CITY OF GRANTS PASS, OREGON

WATER TREATMENT PLANT
FACILITY PLAN
Final Report

MAY 2004

MWH
MONTGOMERY WATSON HARZA

CITY OF GRANTS PASS, OREGON

WATER TREATMENT PLANT
FACILITY PLAN

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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

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 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;

- 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.

FIGURE ES-1: GRANTS PASS WTP SYSTEM IMPROVEMENTS FOR NEXT 10 YEARS

FIGURE ES-2: GRANTS PASS WTP SYSTEM IMPROVEMENTS FOR PLANT EXPANSION TO 30 MGD

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

- 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.

FIGURE 1-1: GRANTS PASS WTP SYSTEM OVERVIEW

FIGURE 1-2: GRANTS PASS WTP PLAN-VIEW LAYOUT

FIGURE 1-3: GRANTS PASS WTP PROCESS FLOW SCHEMATIC

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 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

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

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.

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^{o}F (7^{o}C) and summertime high average temperatures were approximately 61^{o}F (16^{o}C). The lowest observed temperature was 40^{o}F (4.4^{o}C) in February 2002. The highest observed temperature was 74^{o}F (23^{o}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 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 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 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 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 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 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 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.

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 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^{o}F (20^{o}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 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 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^{o}C (68^{o}F). With normal winter water temperatures in the range of 45^{o}F (7^{o}C) and summer normal water temperatures in the range of 61^{o}F (16^{o}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 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:

$$\mathbf{R_e = R_d \times [(UFRV - UBWV)/UFRV]}$$

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 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.

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])

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 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^{o}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

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**).

HISTORICAL PLANT PERFORMANCE

FIGURE 2-1: AVERAGE DAILY RAW AND FINISHED WATER FLOWS

FIGURE 2-2: DAILY AVERAGE RAW WATER TURBIDITY AND PRECIPITATION

FIGURE 2-3: DAILY AVERAGE RAW WATER TEMPERATURE

FIGURE 2-4: DAILY AVERAGE RAW AND FINISHED WATER pH

FIGURE 2-5: 2002 MONTHLY RAW AND FINISHED WATER TOC AND REMOVAL EFFICIENCY

FIGURE 2-6: ROGUE RIVER GEOSMIN LEVELS BETWEEN LOST CREEK DAM AND CITY OF ROGUE RIVER

FIGURE 2-7: DAILY AVERAGE ALUM AND FILTER AID POLYMER DOSE

FIGURE 2-8: DAILY AVERAGE LIME AND PERMANGANATE DOSE

FIGURE 2-9: DAILY AVERAGE MIXED WATER AND EFFLUENT CHLORINE RESIDUALS

FIGURE 2-10: DAILY AVERAGE SEDIMENTATION BASIN EFFLUENT TURBIDITIES

FIGURE 2-11: SEDIMENTATION BASIN TURBIDITY PROBABILITY DISTRIBUTIONS

FIGURE 2-12: TYPICAL FILTER CROSS-SECTIONS

FIGURE 2-13: DAILY AVERAGE FINISHED WATER TURBIDITY

FIGURE 2-14: FINISHED WATER TURBIDITY PROBABILITY DISTRIBUTION

FIGURE 2-15: INDIVIDUAL FILTER EFFLUENT AND COMBINED FINISHED WATER TURBIDITY PROBABILITY DISTRIBUTIONS (5-MINUTE SCADA AVERAGES)

FIGURE 2-16: LIMITING UFRV AND UBWV FOR FILTER PERFORMANCE

FIGURE 2-17: WEEKLY AVERAGE FILTER PRODUCTION EFFICIENCY AND BACKWASH VOLUME

FIGURE 2-18: FILTER PERFORMANCE EVALUATION – BACKWASH TURBIDITY PROFILES

FIGURE 2-19: FILTER PERFORMANCE EVALUATION – FLOC RETENTION PROFILES

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

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.

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

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	-	-

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

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).

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.

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. $\mathbf{T}\ (\text{min}) = \dfrac{[T_{10}/T_{Basin}\ (\text{Reactor Basin Volume}) + T_{10}/T_{CW}\ (\text{Clearwell Volume})](\text{gal})}{\text{Plant Flow (gpm)}}$
2. $\mathbf{C}\ (\text{mg/L}) =$ Minimum In-plant Chlorine Residual
3. $\mathbf{CT}_{achieved}\ (\text{mg/L-min}) = \mathbf{C} \times \mathbf{T}$

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.

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]	^oC	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.

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 ^{o}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 ^{o}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 ^{o}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.

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*** 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	HAA5				
	(mg/L)	(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.

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.

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.

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.

- 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*/C*ryptosporidium*, and the recovery levels measured (the assumption is that the "background" levels of *Giardia*/C*ryptosporidium* 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 ($\geq$0.064 mg/L and/or $\geq$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.

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.

- **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:

- 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.),

- 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.***

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.

FIGURE 3-1: HISTORICAL CT COMPLIANCE

FIGURE 3-2: CT REQUIREMENTS – WORST CASE WINTER CONDITIONS

FIGURE 3-3: CT REQUIREMENTS – WORST CASE SPRING/FALL CONDITIONS

FIGURE 3-4: CT REQUIREMENTS – WORST CASE SUMMER CONDITIONS

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.

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.

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 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

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.

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.

- 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 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 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.

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 $50^{o}F$) 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 $67^{o}F$) 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.

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**.

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

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.

FIGURE 4-1: EXISTING WTP HYDRAULIC PROFILE

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.

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.

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.

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: EXISTING WTP INVENTORY

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

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

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:

- 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 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.

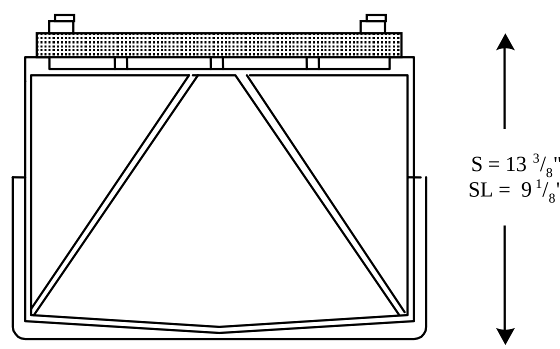
The nozzle design and nozzle spacing must also be determined to meet the needs of the specific installation. The nozzle slit width must ensure controlled air and water distribution as well as retain the smallest media size. Nozzle materials must be carefully selected to avoid erosion of the slits over 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.

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 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^{o} 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^{o} – 35^{o}) • 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 4^{th} 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 4^{th} basin rated at 5 mgd +/-

- 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 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.

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 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 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 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 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.

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.

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.

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.

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

FIGURE 6-1: RECOMMENDED FILTER MODIFICATIONS

FIGURE 6-2: RECOMMENDED IMMEDIATE AND FUTURE BASIN IMPROVEMENTS

FIGURE 6-3: RECOMMENDED LONG-TERM SOLIDS HANDLING IMPROVEMENTS

FIGURE 6-4: RECOMMENDED IMPROVEMENTS TO MAINTAIN EXISTING CAPACITY

FIGURE 6-5: RECOMMENDED IMPROVEMENTS TO INCREASE CAPACITY TO 30 MGD

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

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.

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

- 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

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.

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Oregon

DEPARTMENT OF
HUMAN
RESOURCES

HEALTH DIVISION

October 28, 1997

Mr. Ken Johnson
City of Grants Pass
101 N.W. "A" St.
Grants Pass, OR 97526

Dear Ken:

This letter follows my visit of 10/15/97, during which I updated the sanitary survey data for the City's water system and reviewed the water treatment plant operations. Please also extend my thanks to George Geer for his assistance during my visit.

I note that the installation of filter-to-waste plumbing has now been completed and is functioning well. This, in conjunction with finished water turbidimeters on each filter and finished water consistently below 0.1 NTU, qualifies your treatment plant for a 2.5-log reduction in Giardia-sized particles. The plant was previously credited with a 2.0-log reduction.

This means that, for purposes of demonstrating compliance with the Surface Water Treatment Rule, you only need to demonstrate a 0.5-log reduction for the disinfection process, rather than the 1.0-log reduction previously required.

Congratulations on the excellent operation of your treatment plant, and on making changes to assure that a continuous supply of safe drinking water is delivered to City residents.

