scour. A properly installed and maintained surface wash system will provide enough agitation during backwash to sufficiently clean the media. Therefore, only surface wash systems are recommended for further consideration.

Based on the age and condition of the existing surface wash systems, particularly in the older filters (Filters 1 through 3), it is recommended that the piping, supports and surface wash arms inside the filters be replaced with new equipment. Since this equipment must be removed to rehabilitate the filters, there is not a significant economic incentive to salvage any of the equipment. Costs for these improvements have been included in both the media/gravel replacement, and the filter media and underdrain re-build alternatives.

There are two primary types of surface wash systems, fixed grid and rotating arm. The existing filters use a rotating arm system with straight arms. ***Table 6-1*** compares the pros and cons of the two types of surface wash systems. Although rotating arm systems require more maintenance, they generally provide as effective cleaning action with lower water requirements and less obstruction for filter access. But, they can not provide deep penetration to allow adequate cleaning of deeper media.

Incorporation of a fixed-grid system would require significant improvements to the current surface wash piping system, including a larger transmission pipe from the high surface pump station discharge header, installation of a surface wash grid in each existing filter and additional flow/pressure control devices. Further, it may be difficult to simultaneously keep in service the existing rotating arm system and a new fixed grid system as the filters are individually reconstructed. Therefore, we preliminarily recommend that a rotating arm system continue to be used at the plant. Further review of the preferred surface wash system should be performed during detailed design.

TABLE 6-1: COMPARISON OF ROTATING ARM AND FIXED GRID SURFACE WASH SYSTEMS

Type	Advantages	Disadvantages
Rotating Arm	• Fewer components • Proven effectiveness if system is properly designed and maintained • Lower flow needed (0.5 to 0.7 gpm/sf) • Plant operators familiar with this system • Consistent with existing system which eliminates the need to replace the pump and piping in the filter gallery	• Only 1 or 2 reliable suppliers • Loses effectiveness if bed depth is reduced with 15° nozzle angle • Less effective at cleaning deeper media • More susceptible to clogging with the shallow nozzle angle. • 70 to 100 psi needed to drive arms • Requires greater maintenance
Fixed Grid	• No moving parts • Proven technology with over 50 years of US experience • Needs only 10 psi pressure • Effective even if bed depth reduced from media loss due to angle of jets (25° – 35°) • Can be fabricated by any shop • May be more effective in reaching corners and along walls • Lower maintenance requirements	• Higher flow needed (3 gpm/sf) requiring replacement of all existing piping and pump. • Proper design is essential to performance • Might be more expensive to install • Creates more dirty washwater to dispose of

The rotating arms can be either straight or S-shaped. The S-shaped arm was developed to more effectively reach the corner area during backwash. However, in most cases there is sufficient lateral mixing of the media during backwash to provide effective cleaning with the straight arm system. Since the cost between the two types of arms is not significant, we recommended using the S-shaped rotating arms. The surface wash arms should be located approximately 2 inches above the media surface and the top of the media elevation should be consistently maintained to ensure effective cleaning.

6.2.1.5 Filter Modifications Summary and Recommendations

Planning-level capital cost estimates for the three filter modification alternatives are presented in ***Table 6-2***.

TABLE 6-2: COMPARATIVE PLANNING-LEVEL CAPITAL COST ESTIMATE FOR FILTER MODIFICATION ALTERNATIVES

Alternative	Capital Cost ($)
Option 1: Replace Conventional Media Filters with Membrane Filtration	$11,500,000
Option 2: Replace Existing Media and Gravel Support	$350,000
Option 3: Re-build Filter Media and Underdrains	$600,000

Based on capital cost and overall "value" added to the plant, we recommend re-building all filters with plastic block underdrains and installing a deeper dual-media (20-inches of 1.0 mm anthracite over 10-inches of 0.5 mm sand). ***Figure 6-1*** presents a cross-section of a representative existing filter (Filters 6, 7 and 8) and the recommended filter modification alternative.

Although there are some advantages of rebuilding the filters "bank-by-bank" during individual projects, including optimization of the design based on previous experience, rebuilding all of the filters as part of one construction project will minimize the overall cost of the project and ensure uniformity and consistency throughout construction. Assuming 3-weeks on average for each filter re-build, the construction project will last a total of 24-weeks, or approximately 6-months total. Construction is limited to the "off-peak" season (October through April) due to demand constraints. If construction were started in October, the project could be completed by the end of March, before water demands begin to increase. To meet this schedule, the City would need to issue Notice to Proceed (NTP) to contractor in Spring/early Summer to ensure materials are on-site by early October. Therefore, it is feasible to re-build all of the filters in one year under one construction contract. This would result in a savings of approximately 25% of total costs and effort, when compared to the "bank-by-bank" separate project approach.

The City should also consider incorporating all suggested filter improvements, including valve/actuator replacements discussed later in this Section, as part of one construction effort for economies of scale and for ease of sequencing filter outages during

construction. Combining these projects would add approximately 1-week per filter to the construction schedule, or a total of 32-weeks (8-months) construction duration.

6.2.2 Basin Modifications

As discussed in Section 4, the existing 3 basins have deficiencies with respect to providing optimal pretreatment ahead of the filters. During challenging water quality events (high turbidities, cold water, high alum doses), the settled water turbidity exiting the basins is significantly higher than desired, thereby loading additional solids to the filters, reducing production efficiencies and increasing the risk of poor filtered water quality. Also, Basin #3 suffers from poorer performance than the other 2 basins, due to its square shape, center-feed and peripheral launders, which results in a higher degree of short-circuiting.

At a minimum, flocculation should be added to each basin for faster forming and better settling floc. Currently, none of the basins provide any degree of controlled mixing to enhance floc formation. Flocculation options include mechanical (vertical turbine or horizontal paddle wheels) and hydraulic flocculation using baffles. Basin #3 requires other baffling improvements to minimize flow short-circuiting in addition to adding flocculation. These improvements will optimize plant performance, reduce chemical consumption and improved filtered water quality.

The addition of flocculation to each basin as an immediate improvement should be developed with a plan for the future plant capacity increase. The plant's pre-filtration (flocculation/sedimentation) capacity can be expanded to 30 mgd in a number of different ways for a wide range of costs including:

- Add a 4th basin, rated at 10 mgd +/-, to operate in parallel with the other 3 basins rated at 20 mgd
- Uprate the capacity of the three existing basins to 25 mgd and add a 4th basin rated at 5 mgd +/-

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- Uprate the capacity of the three existing basins to 30 mgd and don't add any new basins
- Use of high-rate proprietary clarification systems, such as Actiflo, SuperPulsators or dissolved air flotation (DAF), to increase capacity within the existing plant's footprint

From a long-term planning perspective, any of these approaches appears to be technically feasible for a range of costs. With respect to decision-making for immediate basin improvements to add flocculation, it is suggested to assume the entire 30 mgd pretreatment capacity will remain inside the existing basin footprint. This will allow the greatest degree of flexibility for future plant expansions that may not occur for another 20 to 25 years. Based on our experience, it is likely that the lowest-cost approach for the plant expansion will also be to uprate the basins to 30 mgd.