Sincerely,

Scott G. Curry, P.E.
Regional Engineer
Drinking Water Program

John A. Kitzhaber
Governor

cc: Rohel Amundson, City of Grants Pass
Josephine Co. HD

800 NE Oregon Street # 21
Portland, OR 97232-2162
(503) 731-4030 Emergency

24-26

GUIDANCE MANUAL

for
Compliance With the
Filtration and
Disinfection Requirements
for Public Water Systems
Using Surface Water
Sources

TABLE E-1
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 0.5 C OR LOWER (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	23	46	69	91	114	137	27	54	82	109	136	163	33	65	98	130	163	195	40	79	119	158	198	237
0.6	24	47	71	94	118	141	28	56	84	112	140	168	33	67	100	133	167	200	40	80	120	159	199	239
0.8	24	48	73	97	121	145	29	57	86	115	143	172	34	68	103	137	171	205	41	82	123	164	205	246
1	25	49	74	99	123	148	29	59	88	117	147	176	35	70	105	140	175	210	42	84	127	169	211	253
1.2	25	51	76	101	127	152	30	60	90	120	150	180	36	72	108	143	179	215	43	86	130	173	216	259
1.4	26	52	78	103	129	155	31	61	92	123	153	184	37	74	111	147	184	221	44	89	133	177	222	266
1.6	26	52	79	105	131	157	32	63	95	126	158	189	38	75	113	151	188	226	46	91	137	182	228	273
1.8	27	54	81	108	135	162	32	64	97	129	161	193	39	77	116	154	193	231	47	93	140	186	233	279
2	28	55	83	110	138	165	33	66	99	131	164	197	39	79	118	157	197	236	48	95	143	191	238	286
2.2	28	56	85	113	141	169	34	67	101	134	168	201	40	81	121	161	202	242	50	99	149	198	248	297
2.4	29	57	86	115	143	172	34	68	103	137	171	205	41	82	124	165	206	247	50	99	149	199	248	298
2.6	29	58	88	117	146	175	35	70	105	139	174	209	42	84	126	168	210	252	51	101	152	203	253	304
2.8	30	59	89	119	148	178	36	71	107	142	178	213	43	86	129	171	214	257	52	103	155	207	258	310
3	30	60	91	121	151	181	36	72	109	145	181	217	44	87	131	174	218	261	53	105	158	211	263	316

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	46	92	139	185	231	277	55	110	165	219	274	329	65	130	195	260	325	390
0.6	48	95	143	191	238	286	57	114	171	228	285	342	68	136	204	271	339	407
0.8	49	98	148	197	246	295	59	118	177	236	295	354	70	141	211	281	352	422
1	51	101	152	203	253	304	61	122	183	243	304	365	73	146	219	291	364	437
1.2	52	104	157	209	261	313	63	125	188	251	313	376	75	150	226	301	376	451
1.4	54	107	161	214	268	321	65	129	194	258	323	387	77	155	232	309	387	464
1.6	55	110	165	219	274	329	66	132	199	265	331	397	80	159	239	318	398	477
1.8	56	113	169	225	282	338	68	136	204	271	339	407	82	163	245	326	408	489
2	58	115	173	231	288	346	70	139	209	278	348	417	83	167	250	333	417	500
2.2	59	118	177	235	294	353	71	142	213	284	355	426	85	170	256	341	426	511
2.4	60	120	181	241	301	361	73	145	218	290	363	435	87	174	261	348	435	522
2.6	61	123	184	245	307	368	74	148	222	296	370	444	89	178	267	355	444	533
2.8	63	125	188	250	313	375	75	151	226	301	377	452	91	181	272	362	453	543
3	64	127	191	255	318	382	77	153	230	307	383	460	92	184	276	368	460	552

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

TABLE E-2
CT VALUES FOR INACTIVATION
OF GIARDIA CYSTS BY FREE CHLORINE
AT 5 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	16	32	49	65	81	97	20	39	59	78	98	117	23	46	70	93	116	139	28	55	83	111	138	166
0.6	17	33	50	67	83	100	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	114	143	171
0.8	17	34	52	69	86	103	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175
1	18	35	53	70	88	105	21	42	63	83	104	125	25	50	75	99	124	149	30	60	90	119	149	179
1.2	18	36	54	71	89	107	21	42	64	85	106	127	25	51	76	101	127	152	31	61	92	122	153	183
1.4	18	36	55	73	91	109	22	43	65	87	108	130	26	52	78	103	129	155	31	62	94	125	156	187
1.6	19	37	56	74	93	111	22	44	66	88	110	132	26	53	79	105	132	158	32	64	96	128	160	192
1.8	19	38	57	76	95	114	23	45	68	90	113	135	27	54	81	108	135	162	33	65	98	131	163	196
2	19	39	58	77	97	116	23	46	69	92	115	138	28	55	83	110	138	165	33	67	100	133	167	200
2.2	20	39	59	79	98	118	23	47	70	93	117	140	28	56	85	113	141	169	34	68	102	136	170	204
2.4	20	40	60	80	100	120	24	48	72	95	119	143	29	57	86	115	143	172	35	70	105	139	174	209
2.6	20	41	61	81	102	122	24	49	73	97	122	146	29	58	88	117	146	175	36	71	107	142	178	213
2.8	21	41	62	83	103	124	25	49	74	99	123	148	30	59	89	119	148	178	36	72	109	145	181	217
3	21	42	63	84	105	126	25	50	76	101	126	151	30	61	91	121	152	182	37	74	111	147	184	221

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	33	66	99	132	165	198	39	79	118	157	197	236	47	93	140	186	233	279
0.6	34	68	102	136	170	204	41	81	122	163	203	244	49	97	146	194	243	291
0.8	35	70	105	140	175	210	42	84	126	168	210	252	50	100	151	201	251	301
1	36	72	108	144	180	216	43	87	130	173	217	260	52	104	156	208	260	312
1.2	37	74	111	147	184	221	45	89	134	178	223	267	53	107	160	213	267	320
1.4	38	76	114	151	189	227	46	91	137	183	228	274	55	110	165	219	274	329
1.6	39	77	116	155	193	232	47	94	141	187	234	281	56	112	169	225	281	337
1.8	40	79	119	159	198	238	48	96	144	191	239	287	58	115	173	230	288	345
2	41	81	122	162	203	243	49	98	147	196	245	294	59	118	177	235	294	353
2.2	41	83	124	165	207	248	50	100	150	200	250	300	60	120	181	241	301	361
2.4	42	84	127	169	211	253	51	102	153	204	255	306	61	123	184	245	307	368
2.6	43	86	129	172	215	258	52	104	156	208	260	312	63	125	188	250	313	375
2.8	44	88	132	175	219	263	53	106	159	212	265	318	64	127	191	255	318	382
3	45	89	134	179	223	268	54	108	162	216	270	324	65	130	195	259	324	389

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

TABLE E-3

CT VALUES FOR INACTIVATION

OF GIARDIA CYSTS BY FREE CHLORINE

AT 10 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0	pH=6.5 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	12	24	37	49	61	73	15	29	44	59	73	88
0.6	13	25	38	50	63	75	15	30	45	60	75	90
0.8	13	26	39	52	65	78	15	31	46	61	77	92
1	13	26	40	53	66	79	16	31	47	63	78	94
1.2	13	27	40	53	67	80	16	32	48	63	79	95
1.4	14	27	41	55	68	82	16	33	49	65	82	98
1.6	14	28	42	55	69	83	17	33	50	66	83	99
1.8	14	29	43	57	72	86	17	34	51	67	84	101
2	15	29	44	58	73	87	17	35	52	69	87	104
2.2	15	30	45	59	74	89	18	35	53	70	88	105
2.4	15	30	45	60	75	90	18	36	54	71	89	107
2.6	15	31	46	61	77	92	18	37	55	73	92	110
2.8	16	31	47	62	78	93	19	37	56	74	93	111
3	16	32	48	63	79	95	19	38	57	75	94	113

CHLORINE CONCENTRATION (mg/L)	pH=7.0 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0	pH=7.5 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	17	35	52	69	87	104	21	42	63	83	104	125
0.6	18	36	54	71	89	107	21	43	64	85	107	128
0.8	18	37	55	73	92	110	22	44	66	87	109	131
1	19	37	56	75	93	112	22	45	67	89	112	134
1.2	19	38	57	76	95	114	23	46	69	91	114	137
1.4	19	39	58	77	97	116	23	47	70	93	117	140
1.6	20	40	60	79	99	119	24	48	72	96	120	144
1.8	20	41	61	81	102	122	25	49	74	98	123	147
2	21	41	62	83	103	124	25	50	75	100	125	150
2.2	21	42	64	85	106	127	26	51	77	102	128	153
2.4	22	43	65	86	108	129	26	52	79	105	131	157
2.6	22	44	66	87	109	131	27	53	80	107	133	160
2.8	22	45	67	89	112	134	27	54	82	109	136	163
3	23	46	69	91	114	137	28	55	83	111	138	166

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0	pH=8.5 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0	pH<=9.0 Log Inactivations 0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	25	50	75	99	124	149	30	59	89	118	148	177	35	70	105	139	174	209
0.6	26	51	77	102	128	153	31	61	92	122	153	183	36	73	109	145	182	218
0.8	26	53	79	105	132	158	32	63	95	126	158	189	38	75	113	151	188	226
1	27	54	81	108	135	162	33	65	98	130	163	195	39	78	117	156	195	234
1.2	28	55	83	111	138	166	33	67	100	133	167	200	40	80	120	160	200	240
1.4	28	57	85	113	142	170	34	69	103	137	172	206	41	82	124	165	206	247
1.6	29	58	87	116	145	174	35	70	106	141	176	211	42	84	127	169	211	253
1.8	30	60	90	119	149	179	36	72	108	143	179	215	43	86	130	173	216	259
2	30	61	91	121	152	182	37	74	111	147	184	221	44	88	133	177	221	265
2.2	31	62	93	124	155	186	38	75	113	150	188	225	45	90	136	181	226	271
2.4	32	63	95	127	158	190	38	77	115	153	192	230	46	92	138	184	230	276
2.6	32	65	97	129	162	194	39	78	117	156	195	234	47	94	141	187	234	281
2.8	33	66	99	131	164	197	40	80	120	159	199	239	48	96	144	191	239	287
3	34	67	101	134	168	201	41	81	122	162	203	243	49	97	146	195	243	292

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

TABLE E-4
CT VALUES FOR INACTIVATION OF GIARDIA CYSTS BY FREE CHLORINE AT 15 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	8	16	25	33	41	49	10	20	30	39	49	59	12	23	35	47	58	70	14	28	42	55	69	83
0.6	8	17	25	33	42	50	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86
0.8	9	17	26	35	43	52	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88
1	9	18	27	35	44	53	11	21	32	42	53	63	13	25	38	50	63	75	15	30	45	60	75	90
1.2	9	18	27	36	45	54	11	21	32	43	53	64	13	25	38	51	63	76	15	31	46	61	77	92
1.4	9	18	28	37	46	55	11	22	33	43	54	65	13	26	39	52	65	78	16	31	47	63	78	94
1.6	9	19	28	37	47	56	11	22	33	44	55	66	13	26	40	53	66	79	16	32	48	64	80	96
1.8	10	19	29	38	48	57	11	23	34	45	57	68	14	27	41	54	68	81	16	33	49	65	82	98
2	10	19	29	39	48	58	12	23	35	46	58	69	14	28	42	55	69	83	17	33	50	67	83	100
2.2	10	20	30	39	49	59	12	23	35	47	58	70	14	28	43	57	71	85	17	34	51	68	85	102
2.4	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86	18	35	53	70	88	105
2.6	10	20	31	41	51	61	12	24	37	49	61	73	15	29	44	59	73	88	18	36	54	71	89	107
2.8	10	21	31	41	52	62	12	25	37	49	62	74	15	30	45	59	74	89	18	36	55	73	91	109
3	11	21	32	42	53	63	13	25	38	51	63	76	15	30	46	61	76	91	19	37	56	74	93	111

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	17	33	50	66	83	99	20	39	59	79	98	118	23	47	70	93	117	140
0.6	17	34	51	68	85	102	20	41	61	81	102	122	24	49	73	97	122	146
0.8	18	35	53	70	88	105	21	42	63	84	105	126	25	50	76	101	126	151
1	18	36	54	72	90	108	22	43	65	87	108	130	26	52	78	104	130	156
1.2	19	37	56	74	93	111	22	45	67	89	112	134	27	53	80	107	133	160
1.4	19	38	57	76	95	114	23	46	69	91	114	137	28	55	83	110	138	165
1.6	19	39	58	77	97	116	24	47	71	94	118	141	28	56	85	113	141	169
1.8	20	40	60	79	99	119	24	48	72	96	120	144	29	58	87	115	144	173
2	20	41	61	81	102	122	25	49	74	98	123	147	30	59	89	118	148	177
2.2	21	41	62	83	103	124	25	50	75	100	125	150	30	60	91	121	151	181
2.4	21	42	64	85	106	127	26	51	77	102	128	153	31	61	92	123	153	184
2.6	22	43	65	86	108	129	26	52	78	104	130	156	31	63	94	125	157	188
2.8	22	44	66	88	110	132	27	53	80	106	133	159	32	64	96	127	159	191
3	22	45	67	89	112	134	27	54	81	108	135	162	33	65	98	130	163	195

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

TABLE E-5
CT VALUES FOR INACTIVATION
OF GIARDIA CYSTS BY FREE CHLORINE
AT 20 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	6	12	18	24	30	36	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62
0.6	6	13	19	25	32	38	8	15	23	30	38	45	9	18	27	36	45	54	11	21	32	43	53	64
0.8	7	13	20	26	33	39	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66
1	7	13	20	26	33	39	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67
1.2	7	13	20	27	33	40	8	16	24	32	40	48	10	19	29	38	48	57	12	23	35	46	58	69
1.4	7	14	21	27	34	41	8	16	25	33	41	49	10	19	29	39	48	58	12	23	35	47	58	70
1.6	7	14	21	28	35	42	8	17	25	33	42	50	10	20	30	39	49	59	12	24	36	48	60	72
1.8	7	14	22	29	36	43	9	17	26	34	43	51	10	20	31	41	51	61	12	25	37	49	62	74
2	7	15	22	29	37	44	9	17	26	35	43	52	10	21	31	41	52	62	13	25	38	50	63	75
2.2	7	15	22	29	37	44	9	18	27	35	44	53	11	21	32	42	53	63	13	26	39	51	64	77
2.4	8	15	23	30	38	45	9	18	27	36	45	54	11	22	33	43	54	65	13	26	39	52	65	78
2.6	8	15	23	31	38	46	9	18	28	37	46	55	11	22	33	44	55	66	13	27	40	53	67	80
2.8	8	16	24	31	39	47	9	19	28	37	47	56	11	22	34	45	56	67	14	27	41	54	68	81
3	8	16	24	31	39	47	10	19	29	38	48	57	11	23	34	45	57	68	14	28	42	55	69	83

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	12	25	37	49	62	74	15	30	45	59	74	89	18	35	53	70	88	105
0.6	13	26	39	51	64	77	15	31	46	61	77	92	18	36	55	73	91	109
0.8	13	26	40	53	66	79	16	32	48	63	79	95	19	38	57	75	94	113
1	14	27	41	54	68	81	16	33	49	65	82	98	20	39	59	78	98	117
1.2	14	28	42	55	69	83	17	33	50	67	83	100	20	40	60	80	100	120
1.4	14	28	43	57	71	85	17	34	52	69	86	103	21	41	62	82	103	123
1.6	15	29	44	58	73	87	18	35	53	70	88	105	21	42	63	84	105	126
1.8	15	30	45	59	74	89	18	36	54	72	90	108	22	43	65	86	108	129
2	15	30	46	61	76	91	18	37	55	73	92	110	22	44	66	88	110	132
2.2	16	31	47	62	78	93	19	38	57	75	94	113	23	45	68	90	113	135
2.4	16	32	48	63	79	95	19	38	58	77	96	115	23	46	69	92	115	138
2.6	16	32	49	65	81	97	20	39	59	78	98	117	24	47	71	94	118	141
2.8	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	95	119	143
3	17	34	51	67	84	101	20	41	61	81	102	122	24	49	73	97	122	146

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

TABLE E-6

CT VALUES FOR INACTIVATION

OF GIARDIA CYSTS BY FREE CHLORINE

AT 25 C (1)

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivations						pH=6.5 Log Inactivations						pH=7.0 Log Inactivations						pH=7.5 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	4	8	12	16	20	24	5	10	15	19	24	29	6	12	18	23	29	35	7	14	21	28	35	42
0.6	4	8	13	17	21	25	5	10	15	20	25	30	6	12	18	24	30	36	7	14	22	29	36	43
0.8	4	9	13	17	22	26	5	10	16	21	26	31	6	12	19	25	31	37	7	15	22	29	37	44
1	4	9	13	17	22	26	5	10	16	21	26	31	6	12	19	25	31	37	8	15	23	30	38	45
1.2	5	9	14	18	23	27	5	11	16	21	27	32	6	13	19	25	32	38	8	15	23	31	38	46
1.4	5	9	14	18	23	27	6	11	17	22	28	33	7	13	20	26	33	39	8	16	24	31	39	47
1.6	5	9	14	19	23	28	6	11	17	22	28	33	7	13	20	27	33	40	8	16	24	32	40	48
1.8	5	10	15	19	24	29	6	11	17	23	28	34	7	14	21	27	34	41	8	16	25	33	41	49
2	5	10	15	19	24	29	6	12	18	23	29	35	7	14	21	27	34	41	8	17	25	33	42	50
2.2	5	10	15	20	25	30	6	12	18	23	29	35	7	14	21	28	35	42	9	17	26	34	43	51
2.4	5	10	15	20	25	30	6	12	18	24	30	36	7	14	22	29	36	43	9	17	26	35	43	52
2.6	5	10	16	21	26	31	6	12	19	25	31	37	7	15	22	29	37	44	9	18	27	35	44	53
2.8	5	10	16	21	26	31	6	12	19	25	31	37	8	15	23	30	38	45	9	18	27	36	45	54
3	5	11	16	21	27	32	6	13	19	25	32	38	8	15	23	31	38	46	9	18	28	37	46	55