A preliminary review of hydraulic capacity and basin configurations suggests the following approach for expanding the basins to 30 mgd:

- Uprate the flow to Basins 1 and 2 to 15 mgd from the current 12 mgd capacity
- Uprate the flow to Basin 3 to 15 mgd from the current 8 mgd capacity

This uprating to 30 mgd would incorporate the use of flocculation, baffling and high-rate tube settlers in all basins. The entrance to Basin 3 would also be changed to the south end (from the existing centerfeed) to promote longitudinal flow. New launders would be required for the basins in conjunction with the tube settlers. With the addition of tube settlers, all basins would require the addition of continuous sludge removal systems. Lamella plate settlers are also an option versus tube settlers, but they typically require a deeper setting than tubes and therefore may conflict with sludge removal systems.

Proposed design criteria for the basins at 30 mgd are shown in ***Table 6-3***.

Based on this analysis, it is suggested to provide flocculation facilities for immediate basin improvements that allow approximately 20 minutes of flocculation time under the

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future 30 mgd capacity scenario. Longer flocculation times would therefore be provided under today's lower flowrates in each basin, which is acceptable.

TABLE 6-3: PROPOSED BASIN DESIGN CRITERIA AT 30 MGD

Parameter	Basin 1	Basin 2	Basin 3
Width x Length (ft)	61 x 98	38 x 98	80 x 80
Avg. Water Depth (ft)	13	13	13
Surface Area, total (sf)	5,980	3,750	6,400
Total Volume (gal)	581,600	364,700	622,400
Nominal Rated Capacity (mgd)	9.5	5.5	15.0
Flocculation Time (min)	20	20	20
Flocculation Volume (cf)	17,500	10,000	27,500
Flocculation Surface Area (sf)	1,350	770	2,120
Flocculation Length (ft)	22	20	26.5
Tube Settler Area (sf)	2,600	1,500	4,200
Length:Width Ratio	1.6:1	2.6:1	2:1
Length:Depth Ratio	1:7.5	1:7.5	1:6.2
Mean Flow Velocity (ft/min)	1.0	1.0	1.2
Overflow Rate at Nominal Capacity (gpm/sf)	1.10	1.02	1.63
Theoretical Total Detention Time at Nominal Rated Capacity (min)	85	90	60

As mentioned previously, flocculation options include mechanical (vertical turbine or horizontal paddle wheels) and hydraulic flocculation using baffles. The use of hydraulic flocculation requires additional headloss, in the range of 9-inches to 24-inches, which may be feasible to consider for 20 mgd, but the higher future flows in each basin might make this a difficult approach. For planning purposes, it is recommended to add mechanical flocculators to each basin with a minimum of two stages, with each stage separated by a baffle wall. Vertical turbine flocculators probably represent a lower cost solution for this retrofit application compared to horizontal flocculators, so this approach is suggested for planning purposes. Detailed comparison of flocculation alternatives should be conducted during preliminary design.

Basin #3 requires other improvements to minimize flow short-circuiting in addition to adding flocculation. The existing center-feed and peripheral launder system will be removed. The raw water pipe will be re-routed to enter the southern part of the basin. New effluent launders will be added to the northern part of the basin. Two divider walls will be installed to create three separate "sub-basins" to improve flow and reduce short-circuiting. Electrical and control improvements will also be required for a complete mechanical flocculation system.

Figure 6-2 indicates the conceptual improvements to allow the basins to treat 30 mgd in the future. The estimated capital cost to add the flocculation systems, baffle walls and other Basin 3 modifications is $600,000. This cost does not include the addition of continuous sludge removal systems, which are included as lower-priority improvement not necessarily required for the immediate improvements, nor does it include addition of tube settlers, which would not be required until the plant capacity is expanded.

These improvements should be constructed during the non-peak demand season, one basin at a time, to keep the plant in service. It is estimated that each basin will require approximately 1 month to modify, on average, for a total on-site construction period of 3 months. The total construction contract duration will be approximately 12 months to allow for submittals, approvals and delivery time for long-lead equipment. Timing of improvements to Basin #3 should be carefully determined when plant production is at its lowest, since it has the highest hydraulic capacity of any of the basins. The City may want to integrate the basin improvements project with the filter rehabilitation project to complete these process upgrades at the same time, in order to reduce total costs and minimize plant disruptions.

6.2.3 Solids Handling and Disposal

A detailed review of solids handling issues and current solids production at the plant is presented in **Section 4**. As previously stated, the existing lagoon at the Mill Pond site is currently full and needs to be cleaned immediately; potential short-circuiting through the lagoon is threatening the release of solids and/or chlorine into Skunk Creek, which would

be in violation of the current NPDES permit. The lagoon was cleaned in 2000 and it has now re-filled. At least a portion, if not all, of the liquid (non-dried) sludge from existing pond needs to be removed and hauled off-site immediately. Since the sludge is less than 15% solids, disposal at a landfill is not an option unless the solids are dewatered first. An alternative site for solids disposal, to accept lower solids concentrations, may need to be identified in the near-term if dewatering is not implemented. In addition to the need for immediate cleaning requirements, a long-term strategy for solids handling should be developed. This long-term strategy should be developed to account for future capacity increases at the plant.

Selection of the appropriate solids handling process depends largely on the methods available for disposal. A brief review of disposal methods for the City is presented below, followed by a discussion of alternatives to meet immediate and long-term solids handling needs at the plant.

6.2.3.1 Method of Disposal

Ultimately, the long-term solids handling strategy will depend on the available methods of disposal. The four disposal options available to the City are:

Option A: Delivery of solids to the Water Restoration Plant (WRP)
Option B: Landfill disposal of dewatered solids
Option C: Dispose of liquid sludge at the Redwood Pump Station site
Option D: Delivery of dewatered solids directly to the City's JO-GRO™ facility

Option A. The City's WRP is approximately one mile west of the WTP. Assuming that the WRP has sufficient solids and hydraulic capacity, and that the inert WTP solids do not negatively affect the WRP solids processes, disposal of the WTP solids to the sanitary sewer is the simplest option for the City since solids dewatering would only occur at one location (at the WRP) versus separate dewatering facilities at each plant. It is understood that the WRP solids are used for composting at the JO-GRO™ facility.

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Waste washwater and basin solids can be equalized and pumped directly to the WRP, or alternatively, thickened to approximately 2 to 4 percent solids to deliver a lower volume (but the same amount of solids) to the WRP. Thickening and equalization substantially reduces the pumping and piping capacity needed to divert solids and reduces the hydraulic load to the WRP. Another WRP disposal approach is to deliver liquid solids to the WRP via tanker trucks, which would require removal of solids from the lagoon on a frequent basis; this approach would eliminate the need to install piping to the WRP. Except for the trucking option, the existing lagoon would no longer be used for WTP solids storage.

The existing sewer line located along "M" Street is 12-inch diameter, and is believed to lack the hydraulic capacity to carry additional flows from the WTP. Additionally, the line is located beneath several buildings along the Rogue River, and is relatively old. The City feels the potential for solids accumulation in this pipeline, coupled with the lack of accessibility, create too great a risk to consider this pipeline for WTP solids discharge. Therefore, a new, dedicated forcemain is presumed to be required between the WTP and the WRP to adequately deliver the solids to the WRP. The size of the pipeline (and pumps) depends on whether all backwash and basin solids and liquids are delivered to the WTP (higher flows) or thickened solids are delivered to the WRP (lower flows). The existing transfer pumps in the WTP's equalization basin may be able to deliver the higher flow alternative. If the City is seriously interested in a WRP disposal option, then it should further explore the possible use of the existing 12-inch sewer main for disposal of thickened solids, to reduce capital costs.