CHLORINE CONCENTRATION (mg/L)	pH=8.0 Log Inactivations						pH=8.5 Log Inactivations						pH<=9.0 Log Inactivations					
	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0	0.5	1.0	1.5	2.0	2.5	3.0
<=0.4	8	17	25	33	42	50	10	20	30	39	49	59	12	23	35	47	58	70
0.6	9	17	26	34	43	51	10	20	31	41	51	61	12	24	37	49	61	73
0.8	9	18	27	35	44	53	11	21	32	42	53	63	13	25	38	50	63	75
1	9	18	27	36	45	54	11	22	33	43	54	65	13	26	39	52	65	78
1.2	9	18	28	37	46	55	11	22	34	45	56	67	13	27	40	53	67	80
1.4	10	19	29	38	48	57	12	23	35	46	58	69	14	27	41	55	68	82
1.6	10	19	29	39	48	58	12	23	35	47	58	70	14	28	42	56	70	84
1.8	10	20	30	40	50	60	12	24	36	48	60	72	14	29	43	57	72	86
2	10	20	31	41	51	61	12	25	37	49	62	74	15	29	44	59	73	88
2.2	10	21	31	41	52	62	13	25	38	50	63	75	15	30	45	60	75	90
2.4	11	21	32	42	53	63	13	26	39	51	64	77	15	31	46	61	77	92
2.6	11	22	33	43	54	65	13	26	39	52	65	78	16	31	47	63	78	94
2.8	11	22	33	44	55	66	13	27	40	53	67	80	16	32	48	64	80	96
3	11	22	34	45	56	67	14	27	41	54	68	81	16	32	49	65	81	97

Notes:

(1) $CT_{99.9}$ = CT for 3-log inactivation

BLACK & VEATCH

MEMORANDUM

Black & Veatch Corporation

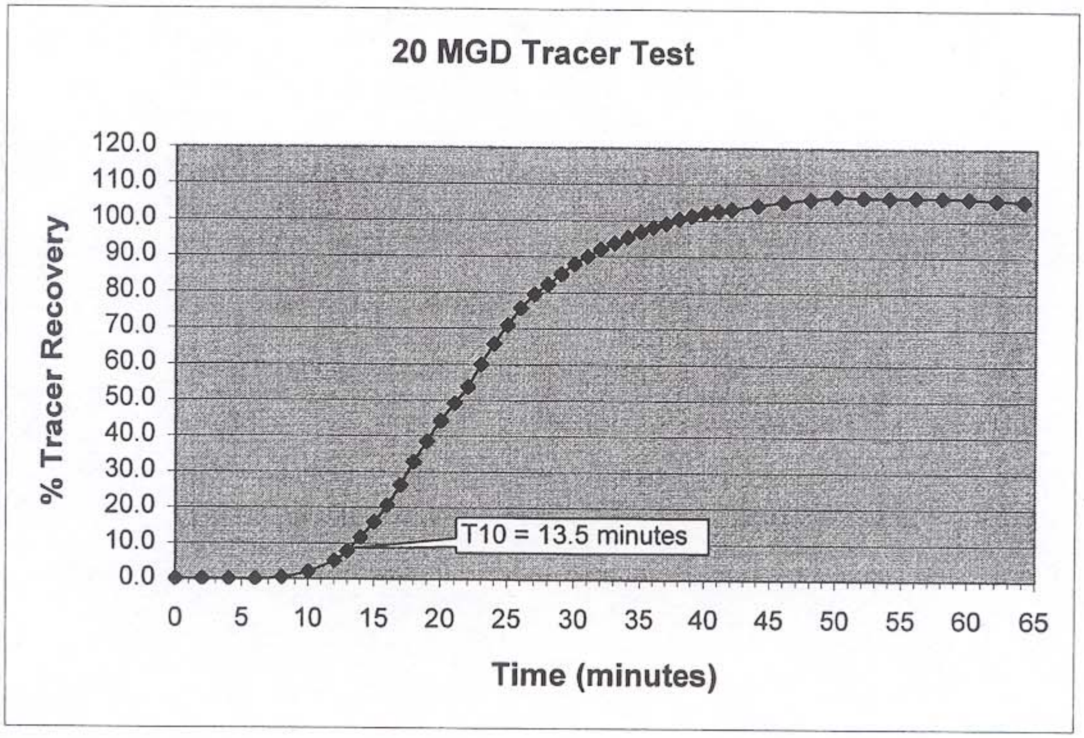
To:	Jason Canady Water Treatment Plant Supervisor	B&V Project No.: 135540
From:	Robert Ward, P.E. Michelle Cheek, E.I.T.	B&V File: B 5.0
Date:	July 9, 2003	
RE:	Grants Pass Tracer Study – Final Report	

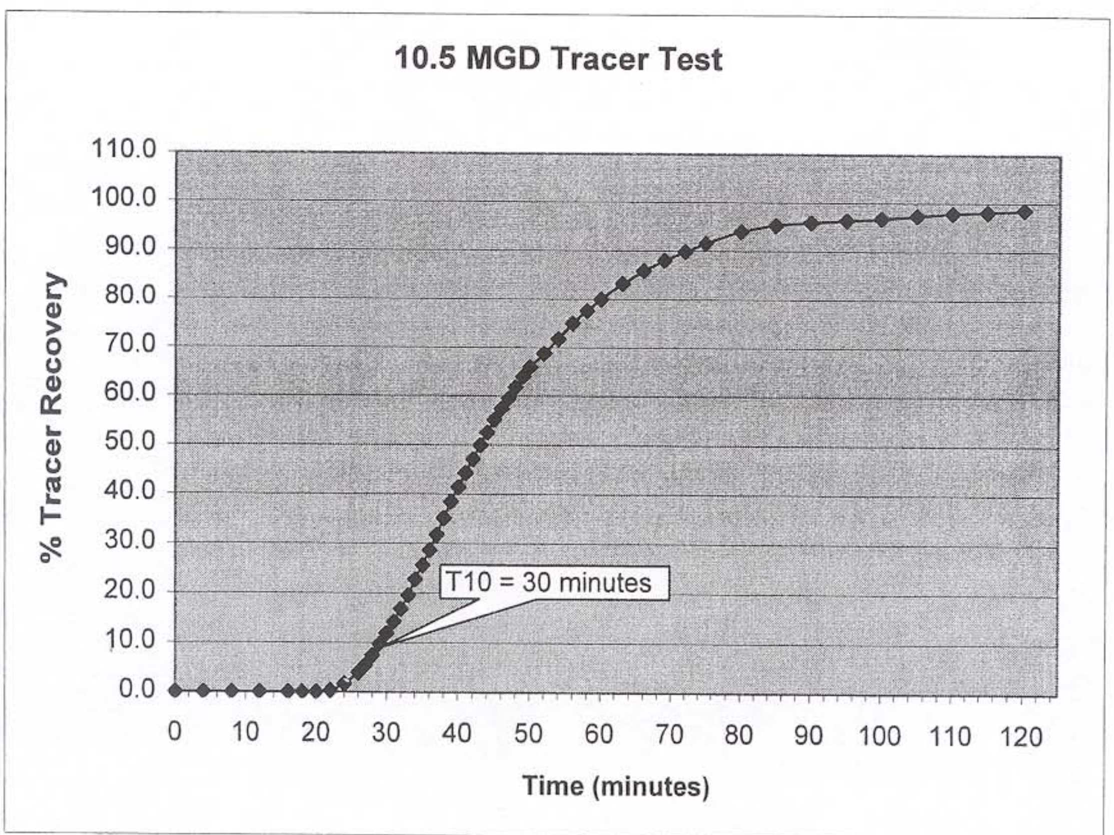
On June 24, 2003 Black and Veatch performed a tracer study of the existing clearwell at the City of Grants Pass Water Filtration Plant. The purpose of the tracer study was to determine the effective contact time for calculation of CT values in the clear well for the purpose of determining compliance with the disinfection requirements of the Surface Water Treatment Rule (SWTR).

To determine the contact time, a slug-dose tracer study was conducted with fluoride (hydrofluosilicic acid) as the tracer compound. Tests were conducted at approximately 7,300 gpm (10.5 mgd) and 13,900 gpm (20 mgd) at a constant clearwell level of 13.6 feet. Fluoride was added at the clearwell inlet and samples were collected at the clearwell discharge for analysis. The quality of the data is greatly enhanced by the ability of the plant staff to maintain constant flows and levels during the study. The Grants Pass treatment staff did an excellent job of maintaining the levels and flows during the test as well as setting up the fluoride insertion and sampling procedures for the testing.

Samples were initially collected every 2 to 5 minutes following the addition of fluoride. As the fluoride concentrations at the clearwell discharge began to increase, the sampling interval was reduced to 1 minute in order to adequately define the fluoride concentration profile. After the fluoride concentration at the clearwell discharge peaked and began to decline, sampling was reduced to every 2 to 5 minutes for the remainder of the test. Background fluoride concentrations at the clearwell inlet were periodically monitored to calculate the tracer recovery and provide a check on the overall accuracy of the testing procedures. Tracer recovery of approximately 90% or higher is considered indicative of a successful test. Tracer recovery of more than 100% may occur if there are minor inaccuracies in the analysis of the samples or the percent available fluoride for the hydrofluosilicic acid is higher than indicated by the fluoride manufacturer.

The two graphs below show the percent tracer recovery versus time for each tracer test. The T10 value (time at which 10% of the water has passed through the reservoir) for each test is noted on each graph and corresponds to the time at which 10% of the tracer was recovered.