Option B. If sludge is hauled to a landfill, the sludge must be thickened and dewatered to a minimum of 15 to 25-percent solids depending on individual landfill requirements. Either a mechanical dewatering process (such as a belt filter press or centrifuge), or gravity dewatering process (such as lagoons, drying beds or Geo-Tubes) could be used. Mechanical dewatering systems are typically only used for very large plants, or plants with significant space constraints. Mechanical dewatering systems can be labor-and power intensive and can only reliably produce 15 to 25 percent solids maximum.

Lagoons and drying beds require less labor to operate, and if designed and operated properly under adequate climatological conditions, can produce greater than 30 percent solids. Another possible dewatering approach which has gained the City's interest is the use of "GeoTubes" which are geotextile products that can be filled with liquid sludge, and then allowed to slowly drain until the solid content has risen for proper handling and disposal. If acceptable to the City, these tubes could be filled and left around the perimeter of the lagoon for long periods of time until properly dewatered.

Option C. The City has recently identified the Redwood Pump Station Site as a potential alternative for solids disposal. The site is relatively large (approximately 40 acres), secluded, and located approximately 8-miles from the WTP. Liquid sludge could be trucked to the Site, and solids holding/dewatering facilities (such as drying beds or lagoons) could be constructed on-site for dewatering; dry solids could potentially be land applied on-site for ultimate disposal.

In recent discussions with the Oregon Department of Environmental Quality (DEQ), the City learned that transfer of solids to this site would not likely fall under the Solids Waste Agency purview, and therefore would not require a solids permit. However, there may be public perception problems or challenges to use of this Site. This alternative site for solids disposal was not seriously considered for this analysis.

Option D. The City has the ability to haul dewatered WTP solids directly to the City's JO-GRO™ facility, which currently accepts dewatered solids from the WRP for use in developing soil amendment products. With proper conditioning and control of mix ratios, it is believed that dewatered alum sludge can be used in a similar manner as the WRP solids. In this case, the City would have to produce dewatered sludge (> 15% solids) for hauling to the facility using one of the techniques mentioned in Option A. The operating costs of this Option would be considerably less than Option A, since there would be no "tipping" or disposal cost incurred.

6.2.3.2 *Alternatives to Address Immediate Solids Handling Needs*

The existing WTP sludge lagoon is full and the City needs to implement a short-term solution to handle its solids until a long-term solution is implemented. The short-term solution requires continued use of the lagoon to store solids, but the lagoon needs to be emptied of solids to allow more solids storage over the next few years. The City has two alternatives to removing solids from the lagoon, including:

- Dredge/remove solids from the lagoon and truck the liquid solids to a site that can accept the liquid solids (either the WRP or a site which may be available to store and dry solids such as the Redwood Pump Station Site), or
- Dredge/remove solids from the lagoon, dewater the liquid solids (either on-site or remotely), and then dispose of the dewatered solids in a landfill or at the City's JO-GRO™ facility.

The existing lagoon is approximately 1.5 acres (65,340 sf), and the lagoon depth is approximately 4-feet average, which is equivalent to 260,000 cubic feet or 2 million gallons of total stored solids. These solids are estimated to be approximately 4-percent by weight on average. The solids in the lower portion of the lagoon may have significantly higher solids content. For discussion purposes, this volume of solids currently stored in the lagoon represents almost 400 tanker truck loads carrying 5,000 gallons each.

For both short-term options, the City may opt to haul and dispose of a minimum amount of solids as soon as possible, and then plan to remove solids annually or semi-annually over the next few years. The costs associated with each option will depend on the total volume removed. As the solids level in the lagoon continue to rise, the "immediate" decision regarding method of disposal may ultimately be based on availability of resources to perform the desired removal and disposal services.

6.2.3.3 Alternatives to Address Long-term Solids Handling Needs

The City has several alternatives to meet long-term solids handling and disposal needs for the WTP, including:

Option 1: Create new sludge drying beds/lagoons at the existing Mill Pond site; dispose of dewatered solids (25-40% solids) in landfill or at JO-GRO™.

Option 2: Construct new mechanical dewatering facility at the Mill Pond site; dispose of the solids (20-25% solids) in a landfill or at JO-GRO™.

Option 3: Equalize waste washwater and basin solids in the existing equalization basin, and pump all of the liquid + solid flow (~0.1% solids average) to the WRP through a new dedicated force main.

Option 4: Equalize waste washwater and basin solids in the existing equalization basin, construct new thickening/clarification facility, and pump the thickened solids (~2% solids) through a new dedicated force main to the WRP.

Option 5: Use existing solids lagoon for storage as currently practiced; install permanent dredging equipment in the lagoon and frequently haul solids (~4% solids) via truck to the WRP, Redwood Pump Station Site or to the JO-GRO™ site.

Option 6: Use existing solids lagoon for storage as currently practiced; install permanent dredging equipment in the lagoon and use Geo-Tubes to dewater the solids removed from the lagoon; dispose of the solids (15-40% solids) in a landfill or at JO-GRO™.

These six options were developed for comparison and evaluation purposes. There may be other variations of these options which could also be considered, but these six represent a wide range for the purposes of this planning effort. These options are discussed in detail in the following sub-sections. Planning-level capital and operations and maintenance cost estimates for each option are presented in ***Table 6-4***.

OPTION 1. The creation of new solids drying beds/lagoons on the site of the existing Mill Pond lagoon, to replace the lagoon, is considered a viable alternative that can meet the City's needs for the next 20 years or more. The site is capable of handling solids production up to a maximum day demand of 20 mgd, at current alum dosages; efforts to optimize coagulation, reduce alum doses and minimize solids production may support solids storage and dewatering beyond the 20 mgd maximum day demand.

The lagoons would receive washwater and solids from the existing equalization basin and the clarified overflow would continue to be discharged to Skunk Creek. Once a certain amount of solids have filled the lagoon, it would be taken off line, slowly decanted (decant to Skunk Creek) and the solids allowed to dry. The dried solids would be removed via a front-end loader and hauled via dump truck to a landfill or to the JO-GRO™ facility.

The new drying beds/lagoons would require sequential construction to keep part of the existing lagoon in service while at least one or 2 beds are completed. Removal of existing solids in the lagoons would be required as part of construction.

The new sludge drying beds would consist of 4 isolated cells, each with a capacity to handle 4 months of sludge production. The operating philosophy would allow two lagoons drying, one available for service and one in service. Maximizing the number of cells increases the flexibility and dewatering conditions considering the limited drying season. The design criteria for each cell is presented below:

- Cell Dimensions: 55' x 255' x 6' (each)
- Decantation facility: telescoping valve(s)
- 10-foot access roadway surrounding each of the cells
- Cement gunite or soil-cement lining

The cells would be capable of dewatering the solids to at least 25%, perhaps as high as 40%, depending on drying conditions. Higher solids content would result in lower removal and disposal costs.

OPTION 2. Mechanical dewatering is a relatively expensive alternative, typically reserved for larger plants, or plants with space constraints. For application in Grants Pass, the process would require a clarifier/thickener (which may also serve as sludge equalization) prior to the dewatering process. There are several mechanical dewatering processes available including diaphragm filter press, conventional filter press, belt filter press and centrifuge. Based on past experience with alum sludge, centrifuges are recommended for further consideration.

Washwater and basin solids would flow from the existing equalization basin to the thickener to create approximately 2% solids. Overflow/supernatant from the thickener would be discharged to Skunk Creek under the existing NPDES permit, if acceptable. Centrifuges typically operate according to a counter-current flow principal; a "scroll" forces dewatered solids to one end of the mechanism, where they are stored and eventually discharged into a truck for transport to a landfill or to JO-GRO™. Liquid centrate from the centrifuge would be combined with that of the thickener and discharged to Skunk Creek. To achieve "optimal" solids concentration, relatively high concentrations of polymer must be added to the sludge, thereby increasing costs associated with operations and maintenance.

The centrifuge would be located inside a two-story building which would allow gravity flow of dewatered solids from the centrifuge into a dump truck below. Alternatively, a single-story building could be used with a conveyor system to deliver solids to the truck.

OPTION 3. Discharging all of the washwater and basin solids to the WRP would require the installation of a new 12-inch dedicated force main, sized for approximately 2,000 gpm instantaneous flow. For this analysis, it was assumed that the existing equalization basin and transfer pumps are sufficiently sized to pump the liquid/solids to

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the WRP (approximately 1 mile). Operations and maintenance costs for this alternative include WRP charges for the discharged liquid plus solids, in addition to pumping costs (which are presumably about the same as the current pumping costs to deliver the washwater to the lagoon across the street).

OPTION 4. The addition of a thickener or clarifier at the WTP site would significantly decrease the volume of liquid/solids discharged to the WRP. A thickener/clarifier that increases the solids concentration to 2% solids would reduce the overall volume discharged to the WRP by a factor of 20 or more. The discharge fee to the WRP would presumably be less to handle less volume compared to Option 3 although the total solids delivered to the WRP would be the same. A new 4-inch forcemain to the WRP would be required, but smaller and less costly than for Option 3. Overflow/supernatant from the thickener/clarifier would be discharged to Skunk Creek under the existing NPDES permit. Operations and maintenance costs for this alternative include WRP charges for the discharged liquid plus solids, in addition to pumping costs.

OPTION 5. This alternative requires no immediate improvements to the existing lagoon, however, a capital investment associated with the installation of permanent dredging equipment at the pond is required. For this analysis, it was assumed that the existing lagoon is capable of creating solids up to 4% by weight on average. Supernatant would continue to be discharged to Skunk Creek. Solids from the lagoon would be periodically pumped by the dredge into a tanker truck (perhaps on a weekly or monthly basis) and hauled to the WRP site in a tanker truck for disposal. The liquid solids could alternatively be hauled to the Redwood Pump Station Site or to the JO-GRO™ site for dewatering and disposal. Operational and maintenance charges associated with this alternative include WRP charges if this approach is used (less than Option 4 because the total volume is less), in addition to those associated with operating and maintaining the dredge and trucking the liquid plus solids to the WRP on a frequent basis.

OPTION 6. This alternative requires no immediate improvements to the existing lagoon, however, a capital investment associated with the installation of permanent dredging

equipment at the pond is required. For this analysis, it was assumed that the existing lagoon is capable of creating solids up to 4% by weight on average. Supernatant would continue to be discharged to Skunk Creek. Solids from the lagoon would be periodically pumped by the dredge into Geo-Tubes located around the pond perimeter and allowed to dewater through the Geo-Tube fabric by gravity. Conditioning polymer would be added to assist with dewatering. Once full and at the proper solids concentration, the Geo-Tube would be hauled to the landfill or to JO-GRO™ where the dewatered solids would be released from the tube. In addition to the costs associated with operating and maintaining the dredge and trucking the solids to the on an annual or semi-annual basis, depending on how long the tubes take to dewater the solids. Of all the options available, Option 6 has perhaps the highest risk since there is little proven experience with this method at other western US water treatment plants.

SUMMARY AND RECOMMENDATION. Planning-level costs for the long-term solids handling alternatives discussed above are presented below in ***Table 6-4***. A comparison of the relative advantages and disadvantages associated with each alternative is presented in ***Table 6-5***.

TABLE 6-4: PLANNING-LEVEL COSTS FOR COMPARISON OF LONG-TERM SOLIDS HANDLING ALTERNATIVES

Option	Capital Costs ($)	Annual O & M Costs ($/year)	Total Present Worth ($)
Option 1	600,000	45,000	1,212,000
Option 2	800,000	65,000	1,684,000
Option 3	700,000	45,000	1,312,000
Option 4	800,000	35,000	1,274,000
Option 5	175,000	75,000	1,195,000
Option 6	175,000	35,000	650,000

TABLE 6-5: COMPARISON OF LONG-TERM SOLIDS HANDLING ALTERNATIVES

Option	Advantages	Disadvantages
Option 1	• City owned property • Existing NPDES permit for discharge into Skunk Creek • No need to "re-handle" solids • Potentially high dewatering efficiencies (up to 30% solids)	• Appears less "natural" than existing lagoon • Dewatering efficiency significantly impacted during winter rainy season • Requires careful management of drying process to ensure high % solids
Option 2	• Relatively small foot-print; facilities can be installed at existing WTP site or at pond site • Without lagoon, Mill Pond site might be available for alternative uses (i.e. park expansion or commercial)	• Thickening and dewatering processes require careful operator attention to ensure proper dewatering • Relatively expensive • Additional chemical requirements • Increased O&M costs
Option 3	• Simplest approach for WTP operation • Potential benefit to WRP pre-treatment due to alum • Eliminates discharge to Skunk Creek • Mill Pond site available for alternative uses (i.e. park expansion or commercial)	• Will WRP accept WTP solids? • Potential impacts to WRP solids processes • Higher hydraulic loading to WRP
Option 4	• Potential benefit to WRP pre-treatment due to alum • At least 20 times less volume pumped to the WRP	• Will WRP accept WTP solids? • Potential impacts to WRP solids processes • Additional WTP operations associated with thickening process
Option 5	• Minimal capital investment • Minimum footprint • No adjustments to current plant operations • Potential benefit to WRP pre-treatment due to alum	• Will WRP accept solids? • Potential impacts to WRP solids processes • Need for dewatering at other sites besides WRP • Operator intensive for dredging operations • Truck traffic
Option 6	• Minimal capital investment • Lowest O&M costs • Minimum footprint • No adjustments to current plant operations • Little increase in truck traffic	• Geo-Tubes un-proven • How fast will solids dewater? • Safety and security issues with tubes around pond • Operator intensive for dredging operations • Requires careful management of polymer addition and drying process to ensure high % solids

The capital costs include a 40% markup over the estimated construction costs for contingencies and engineering. Present worth of annual O&M costs were determined based on 20-year period at an interest rate of 4%.

The annual O&M costs are based on an annual solids production of 140 dry tons/year (= 900 pounds per day average) assuming current/historical alum doses and river turbidities, and an annual average WTP flow of 7.2 mgd which represents a 15 mgd maximum day flow. This condition was used to represent an average condition over the next 20 years considering the current WTP production (10.5 mgd max. day, 5 mgd annual average) and the WTP production in the future (20 mgd max. day, 9.5 mgd annual average). Solids production will vary seasonally.

The analysis for Option 3 also assumes that the plant will operate at 97% production efficiency and that 3% of the water will be produced as washwater and solids flows, resulting in an annual average flow from the WTP to the WRP of approximately 200,000 gallons per day (gpd), with ranges from 100,000 gpd to 500,000 gpd during the year. Instantaneous flows to the WRP for Option 3 were assumed to be 2,000 gpm maximum. Option 4 assumed 20 times less flow to the WRP which represents an annual average flow of 10,000 gpd with instantaneous flows to be 200 gpm maximum.

O&M costs for dewatering options (1, 2 and 6) include $75/wet ton for landfill disposal including handling and removal, trucking and landfill tipping fees. Assume drying beds and Geo-Tubes produce 30 % solids and a centrifuge produces 20 % solids. The O&M costs for these options will be significantly lower if solids are disposed of at the JO-GRO™ facility.