20 MGD Tracer Test
120.0
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
% Tracer Recovery
T10 = 13.5 minutes
0
5
10
15
20
25
30
35
40
45
50
55
60
65
Time (minutes)


10.5 MGD Tracer Test
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
% Tracer Recovery
T10 = 30 minutes
0
10
20
30
40
50
60
70
80
90
100
110
120
Time (minutes)

The hydraulic detention time (clearwell volume divided by flow) was also calculated for each of the tracer tests and was used to determine the baffling factor (T10 divided by hydraulic detention time). As the T10 value approaches the hydraulic detention time, the baffling factor increases indicating flow conditions approaching perfect or plug flow thru the clearwell. A baffling factor of 0.5 is considered average and a baffling factor of 0.7 or higher is considered superior. The T10, hydraulic detention time, and baffling factor for each tracer test are as follows:

Flow (mgd)	T10 (minutes)	Hydraulic Detention Time (minutes)	Baffling Factor
10.5 (7,300 gpm)	30.0	49.6 (362, 000 gal/7,300 gpm)	0.60
20 (13,900 gpm)	13.5	26.1 (362,000 gal/13,900 gpm)	0.50

The calculated T10 estimated during a Comprehensive Performance Evaluation (CPE) performed in 1993 by the Oregon Department of Human Services Drinking Water Program (DWP) are as follows:

Flow (mgd)	Calculation	Calculated T10 (minutes)
9.2 (6,400 gpm)	303,000 gal/6,400 gpm	47.3
18.4 (12,800 gpm)	303,000gal/ 12,800 gpm	23.7

The clearwell volume (303,000 gal vs. 362,000gal) and flows (7,300 gpm and 13,900 gpm versus 6,400 gpm and 12, 800 gpm) used in the CPE are different than those used during the tracer tests, and the calculated T10 time assumes a baffling factor of 1.0. Assuming a baffling factor of 1.0 results in a significantly longer T10 time. The T10 values from the tracer tests are significantly lower than the calculated T10 times estimated during the CPE performed in 1993. If a baffling factor of 0.6 is used in calculating the T10 time from the CPE, the resulting T10 times for the CPE are 28.4 minutes for 6,400 gpm and 14.2 minutes for 12,800 gpm.

Recommendations:

Based on the baffling factors calculated, the clearwell appears to exhibit average to above average baffling. However, the octagonal portion of the clearwell does not have any baffles. Adding baffles to this portion of the clearwell may increase the contact time through the clearwell. The positive impact from adding baffles to the clearwell may be limited. Other modifications that may increase the contact time through the clearwell should be evaluated and considerations should be given to increasing the size of the clearwell.

From the tracer study, it appears that chlorination at the current level in the clearwells cannot solely provide the contact time needed to provide adequate disinfection at current chlorination levels. Pre-chlorination will need to continue until modifications are made to increase the contact time. Pre-chlorination currently takes place at the head of the treatment plant. Consideration should be given to pre-filter chlorination in lieu of pre-chlorination at the head of the treatment plant. Pre-filter chlorination will provide contact time through the filter and will reduce the potential for forming disinfection by-products (DBPs). Enhanced coagulation, adding higher doses of alum, should also be considered as it may remove more DBP precursors from the water which will in turn decrease the potential for forming DBPs.

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Tracer Study Testing Protocol

City of Grants Pass, OR
Testing Protocol
Treated Water Clearwell Tracer Evaluation

1. General

The pulse input ("slug dose") tracer application method will be utilized, with fluoride (hydrofluosilicic acid) as the tracer compound. Fluoride will be added at the reservoir inlet, and fluoride residual will be monitored at the reservoir discharge at a point to be determined through discussions with plant staff prior to initiation of testing. Testing will be conducted at two flow rates: (1) a maximum rate equivalent to the current rated plant capacity (12,500 gpm), or, at the maximum rate that can be reliably maintained over the anticipated testing period, and (2) a reduced rate consistent with average system demands (7,000 gpm). The clearwell will be maintained at the "typical" operating level (per plant staff, average 13.6 ft. water depth, which corresponds to a water volume of 396,650 gallons throughout the testing period. (It is extremely important that a consistent flow rate and clearwell operating level be maintained during the testing period.) Fluoride residual concentrations will be determined using a Hach DR2000 spectrophotometer and the SPADNS method with appropriate reagents. (Samples with fluoride ion concentrations exceeding 2.0 mg/L will be diluted 50:50 using distilled or deionized water prior to analysis.)

2. Testing Duration

It will be necessary to continue the test until fluoride concentrations at the clearwell discharge return to "background" levels (i.e., until all of the fluoride added at the inlet has passed through the clearwell). While the required testing period for a given facility will be determined by several factors (level of internal clearwell baffling present, flow rate and clearwell operating depths, degree of initial fluoride mixing provided), previous experience indicates that monitoring will likely need to be continued for a period equivalent to approximately 2.5 to 3 times the theoretical detention time of the clearwell. Assuming operation of the clearwell at a volume of 396,650 gallons, theoretical detention time at test flow rate of 7,000 gpm and 12,500 gpm would be approximately 57 minutes and 32 minutes, respectively. Therefore, the required testing period will likely be approximately 3 hours for the 7,000 gpm flow rate, and 1.5 hours for the 12,500 gpm flow rate.

3. Tracer Volume and Application

Recommended tracer dosage is approximately 7-8 gallons of 23 to 25 percent hydrofluosilicic acid per million gallons of clearwell capacity. Based on previous testing experience, this dosage should yield a peak fluoride concentration of 3.0 to 3.5 mg/L at the clearwell discharge. (This peak fluoride concentration is typically only experienced for a relatively short period; average finished water fluoride concentrations over the entire testing period will be less than 2.0 mg/L.) At the anticipated clearwell operating level, required tracer addition would therefore be approximately 3.5 gallons per test. Acid strength (% H_2SiF_6) and specific gravity/density should be recorded (this information is typically provided by the chemical supplier).

The hydrofluosilicic acid should be applied using one or more small containers to facilitate rapid introduction of the acid into the process stream at the point of application (5-gallon carboys are particularly well suited for this application). Provisions should be made for feeding the required acid volume in one minute or less. (Plant staff should be made aware of the hazards associated with handling hydrofluosilicic acid; appropriate safety equipment should be available.) As the accuracy of the test results obtained will be dependent in large part upon the ability to accurately determine the amount of hydrofluosilicic acid added, efforts should be made to ensure accurate measurement. This may involve marking the level of acid in the container(s), with subsequent volumetric determination of actual liquid volumes using water, and/or weighing of the container and acid both prior to and after introduction of the acid at the clearwell inlet.

4. Tasks to be Completed Prior to Testing

a. Identify a readily-accessible fluoride monitoring point near the clearwell discharge. Ideally, this will be a sample tap that can be opened and allowed to flow continuously during the testing period. If feasible, the fluoride analysis equipment should be relocated to a point as near as possible to the fluoride monitoring point to minimize sample transport times.

b. Calibrate the fluoride analysis equipment.

c. Backwash filters just prior to testing such that the need for backwashing during testing is minimized or eliminated.

d. Stabilize the flow through the plant and clearwell, and adjust the water level in the clearwell (if necessary) to a level corresponding to "typical" operating conditions.

e. Determine the "background" fluoride concentration at the filter discharge prior to the point of fluoride addition.

5. Initial Testing

a. Add fluoride at the clearwell inlet, and collect samples at the clearwell discharge for fluoride analysis every 1-2 minutes following addition of the fluoride.

b. As fluoride concentrations at the clearwell discharge begin to increase rapidly, the sampling interval should be reduced to 1 minute in order to adequately define the fluoride concentration profile. (The overall accuracy of the test increases with increasing number of samples analyzed.)

c. Monitoring results, including sampling times, fluoride concentrations, clearwell water levels, flow rates, and water temperatures should be recorded on the data sheets provided by the Engineer (example data logging sheets are attached).

6. Remainder of Testing Period

a. When fluoride concentrations at the clearwell discharge peak and then begin to decline, frequency of sampling can be gradually reduced to every 3-5 minutes for the remainder of the test. (Actual sampling intervals will be determined during testing based on the intensity of the fluoride concentration peak obtained and the rate at which fluoride concentrations subsequently decline.)

b. Periodically monitor background fluoride concentrations at the clearwell inlet (approximately every 2 hours as a minimum).

c. Testing should ideally be continued until the fluoride concentration at the clearwell discharge is equal to the fluoride concentration at the clearwell inlet. However, should continued testing to achieve equivalent clearwell inlet and discharge concentrations prove to be impractical due to system demands, testing may be terminated when clearwell inlet and discharge fluoride concentrations are within approximately 10 to 15 percent. (It is emphasized, however, that early termination of testing will reduce the accuracy of the results obtained.)

7. Following Completion of Testing

The Engineer will evaluate the data obtained for all tests. The T_{10} detention time, and the ratio of the T_{10} time to the clearwell's theoretical hydraulic retention time ("T_{10}/DT") will be determined for each test. Tracer recovery for each test will be calculated to provide a check on the overall accuracy of the testing procedures (tracer recovery of approximately 90% or higher will be considered indicative of a successful test). A letter report summarizing the testing procedures and results for all tests will be prepared and submitted to the Owner.