O&M costs for the WRP disposal options (3, 4 and 5) were assumed to be:

- Annual discharge fee = $350/MG + $50/1000 lbs of solids + $12/1000 lbs of COD
- Assume COD of WTP solids = 0 mg/L
- An initial connection fee" to the WRP of $100,000 required for Options 3 and 4
- Tanker truck costs for Option 5 = $150 per trip with at 5,000 gallons per trip

The lowest capital cost Options are 5 and 6. Option 6 has the lowest present worth costs. The City prefers to implement a low-cost solution using a permanent dredge which allows use of either Option 5 or Option 6. Initially, the City will use the Geo-Tube approach for on-site dewatering, and use Option 5 as a fall-back approach. The preferred disposal location for dewatered solids is at the JO-GRO™ facility assuming that this material can be properly mixed with other products to achieve a desirable soil amendment product.

Option 1 should be considered from a long-term planning perspective as the plant continues to increase water production and subsequent solids production. Over time, use of the dredge and Geo-Tube approach may become infeasible or requires too much operator time. ***Figure 6-3*** presents a schematic of sludge drying beds/lagoons at the Mill Pond site to replace the existing storage lagoon in the future. This approach has the lowest capital cost for Options 1 through 4 and offers simpler operations. Further discussion about the feasibility and costs associated with WRP discharge will be required before implementing Option 1. Currently, the City does not prefer the WRP discharge option.

This discussion of solids handling options assumes that the City will continue to receive extensions of its NPDES permit to Skunk Creek and that recycling of lagoon overflow/decant will not be required. However, the City should consider the possibility that discharge to the creek will not be allowed indefinitely, and that recycle may eventually be required. Planning for potential recycle should be considered when making any major plant modifications in the next 5 to 10 years.

6.2.4 Intake Modifications

As discussed previously, the intake requires modifications to meet fish protection criteria. A Technical Memorandum that summarizes current intake deficiencies and improvement options is included in ***Appendix D***.

The recommended approach for planning purposes includes modifications to the existing structure to install flat-plate screens into the river and away from the existing back-eddy. A figure representing the conceptual design for this approach is contained in *Appendix D*. A new screen cleaning system will be required and the existing travelling screen will be removed. The modifications should be designed to allow a 30 mgd withdrawal rate to avoid the need for future (expensive) work in the river when the plant capacity is expanded. The City should make minimal investments in the existing travelling screen to keep it functional for the new few years, but don't purchase and install a new travelling screen as currently budgeted.

The bulk of the construction work for the intake modifications needs to be accomplished during the 6-to-8 week in-water work period during July and August. The predesign, permitting, design and construction will require approximately two years to complete.

The City will need to integrate certain features of the new intake system, including headloss monitoring and cleaning initiation, into its existing WTP SCADA/control system. The estimated project cost for the preferred intake improvement approach is $1.6 million.

6.3 Improvements To Maintain Existing Capacity

Based on the information presented previously, there are significant improvements to be made at the existing WTP to maintain the existing 20 mgd rated capacity, to ensure continued compliance with increasingly-stringent drinking water quality and other regulations, and to improve operations and cost-effectiveness for the plant's remaining useful life. Based on discussions with staff, the recommended improvements are divided into two categories based on prioritized need and/or benefit. Tier-one improvements should be implemented as soon as possible and are considered to be the highest priority.

Tier-two improvements are considered to be important for long-term benefits, but of a lower priority than Tier-one. The recommended Tier-two improvements should be implemented soon after the Tier-one improvements are made, or incorporated into Tier-

one projects if funds are available due to economies of scale (such as re-building filter gallery piping and valves, combined with the filter re-build effort). Some of the Tier-two improvements have lower priority than others and can be deferred until funds are available.

Table 6-6 presents Tier-one improvements and costs followed by brief descriptions of each recommended improvement. ***Table 6-7*** presents Tier-two improvements and costs followed by brief discussions of each improvement. ***Figure 6-4*** indicates the proposed improvements to maintain existing plant capacity and to improve operations.

Total estimated project costs for improvements to the existing plant are $3.0 million for Tier-one and $1.8 million for Tier-two in 2003 dollars. These costs should be escalated due to inflation depending on when the improvements are actually made.

Project costs represent the total estimated cost of implementation including construction costs, engineering and construction management costs, administrative and legal costs, and also contingencies. Estimated construction costs were developed and then 40% was added to develop the project cost estimate. Intake improvements used 50% markup above estimated construction costs due to greater level of uncertainty and risk. The level of accuracy of these estimates represents planning-level within +/- 30% of actual costs.

TABLE 6-6: RECOMMENDED TIER-ONE PLANT IMPROVEMENTS AND COSTS

Improvement/Description	Estimated Project Cost
1. Re-build Existing Filters with surface wash improvements	$ 600,000
2. Add flocculation and baffling to Existing Basins	$ 600,000
3. Solids Handling and Disposal Improvements	$ 175,000
4. Intake Modifications (for 30 mgd capacity)	$1,600,000
Total	***$2,975,000***

TABLE 6-7: RECOMMENDED TIER-TWO PLANT IMPROVEMENTS AND COSTS

Improvement/Description	Estimated Project Cost
5. Replace existing filter valves and new electric actuators	$ 450,000
6. Rebuild filter gallery piping	$ 150,000
7. Replace and relocate filter effluent/filter-to-waste meters	$ 80,000
8. Spare backwash pump and motor	$ 50,000
9. Install continuous sludge removal systems in basins	$ 300,000
10. Relocate lime addition to clearwell; repair clearwell piping	$ 75,000
11. New coagulant feed and injection system	$ 75,000
12. New flowmeters for Basin #3, Raw water and Finished water	$ 75,000
13. Filter effluent Particle Counters	$ 60,000
14. Spectrophotometer for UV_{254} measurements	$ 10,000
15. Containment for Alum Tanks	$ 30,000
16. Storage and Maintenance Area	$ 75,000
17. HVAC Upgrades	$ 75,000
18. Seismic Vulnerability Study	$ 25,000
19. Emergency Power for 5 mgd	$ 300,000
Total	***$1,830,000***

6.3.1 Tier-One Improvements

6.3.1.1 Re-Build Existing Filters with Surface Wash Improvements

As discussed in previous sections, the existing filter media is in poor condition, is very shallow, is not the same in each filter and can not be cleaned properly. The poor media conditions, operating at relatively high filtration rates, are the biggest reason why the

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plant production efficiencies are too low. It is therefore recommended to install new media in each filter. This will first require removal of all support gravel and media.

With the advent of "gravel-less" underdrains which do not require support gravel, it is possible to install a deeper filter media while keeping the elevation of the top of media elevation low enough to minimize the potential for media carryover to the troughs. A larger and deeper dual media of anthracite and sand is recommended to enhance the storage capacity, effluent quality and filter run times compared to the original tri-media of anthracite, sand and garnet.

The filter re-build effort should also include demolition of the existing filter underdrains (Hydro-cones) and installation of low-profile plastic block gravel-less underdrains. Ten inches of 0.5 mm effective size sand and approximately 20 inches of 1.0 mm effective size anthracite can then be installed for a total media depth of 30 inches. MWH has successfully implemented filter re-builds using this same approach in numerous plants in the Pacific Northwest and around the country.

The filters must be re-built one at a time so that the remaining filters are available for treatment. Construction will require approximately 3 weeks per filter for a total of 24 weeks. This field work should begin in the fall following the high demand summer season and should be completed in the spring prior to the beginning of the next high demand season. The entire project duration will be approximately 18 months including design, bidding and construction.

The existing filters have rotating arm surface wash systems which require repair/replacement. The rotating arms, if used, need to be located approximately 2-inches above the top of the new filter media for optimal cleaning. Use of air scour for auxiliary filter cleaning is an attractive idea, but will be significantly more costly compared to surface wash since this would require new air blower(s), piping, valves and electrical/controls. The relatively shallow filter media can be cleaned with surface wash, and therefore this approach is assumed for planning purposes. The City should review

the use of rotating arms and a fixed grid system for surface wash during detailed design. The City will need to modify the backwash sequencing and controls to accommodate the new filters.

6.3.1.2 Add Flocculation and Baffling to Existing Basins

A review of possible basin improvements for improved pretreatment prior to filtration is presented in **Section 6.2**. At a minimum, flocculation should be added to each basin for faster forming and better settling floc. Flocculation options include mechanical (vertical turbine or horizontal paddle wheels) and hydraulically using baffles. Basin #3 requires other improvements to minimize flow short-circuiting in addition to flocculation.

The addition of flocculation to each basin should be developed with a plan for the future plant capacity increase. The plant's pre-filtration (flocculation/sedimentation) capacity can be expanded in a number of different ways for a wide range of costs. It is possible to increase the pre-filtration capacity without adding new structures (to avoid increasing the site's footprint). The lowest cost expansion approach appears to be modifications to Basins 1 and 2 to treat 15 mgd and modifications to Basin #3 to also treat 15 mgd. Currently, Basins 1 and 2 treat 12 mgd and Basin 3 treats 8 mgd. These improvements can be accomplished by adding the proper type of flocculation combined with high-rate settling devices (tube settlers). Continuous sludge removal systems would also be required in each basin.

Therefore, it is recommended that the City add new flocculation systems to each Basin, as part of Tier-one improvements, that are capable of treating the higher flows through each basin in the future. The preliminary improvements plan and cost estimate were based on the addition of vertical turbine flocculators to each basin for planning purposes. Electrical and control improvements will also be required for a complete system.

These improvements should be constructed during the non-peak demand season beginning in the fall, one basin at a time, to keep the plant in service. Timing of improvements to Basin #3 should be carefully determined when plant production is at its

lowest, since it has the highest hydraulic capacity of any of the basins. Similar to the filter re-build project, the project duration is approximately 18 months including design, bidding and construction.

6.3.1.3 Solids Handling and Disposal Improvements

As discussed previously, the City needs to implement a solids handling and disposal system to proactively manage the solids produced at the WTP. Options include discharge to the City's WRP, on-site dewatering for disposal to a landfill or to the JO-GRO™ facility, or trucking liquid solids to an off-site facility for storage, dewatering and ultimate disposal. Discharge of all WTP solids and liquid residuals to the WRP is the simplest approach for the WTP, but it may be not be acceptable from the WRP's perspective, with respect to the solids' impact to the WRP digestion process. The City should continue reviewing the feasibility and costs of the WRP discharge option over the next few years while the short-term improvements are implemented and proven.

The preferred short-term solids handling and disposal option includes the installation of a permanent dredge at the existing pond, and on-site dewatering using Geo-Tubes. The City's preferred disposal site is the JO-GRO™ facility assuming that the dewatered solids can be used as a soil amendment. The City has performed preliminary dewatering tests using polymer addition and alternative Geo-Tube fabrics and feels that this approach has a good chance of success. As a fallback option, the liquid solids removed by the dredge can be trucked off-site.

For long-term planning purposes when the plant exceeds 20 mgd production capacity, the City should plan to develop a series of new sludge drying beds/lagoons at the existing Mill Pond site to replace the existing solids storage lagoon. Water from the equalization basin will be pumped to the lagoons and the clarified overflow will continue to be discharged to Skunk Creek under the City's NPDES permit. When solids fill a lagoon, another lagoon will be put into service and the liquid will be decanted (to Skunk Creek) to allow the solids to dry enough to be removed and hauled to a landfill or to the JO-GRO™ facility.

Construction of the new lagoons would need to be carefully planned to keep part of the existing lagoon in service to allow the WTP to continue discharging solids. Since this is mostly an earthwork project, construction should be planned to occur during the driest months from April through October. Similar to the other projects, the total project duration is approximately 18 months including design, bidding and construction.

6.3.1.4 Intake Improvements

As discussed previously, the intake requires modifications to meet fish protection criteria. The recommended approach for planning purposes includes modifications to the existing structure to install flat-plate screens into the river and away from the existing back-eddy. A new screen cleaning system will be required and the existing travelling screen will be removed. The modifications should be designed to allow a 30 mgd withdrawal rate to avoid the need for future work in the river when the plant capacity is expanded.

The bulk of the construction work needs to be accomplished during the 6-to-8 week in-water work period in July and August. The predesign, permitting, design and construction will require approximately 30 months to complete.

The City will need to integrate certain features of the new intake system, including headloss monitoring and cleaning initiation, into its existing WTP SCADA/control system.

6.3.2 Tier-Two Improvements

6.3.2.1 New Electric Valves/Actuators for Filters

The existing pneumatic valve actuators for the filter process piping are past their useful service life and in need of replacement. Pneumatic actuators are becoming somewhat obsolete in the water industry as utilities migrate towards electric and electronic devices. Replacement and repair parts for pneumatic actuators are also becoming more difficult to

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obtain. Filter control will be simplified and more exact with electric actuators that can be directly linked to the SCADA/control system via new PLC(s).

The City desires to replace the valve actuators with new electric actuators. Also, due to the age of the valves and because some are leaking, it is appropriate to replace the valves since the valves are relatively inexpensive compared to the actuators. Purchasing and installing new valves and actuators may have a similar cost to just replacing the actuators since the actuators can be factory-mounted with the valves versus field installation of the actuators. Providing new valves with the actuators also makes it possible to assign a single point of responsibility for warranty and repair issues, if required.

Each filter has five valves/actuators, of different sizes depending on the filter surface area, which require replacement:

- Open/close influent gate valve
- Modulating effluent valve
- Open/close FTW valve (these valves/electric actuators were installed in 2001)
- Open/close backwash valve
- Open/close waste washwater valve

The new valve actuators will require control stations in the gallery to allow auto/manual valve control, and the valve actuators will also require a dedicated power supply, preferably linked to an Uninteruptible Power Supply (UPS), to operate properly under all conditions. The existing filter control panels located upstairs in the Filter Area should probably remain to allow filter control if the SCADA/computer system is down for any reason.

All of the existing air lines and pneumatic equipment in the gallery should be removed. Once the replacement is completed, the new air compressor and air filter system may no longer be required at the plant, except for perhaps a source of laboratory air.

This work needs to be accomplished in the fall or spring since filters can not be out of service during the peak demand summer season. One filter at a time should be upgraded so that the remaining filters are available for treatment. It is suggested that this work be included with new filter gallery piping and new filter effluent/filter-to-waste meter replacements as part of one project.

For additional economies of scale and to minimize plant disruptions, the City should consider incorporating these filter improvements with the filter media upgrades described for the Tier-one projects. In that case, each filter may require 4 weeks each to re-build, for a total construction duration of 32 weeks.