8. Owner / Engineer Responsibilities

The Owner will be responsible for acquiring the volume of hydrofluosilicic acid required to conduct all tests, as specified by the Engineer. If the acid is obtained in drum quantities, the Owner should also obtain sufficient smaller containers (typically 5 gallon carboys) as appropriate to facilitate addition of the acid in a safe manner at the specified feed point. The Owner will be responsible for transferring of the acid from the drum(s) to these containers prior to testing, and for transporting these containers to the point of fluoride application. Approximately 40 to 50 small (100-200 mL) plastic sample bottles should also be obtained for collection of fluoride residual samples. The Owner will be responsible for providing the equipment used for analysis of fluoride residual concentrations, for initial calibration of this equipment, and for analysis of the samples collected and periodic verification of calibration accuracy as necessary. The Engineer will be responsible for collection and transport of all samples to the analysis location, and for compiling, analyzing, and reporting of the resulting data.

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Tracer Study Data

Grants Pass Tracer Study
June 24, 2003
20 MGD Slug Dose Tracer Test

Location:	Grant's Pass WTP
Test Date:	June 24, 2003
Unit(s) Evaluated:	0.433 MG Clearwell

Fluoride Data:

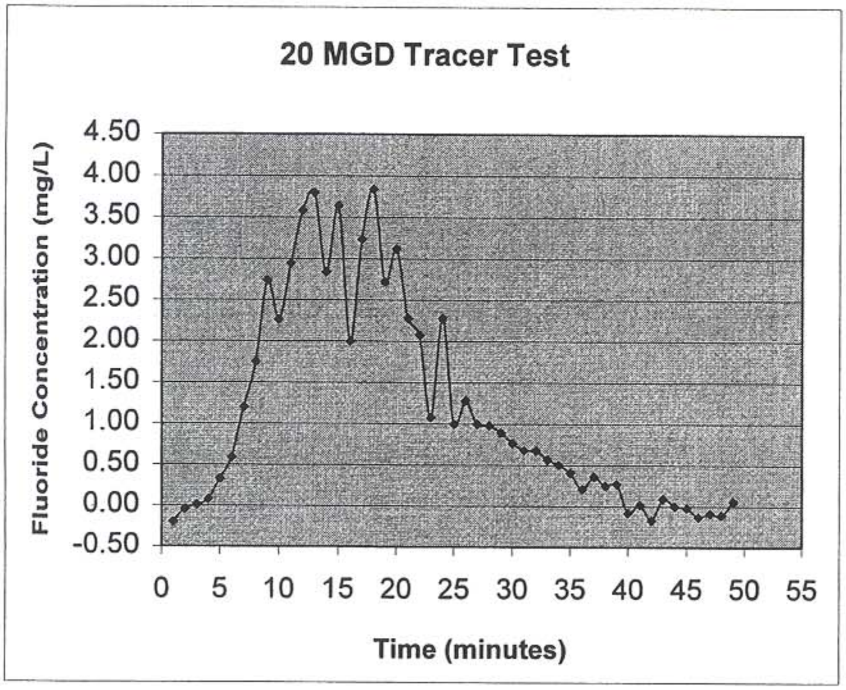
% Acid Used:	25	(From chemical supplier information)
Spec. Gravity:	1.234	(From chemical supplier information)
Available F, lbs/gallon:	2.0352	
Acid Added, gallons:	3.25	(Typically 7-8 gallons of hydrofluosilicic acid per MG of unit capacity evaluated)
Fluoride Ion Added, lbs:	6.6143	

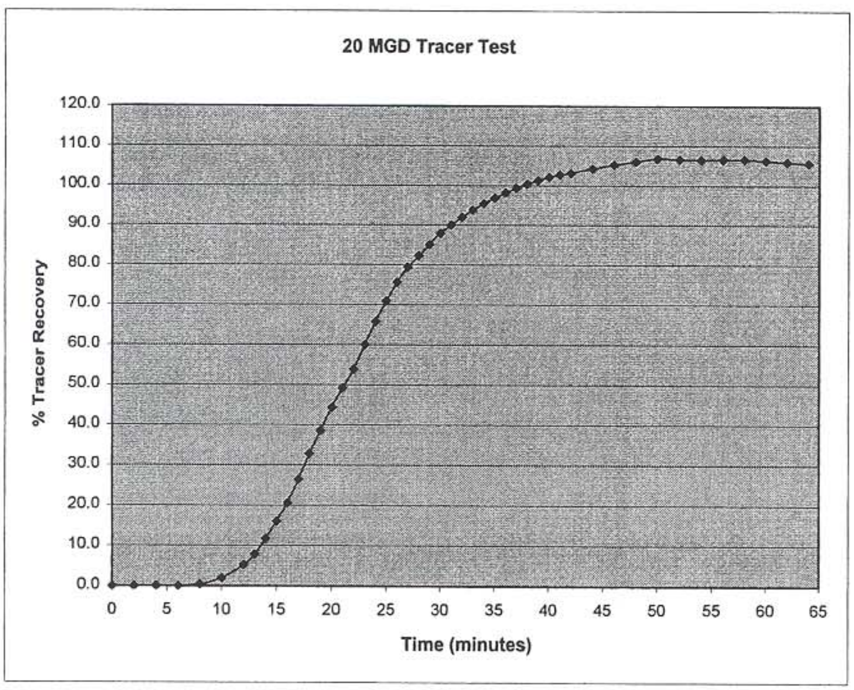
Unit Data:

Maximum Volume, MG:	0.433	
Maximum SWD, feet:	14.5	
Volume per foot depth, MG	0.0299	(Only valid if floor elevation is consistent for entire unit being tested)

		Clock	Time in Minutes		Fluoride Concentration, mg/L			Reservoir		Tracer, mg-min/L		Tracer Recovered, lbs		Tracer
		Time	Elapsed	Increment	Sample	Background	Tracer	Depth, ft	Flow, mgd	Increment	Cumulative	Increment	Cumulative	Recovery, %
(Tracer Addition Time)	1	0908	0	0	0.00	0.20	-0.20	13.72	20.00	0.00	0.00	0.000	0.000	0.0
	2	0910	2	2	0.16	0.20	-0.04		20.00	-0.24	-0.24	-0.028	-0.028	0.0
	3	0912	4	2	0.21	0.20	0.01		20.00	-0.03	-0.27	-0.003	-0.031	0.0
	4	0914	6	2	0.28	0.20	0.08		20.00	0.09	-0.18	0.010	-0.021	0.0
	5	0916	8	2	0.53	0.20	0.33		20.00	0.41	0.23	0.048	0.027	0.4
	6	0918	10	2	0.79	0.20	0.59	13.64	20.00	0.92	1.15	0.107	0.133	2.0
	7	0920	12	2	1.40	0.20	1.20		20.00	1.79	2.94	0.207	0.341	5.2
	8	0921	13	1	1.95	0.20	1.75		20.00	1.48	4.42	0.171	0.512	7.7
	9	0922	14	1	2.94	0.20	2.74		20.00	2.25	6.66	0.260	0.772	11.7
	10	0923	15	1	2.46	0.20	2.26		20.00	2.50	9.16	0.290	1.062	16.1
	11	0924	16	1	3.14	0.20	2.94		20.00	2.60	11.76	0.301	1.363	20.6
	12	0925	17	1	3.78	0.20	3.58		20.00	3.26	15.02	0.378	1.741	26.3
	13	0926	18	1	4.00	0.20	3.80		20.00	3.69	18.71	0.428	2.168	32.8
	14	0927	19	1	3.04	0.20	2.84	13.63	20.00	3.32	22.03	0.385	2.553	38.6
	15	0928	20	1	3.84	0.20	3.64		20.00	3.24	25.27	0.376	2.929	44.3
	16	0929	21	1	2.20	0.20	2.00		20.00	2.82	28.09	0.327	3.256	49.2
	17	0930	22	1	3.44	0.20	3.24		20.00	2.62	30.71	0.304	3.559	53.8
	18	0931	23	1	4.04	0.20	3.84		20.00	3.54	34.25	0.410	3.969	60.0
	19	0932	24	1	2.92	0.20	2.72		20.00	3.28	37.53	0.380	4.350	65.8
	20	0933	25	1	3.32	0.20	3.12		20.00	2.92	40.45	0.338	4.688	70.9
	21	0934	26	1	2.48	0.20	2.28		20.00	2.70	43.15	0.313	5.001	75.6
	22	0935	27	1	2.28	0.20	2.08		20.00	2.18	45.33	0.253	5.254	79.4
	23	0936	28	1	1.28	0.20	1.08	13.68	20.00	1.58	46.91	0.183	5.437	82.2
	24	0937	29	1	2.48	0.20	2.28		20.00	1.68	48.59	0.195	5.631	85.1
	25	0938	30	1	1.20	0.20	1.00		20.00	1.64	50.23	0.190	5.821	88.0
	26	0939	31	1	1.48	0.20	1.28		20.00	1.14	51.37	0.132	5.954	90.0
	27	0940	32	1	1.20	0.20	1.00		20.00	1.14	52.51	0.132	6.086	92.0
	28	0941	33	1	1.18	0.20	0.98		20.00	0.99	53.50	0.115	6.200	93.7
	29	0942	34	1	1.09	0.20	0.89		20.00	0.94	54.44	0.108	6.309	95.4
	30	0943	35	1	0.97	0.20	0.77		20.00	0.83	55.27	0.096	6.405	96.8
	31	0944	36	1	0.88	0.20	0.68		20.00	0.73	55.99	0.084	6.489	98.1
	32	0945	37	1	0.88	0.20	0.68	13.65	20.00	0.68	56.67	0.079	6.568	99.3
	33	0946	38	1	0.77	0.20	0.57		20.00	0.63	57.30	0.072	6.640	100.4
	34	0947	39	1	0.70	0.20	0.50		20.00	0.54	57.83	0.062	6.702	101.3
	35	0948	40	1	0.61	0.20	0.41		20.00	0.46	58.29	0.053	6.755	102.1
	36	0949	41	1	0.41	0.20	0.21		20.00	0.31	58.60	0.036	6.791	102.7
	37	0950	42	1	0.56	0.20	0.36		20.00	0.29	58.88	0.033	6.824	103.2
	38	0952	44	2	0.45	0.20	0.25		20.00	0.61	59.49	0.071	6.895	104.2
	39	0954	46	2	0.47	0.20	0.27	13.63	20.00	0.56	60.05	0.065	6.960	105.2
	40	0956	48	2	0.12	0.20	-0.08		20.00	0.45	60.50	0.052	7.012	106.0
	41	0958	50	2	0.22	0.20	0.02		20.00	0.47	60.97	0.054	7.066	106.8
	42	1000	52	2	0.03	0.20	-0.17		20.00	-0.15	60.82	-0.017	7.049	106.6
	43	1002	54	2	0.29	0.20	0.09		20.00	-0.08	60.74	-0.009	7.040	106.4
	44	1004	56	2	0.20	0.20	0.00	13.65	20.00	0.09	60.83	0.010	7.050	106.6
	45	1006	58	2	0.18	0.20	-0.02		20.00	-0.02	60.81	-0.002	7.048	106.6
	46	1008	60	2	0.07	0.20	-0.13		20.00	-0.15	60.66	-0.017	7.030	106.3
	47	1011	62	2	0.11	0.20	-0.09	13.64	20.00	-0.22	60.44	-0.025	7.005	105.9
	48	1014	64	2	0.09	0.20	-0.11		20.00	-0.20	60.24	-0.023	6.982	105.6
	49	1017	64	0	0.26	0.20	0.06		20.00	0.00	60.24	0.000	6.982	105.6
Average Values:								13.66	20.00					