6.3.2.2 New Filter Gallery Piping

Most of the filter gallery piping is old and leaks in places. Some of the piping materials do not meet today's standards. The pipe supports and joint restraints do not appear to be adequate to ensure a reliable useful life for the next 20+ years. The backwash header recently was damaged due to a hydraulic surge, which pulled a joint apart and started leaking.

Therefore, it is recommended to implement a filter gallery pipe replacement program that should be coordinated with other filter gallery improvements including new valves and actuators, and meter relocations. The City should consider steel pipe and ductile iron as alternative pipe materials.

Each filter's piping would be replaced in such as way as to keep the rest of the plant in service. Ideally, this work would be completed at the same time as the filter re-builds (per Tier-one improvements) to minimize the total disruption to the plant and to achieve economies of scale to lower the costs. If this work is constructed separately, then it will take approximately 3 weeks per filter to complete or 24 weeks total.

6.3.2.3 New Filter Effluent/Filter-to-Waste Flowmeters and Other Instrumentation

The other main components of the filter control system include the filter effluent flowmeters and filter headloss (differential pressure) sensors. It is recommended to

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replace all of these systems with new equipment while the valves and actuators are being replaced. The new flowmeters are required to measure both filter effluent and filter-to-waste flows for better control and process optimization. These meter replacements would best be accomplished with the filter gallery piping and valve/actuator replacements described above. The City should consider using meter technology that matches other plant meters. Magnetic-type meters may be preferable considering the City recently replaced the backwash flowmeter with a magnetic flowmeter.

The existing headloss measurement systems are relatively old and may not be functioning properly since the pressure-sensing tubing may be clogged, and new equipment would ensure a long remaining useful life. The new electronic controls will be linked directly to the SCADA/control system via PLCs which will be installed for the filter valve/actuator controls.

6.3.2.4 Spare Backwash Pump

The plant does not have a reliable backup method for providing backwash water to the filters. If the existing backwash pump fails for any reason, the filters could not be backwashed until the pump is fixed. This would severely limit plant production, especially during summer months when extended operating time is required.

Options for correcting this deficiency include installing a new 2nd backwash pump, improving the design and control of the inter-connect with the high service header to ensure that overpressurizing the underdrains does not occur, or purchasing a new spare pump and motor (un-installed). Based on discussions with staff, the purchase of a new pump is preferred, to save the pump space for a future high service pump and for other reasons. Therefore, the purchase cost of a new pump/motor is included for planning purposes.

6.3.2.5 Continuous Sludge Removal Systems in Basins

The existing basins fill with sludge and need to be manually removed twice per year. The basins must be drained and hosed out, and solids are dumped into the equalization basin, where solids are then pumped to the lagoons. The basins can not be cleaned during

the peak summer demand season. As demands increase, sludge production will also increase and basins will fill more quickly.

The accumulation of sludge in the basins reduces the effective volume for settling and disinfection. The sludge also has potential for causing tastes and odors. The large volumes of sludge discharged during semi-annual cleanings create solids management challenges and can "overwhelm" the lagoon and create possible NPDES discharge violations. If high-rate settling devices (tube settlers or plates) are installed for a capacity expansion, then installation of a solids removal system is required.

It is recommended to add a continuous sludge removal system in each basin. There are a number of options to consider including TracVac, chain and flight and SuperScraper. For this planning analysis, assume the use of a TracVac system in each basin, along with floor modifications to accept the mechanisms and piping modifications to discharge the sludge to the equalization basin. Electrical and control improvements are also required.

This work should be constructed during fall or spring when a basin can be taken off line. This work should be coordinated with planned sludge removal operations and may require each basin to be out of service for up to one week. If the City has available budget, it should consider including this work with the Tier-one basin modifications.

6.3.2.6 Relocate Lime Addition Point to Clearwell; Repair Clearwell Piping

Lime is currently added as a slurry to the end of Basin #2. This hinders plant performance (coagulation/filtration and disinfection) by raising the pH above 9.0 in this water and probably is a cause for manganese deposits and alum "after-floccing" in the distribution system.

Since lime is the most economical pH adjustment chemical, the City should endeavor to continue using lime, but add it to the latter stage of the clearwell to optimize disinfection. There may be concerns about inert particulate matter in the lime slurry increasing the filtered water turbidity, but MWH does not believe this is a health concern. New slurry

piping should be installed to deliver the lime to the clearwell. The City should consult with DHS to confirm that this approach will meet regulatory acceptance before implementing the improvements.

The connecting pipe between clearwell sections (actually a piece of culvert pipe) appears to be leaking based on a visual inspection. The City should make appropriate repairs to this leak when the clearwell can be taken out of service for a period of time.

6.3.2.7 New Coagulant Feed and Injection System

If testing is successful, the City will need to implement a new chemical feed system to add another coagulant in addition to alum. For now, it is assumed that cationic polymer will be used as a coagulant aid. At 1.0 mg/L average dose, this represents a current peak usage rate of 90 pounds per day (ppd) which is equivalent to 10 gallons per day. Therefore, 300 gallons provides 30 days of storage under current conditions.

It is recommended to store cationic polymer in 250 or 400 gallon portable "totes" and feed with a metering pump directly from the tote. Two metering pumps would be required, one for standby. It is recommended to add carrier water to the neat polymer solution for delivery to the raw water feed point. A new polymer feed line needs to be installed from the chemical room to the raw water meter vault. Ideally, the injection of cationic polymer should be prior to alum addition to optimize the reduction in alum dose. The proper injection location should be determined during design.

If the City determines that another coagulant/coagulant aid is preferable compared to cationic polymer (such as ACH or PACl), then the storage and feed system requirements need to be reviewed. It may be possible to use totes if the dosages are low enough, or it may require a new bulk storage tank. In the interim, it may be possible to use one of the alum tanks for the alternative coagulant storage, but this would result in a loss of alum storage capacity and plant reliability in the long-term.

6.3.2.8 New Flowmeters for Basin #3, Raw Water and Finished Water

The plant does not have a functional flowmeter for the flow to Basin #3. It has not been operational for many years. Having a functional meter in this location would allow more accurate raw water flow split to all of the basins. Currently, the flow split is accomplished inexactly by manual means.

It is likely that the existing meter sensing lines have been plugged and the underground location may create additional problems. A new magnetic flowmeter is suggested for installation that can withstand the environmental conditions.

The City should also consider replacing the raw water and finished water flowmeters with similar technology as the new basin #3 meter for consistency and ease of maintenance. The new backwash flowmeter is magnetic and the proposed new filter effluent flowmeters may also be magnetic-type.

This work should be constructed during the fall or spring when the Basin #3 can be taken off-line for a long enough duration to not disrupt plant production.

6.3.2.9 New Filter Particle Counters

The City currently measures the filtered water turbidity from each filter as required by the Surface Water Treatment Rule. The City should consider installing particle counters for each filter effluent to assist in further optimization of plant and filter performance. Particle counting is a more-sensitive measurement than turbidity and can detect the breakthrough of cyst-sized particles sooner than a turbidimeter can. Many surface water treatment plants throughout the Pacific Northwest and the United States have been using particle counting for many years.

6.3.2.10 New UV Spectrophotometer

The City should monitor the TOC of its raw and filtered water periodically and on a routine basis to better understand the removal of organics through the WTP. This monitoring will also benefit the reduction of DBPs in the distribution system by targeting lower pre-chlorine doses when the raw water TOC is higher.

Measurement of TOC is expensive and usually requires out-sourcing to a lab. Alternatively, the City can measure the UV absorbance of the water at 254 nanometers as a surrogate to TOC measurements. In most waters, it is possible to develop a statistical equation between UV_{254} and TOC. Therefore, it is recommended that the City purchase a spectrophotometer capable of measuring UV_{254}.

6.3.2.11 Alum Tank Containment

The City should construct a containment wall around the two existing bulk alum tanks to protect against a catastrophic rupture or leak. The wall will have to be at least 2 feet tall with a surface area of 900 sf to contain 12,000 gallons of liquid alum.

6.3.2.12 Storage and Maintenance Area/Building

The WTP has inadequate protected and sheltered space for storing spare equipment and materials, as well as having limited maintenance/workshop space. It is recommended that the plant construct a 1,000 sf +/- "low-cost" building on the plant site for more-permanent storage. This building could also serve as a limited maintenance area also.

6.3.2.13 HVAC Upgrades

The City should implement improvements to the HVAC system to provide efficient climate control; temperatures are often too hot in the summer and too cold in the winter. Improvements to update heating, cooling and ventilation systems in the Control and Break Rooms located within the Control Building are recommended.

6.3.2.14 Seismic Vulnerability Study

It is recommended to perform a seismic and structural evaluation of the existing plant's buildings, piping and structures to determine if significant improvements may be required to prevent catastrophic damage during a seismic event. The site stability should also be evaluated by a geotechnical professional to determine if there are any issues relative to the long-term viability of the site, including potential issues related to plant modification and expansion improvements.

6.3.2.15 Emergency Power for 5 mgd

The City should plan to add an emergency power generation system at the plant to protect against prolonged power outages. Providing the ability to pump and treat 5 mgd with a backup power supply appears to be adequate to serve the baseload needs of the City. Preliminary sizing indicates that a 500 kW diesel engine generator would be able to operate one raw water pump (75 Hp), one finished water pump (300 Hp) and smaller base plant loads to produce 5 mgd. The generator would be built with its own weatherproof, soundproof enclosure and would require a transfer switch and other electrical work to tie into the plant's existing electrical system. A one to two day fuel storage tank should also be included. Further review and discussion with City staff is required to refine the design, costs and location.

6.3.2.16 Items Not Included

Not included in the lists of recommended improvements and costs presented in ***Tables 6-6 and 6-7*** are:

- New caustic soda or soda ash systems if lime addition to clearwell isn't feasible
- Alternative disinfection system, if required, to meet future regulations such as the D/DBP Rule or the LT2ESWTR
- Seismic and/or structural improvements recommended as a result of the Seismic Evaluation
- OSHA and ADA improvements
- Other items not identified herein

Additional clearwell volume is recommended to be included in the plant expansion improvements as discussed in the following section.

6.4 IMPROVEMENTS TO INCREASE CAPACITY

Various improvements are required to expand the plant's capacity as discussed in **Section 4**. The existing plant site, intake, basins and yard piping are capable of supporting a maximum capacity of 30 mgd. ***Table 6-8*** summarizes the recommended improvements

and project cost estimates for a capacity increase to 30 mgd. Each of the improvements is briefly described and discussed following the table. ***Figure 6-5*** indicates proposed improvements to increase the capacity to 30 mgd.

TABLE 6-8: RECOMMENDED PLANT EXPANSION IMPROVEMENTS (TO 30 MGD) AND COSTS

Improvement/Description	Estimated Project Cost
1. Two new raw water pumps	$ 100,000
2. Chemical System Improvements	$ 150,000
3. Basin Improvements	$2,800,000
4. Three new filters	$1,500,000
5. Additional clearwell volume	$1,250,000
6. Two new high service pumps	$ 200,000
7. Yard Piping Improvements	$ 200,000
8. Surge control improvements	$ 150,000
9. Increase site electrical service	$ 200,000
10. Site electrical improvements to support upgrades	$ 750,000
11. Instrumentation and control improvements to support upgrades	$ 150,000
Total	***$7,350,000***

Total estimated project costs for expanding the plant capacity to 30 mgd are $7.4 million in 2003 dollars. This equates to an approximate unit cost of $0.74 per gallon of added capacity. These costs should be escalated due to inflation and construction cost indices according to when the improvements are actually made. Based on the current rate of demand growth, the plant expansion is not expected to be required until approximately 2025 assuming the City continues to operate the existing plant for longer durations until the 20 mgd production capacity is observed.

Project costs represent the total estimated cost of implementation including construction costs, engineering and construction management costs, administrative and legal costs, and also contingencies. Estimated construction costs were developed and then 40% was

added to develop the project cost estimate. The level of accuracy of these estimates represents planning-level within +/- 30% of actual costs.

6.4.1.1 Two New Raw Water Pumps

A summary of the condition and requirements for the intake and raw water pump station is discussed in previous sections and also in the Intake Review Technical Memorandum (in ***Appendix D***),

The existing four raw water pumps provide a total pumping capacity of 20 mgd and a firm, reliable capacity of 15 mgd with one pump out of service. There is currently space to add two more pumps for capacity expansion.

There are a number of options for increasing the pumping capacity. If two additional 5 mgd pumps are added, then the total installed pumping capacity will be 30 mgd. The firm capacity with this arrangement would be 25 mgd. To develop a firm 30 mgd pumping capacity, two of the 5 mgd pumps would have to be replaced with 10 mgd pumps. Upsizing existing pumps requires careful evaluation of the electrical equipment and motor control center. At least two of the raw water pumps should be equipped with VFDs for optimum plant flow control.

For planning purposes, it is assumed that two new 5 mgd pumps will be added to the Raw Water Intake. One pump will be provided with a new VFD since the plant is currently planning to add one VFD to a raw water pump within the next two years. As discussed previously, at least one of the new pumps should be added prior to the full plant expansion, when demands exceed 15 mgd, to provide a firm/reliable pumping capacity of 20 mgd. Based on current growth projections, the new raw water pump will be required in the next 10 to 15 years.

Improvements to the intake to bring it into compliance with fish protection criteria and to expand the capacity to 30 mgd were presented previously in this section.

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6.4.1.2 Chemical System Improvements

As part of the plant expansion, it is recommended to replace all chemical metering pumps and delivery systems assuming that the equipment currently installed will exceed its useful life within the 20-year planning period. The bulk chemical storage systems will likely need replacement or upsizing also to handle the higher plant flows and higher chemical usage rates.

6.4.1.3 Basin Improvements

Proposed basin improvements to achieve the ultimate capacity of 30 mgd were discussed previously. It is possible to achieve the pre-treatment capacity inside the existing basins via the use of high-rate tube settlers which will minimize the site impact and also provide the lowest cost approach. Basin improvements for Tier-one modifications will provide flocculation to meet the future 30 mgd capacity requirements. Tube settlers will be added for the expansion, including the structural supports and launder modifications required for proper performance. Costs for continuous sludge removal systems are shown for Tier-two improvements.

6.4.1.4 Three New Filters

To achieve a plant capacity of 30 mgd, three new gravity filters should be constructed by extending the existing filters and gallery to the west. The filter surface area, underdrains, media, cleaning systems, and piping/valves should match the systems in the existing filters 6 through 8 (= 324 sf each). The total filter surface area for all 11 filters would then be 3,465 sf. At 30 mgd with one large filter out of service for backwashing, the maximum filtration rate will then be 6.6 gpm/sf.

It is feasible to consider constructing part of the new clearwell volume underneath the new filters as discussed below. Various buried piping needs to be demolished and/or relocated modified to allow the filters to be constructed in the designated area and a construction sequencing plan needs to be developed to ensure that the existing filters remain in service during construction.