T10 Detention Time From Above:	13	minutes	(Enter elapsed time at which 10% tracer recovery is achieved)
Average Test Flow Rate:	20.00	mgd	
Average Operating Depth:	13.66	feet	
Average Reservoir Volume:	0.362	MG	
Average DT:	26.06	minutes	
T10 / DT (baffling factor):	0.50		


20 MGD Tracer Test
Fluoride Concentration (mg/L)
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
-0.50
0 5 10 15 20 25 30 35 40 45 50 55
Time (minutes)


20 MGD Tracer Test
% Tracer Recovery
120.0
110.0
100.0
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
0 5 10 15 20 25 30 35 40 45 50 55 60 65
Time (minutes)

Grants Pass Tracer Study
June 24, 2003
10.5 MGD Slug Dose Tracer Test

Location:	Grant's Pass WTP
Test Date:	June 24, 2003
Unit(s) Evaluated:	0.433 MG Clearwell

Fluoride Data:

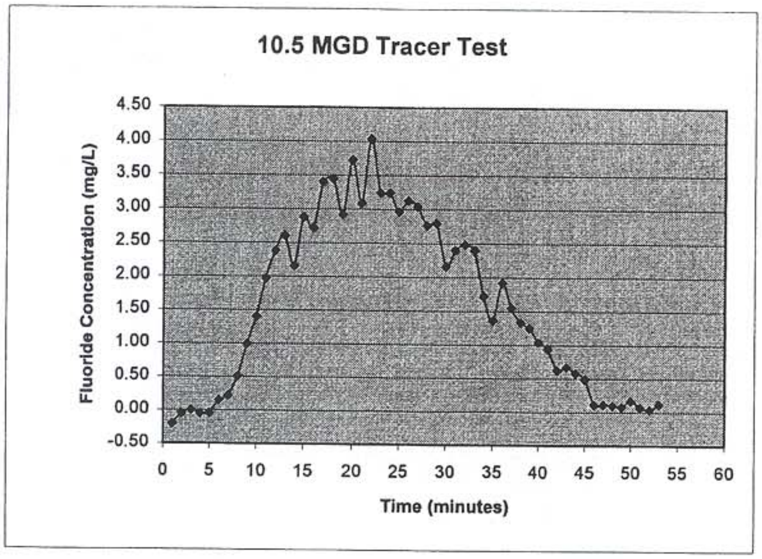
% Acid Used:	25	(From chemical supplier information)
Spec. Gravity:	1.234	(From chemical supplier information)
Available F, lbs/gallon:	2.0352	
Acid Added, gallons:	3.25	(Typically 7-8 gallons of hydrofluosilicic acid per MG of unit capacity evaluated)
Fluoride Ion Added, lbs:	6.6143	

Unit Data:

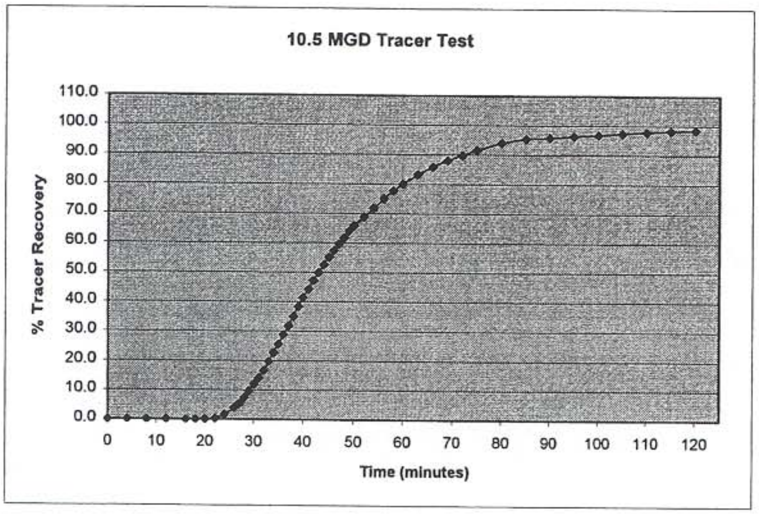
Maximum Volume, MG:	0.433	
Maximum SWD, feet:	14.5	
Volume per foot depth, MG	0.0299	(Only valid if floor elevation is consistent for entire unit being tested)

Sample #	Clock Time	Time in Minutes Elapsed	Time in Minutes Increment	Fluoride Concentration, mg/L Sample	Fluoride Concentration, mg/L ackground	Fluoride Concentration, mg/L Tracer	Reservoir Depth, ft	Flow, mgd	Tracer, mg-min/L Increment	Tracer, mg-min/L umulative	Tracer Recovered, lbs Increment	Tracer Recovered, lbs umulative	Tracer ecovery, %
(Tracer Addition Time)	1335	0	0	0.00	0.20	-0.20	13.57	10.50	0.00	0.00	0.000	0.000	0.0
1	1339	4	4	0.15	0.20	-0.05		10.50	-0.50	-0.50	-0.030	-0.030	0.0
2	1343	8	4	0.20	0.20	0.00		10.50	-0.10	-0.60	-0.006	-0.037	0.0
3	1347	12	4	0.15	0.20	-0.05		10.50	-0.10	-0.70	-0.006	-0.043	0.0
4	1351	16	4	0.15	0.20	-0.05		10.50	-0.20	-0.90	-0.012	-0.055	0.0
5	1353	18	2	0.35	0.20	0.15		10.50	0.10	-0.80	0.006	-0.049	0.0
6	1355	20	2	0.42	0.20	0.22		10.50	0.37	-0.43	0.023	-0.026	0.0
7	1357	22	2	0.71	0.20	0.51	13.53	10.50	0.73	0.30	0.044	0.018	0.3
8	1359	24	2	1.19	0.20	0.99		10.50	1.50	1.80	0.091	0.110	1.7
9	1401	26	2	1.60	0.20	1.40		10.50	2.39	4.19	0.145	0.255	3.9
10	1402	27	1	2.17	0.20	1.97		10.50	1.69	5.88	0.103	0.357	5.4
11	1403	28	1	2.58	0.20	2.38		10.50	2.18	8.05	0.132	0.490	7.4
12	1404	29	1	2.80	0.20	2.60		10.50	2.49	10.54	0.152	0.641	9.7
13	1405	30	1	2.36	0.20	2.16		10.50	2.38	12.92	0.145	0.786	11.9
14	1406	31	1	3.08	0.20	2.88		10.50	2.52	15.44	0.153	0.939	14.2
15	1407	32	1	2.92	0.20	2.72		10.50	2.80	18.24	0.170	1.110	16.8
16	1408	33	1	3.60	0.20	3.40		10.50	3.06	21.30	0.186	1.296	19.6
17	1409	34	1	3.64	0.20	3.44		10.50	3.42	24.72	0.208	1.504	22.7
18	1410	35	1	3.12	0.20	2.92		10.50	3.18	27.90	0.193	1.698	25.7
19	1411	36	1	3.92	0.20	3.72		10.50	3.32	31.22	0.202	1.900	28.7
20	1412	37	1	3.28	0.20	3.08		10.50	3.40	34.62	0.207	2.106	31.8
21	1413	38	1	4.24	0.20	4.04		10.50	3.56	38.18	0.217	2.323	35.1
22	1414	39	1	3.44	0.20	3.24		10.50	3.64	41.82	0.221	2.545	38.5
23	1415	40	1	3.44	0.20	3.24		10.50	3.24	45.06	0.197	2.742	41.5
24	1416	41	1	3.16	0.20	2.96		10.50	3.10	48.16	0.189	2.930	44.3
25	1417	42	1	3.32	0.20	3.12		10.50	3.04	51.20	0.185	3.115	47.1
26	1418	43	1	3.24	0.20	3.04		10.50	3.08	54.28	0.187	3.303	49.9
27	1419	44	1	2.96	0.20	2.76		10.50	2.90	57.18	0.176	3.479	52.6
28	1420	45	1	3.00	0.20	2.80		10.50	2.78	59.96	0.169	3.648	55.2
29	1421	46	1	2.36	0.20	2.16		10.50	2.48	62.44	0.151	3.799	57.4
30	1422	47	1	2.60	0.20	2.40		10.50	2.28	64.72	0.139	3.938	59.5
31	1423	48	1	2.68	0.20	2.48		10.50	2.44	67.16	0.148	4.086	61.8
32	1424	49	1	2.60	0.20	2.40	13.62	10.50	2.44	69.60	0.148	4.235	64.0
33	1425	50	1	1.92	0.20	1.72		10.50	2.06	71.66	0.125	4.360	65.9
34	1427	52	2	1.56	0.20	1.36		10.50	3.08	74.74	0.187	4.548	68.8
35	1429	54	2	2.12	0.20	1.92		10.50	3.28	78.02	0.200	4.747	71.8
36	1431	56	2	1.75	0.20	1.55		10.50	3.47	81.49	0.211	4.958	75.0
37	1433	58	2	1.53	0.20	1.33		10.50	2.88	84.37	0.175	5.134	77.6
38	1435	60	2	1.45	0.20	1.25		10.50	2.58	86.95	0.157	5.291	80.0
39	1438	63	3	1.24	0.20	1.04		10.50	3.44	90.39	0.209	5.500	83.1
40	1441	66	3	1.14	0.20	0.94		10.50	2.97	93.36	0.181	5.680	85.9
41	1444	69	3	0.82	0.20	0.62		10.50	2.34	95.70	0.142	5.823	88.0
42	1447	72	3	0.87	0.20	0.67		10.50	1.94	97.63	0.118	5.940	89.8
43	1450	75	3	0.78	0.20	0.58	13.67	10.50	1.88	99.51	0.114	6.054	91.5
44	1455	80	5	0.69	0.20	0.49		10.50	2.68	102.18	0.163	6.217	94.0
45	1500	85	5	0.31	0.20	0.11		10.50	1.50	103.68	0.091	6.308	95.4
46	1505	90	5	0.31	0.20	0.11		10.50	0.55	104.23	0.033	6.342	95.9
47	1510	95	5	0.30	0.20	0.10		10.50	0.53	104.76	0.032	6.374	96.4
48	1515	100	5	0.28	0.20	0.08		10.50	0.45	105.21	0.027	6.401	96.8
49	1520	105	5	0.37	0.20	0.17	13.62	10.50	0.63	105.83	0.038	6.439	97.4
50	1525	110	5	0.27	0.20	0.07		10.50	0.60	106.43	0.037	6.476	97.9
51	1530	115	5	0.24	0.20	0.04		10.50	0.28	106.71	0.017	6.493	98.2
52	1535	120	5	0.32	0.20	0.12		10.50	0.40	107.11	0.024	6.517	98.5
Average Values:							13.60	10.50					

T10 Detention Time From Above:	30	minutes	(Enter elapsed time at which 10% tracer recovery is achieved)
Average Test Flow Rate:	10.50	mgd	
Average Operating Depth:	13.60	feet	
Average Reservoir Volume:	0.362	MG	
Average DT:	49.65	minutes	
T10 / DT (baffling factor):	0.60		



10.5 MGD Tracer Test
Fluoride Concentration (mg/L)
Time (minutes)



10.5 MGD Tracer Test
% Tracer Recovery
Time (minutes)

BLACK & VEATCH

MEMORANDUM **Black & Veatch Corporation**

To:	Jason Canady Water Treatment Plant Supervisor	B&V Project No.: 135540
From:	Gary Logsdon, D.Sc., P.E. Robert Ward, P.E. Michelle Cheek, E.I.T.	B&V File: B 5.0
Date:	July 9, 2003	
RE:	Grants Pass Filter Evaluation – Final Report	

On May 28, 2003 Black and Veatch performed a filter investigation and evaluation at the City of Grants Pass Water Filtration Plant. The primary purpose of the filter investigation was to determine the effectiveness of the filter backwashing process which affects filter performance. The following two methods were used to evaluate filter backwashing:

1. Core sampling of Filter No. 3 both before and after filter backwash – Two core samples were taken from the filter bed before backwash and one core sample was taken after backwash. Layers of the core samples were then tested for the amount of turbidity that could be removed from a measured amount of filter media by shaking the media in a measured amount of water. The difference in turbidity removed from the media before and after backwash represents the amount of floc that was removed during backwash and thus indicates the effectiveness of the backwash process.
2. Measuring the turbidity at regular intervals during the backwashing process -- Upon initiating the backwash process, the turbidity was measured at time zero and at 30 second intervals for the first five minutes of backwash, then at 60 second intervals up to 15 minutes of backwashing, and finally at the end of the backwash cycle after 20 minutes of backwashing.

The following three tables and graph present the results of the core sampling process. Each table includes the core location, the depth profiled, and the turbidity (per 100 mL) of each depth profiled.

FILTER #3

Pre-backwash Core Samples

Core #1 – Between Two North Sweeps		
Depth Profiled (inches)	**Sample Size (mL)**	**Turbidity (per 100 mL)**
0-2	50	232
2-6	50	186
6-12	50	258
12-18	50	208
18-22*	50	178

Core #2 – NW Corner		
Depth Profiled (inches)	**Sample Size (mL)**	**Turbidity (per 100 mL)**
0-2	50	222
2-6	50	228
6-12	50	204
12-18	50	130
18-20**	35	158

*22" – Top of Gravel Layer
**20" – Top of Gravel Layer

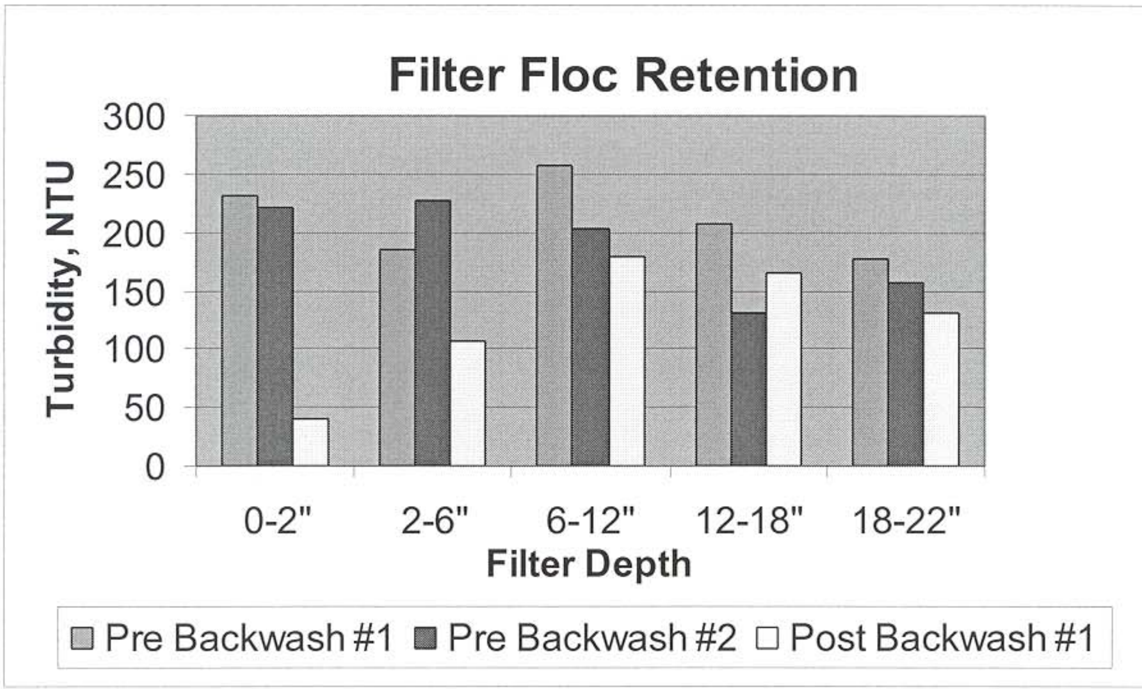
Post-backwash Core Sample

Core #3 – Between Two North Sweeps		
Depth Profiled (inches)	**Sample Size (mL)**	**Turbidity (per 100 mL)**
0-2	50	40
2-6	50	106
6-12	50	180
12-18	50	166
18-22"*	50	130

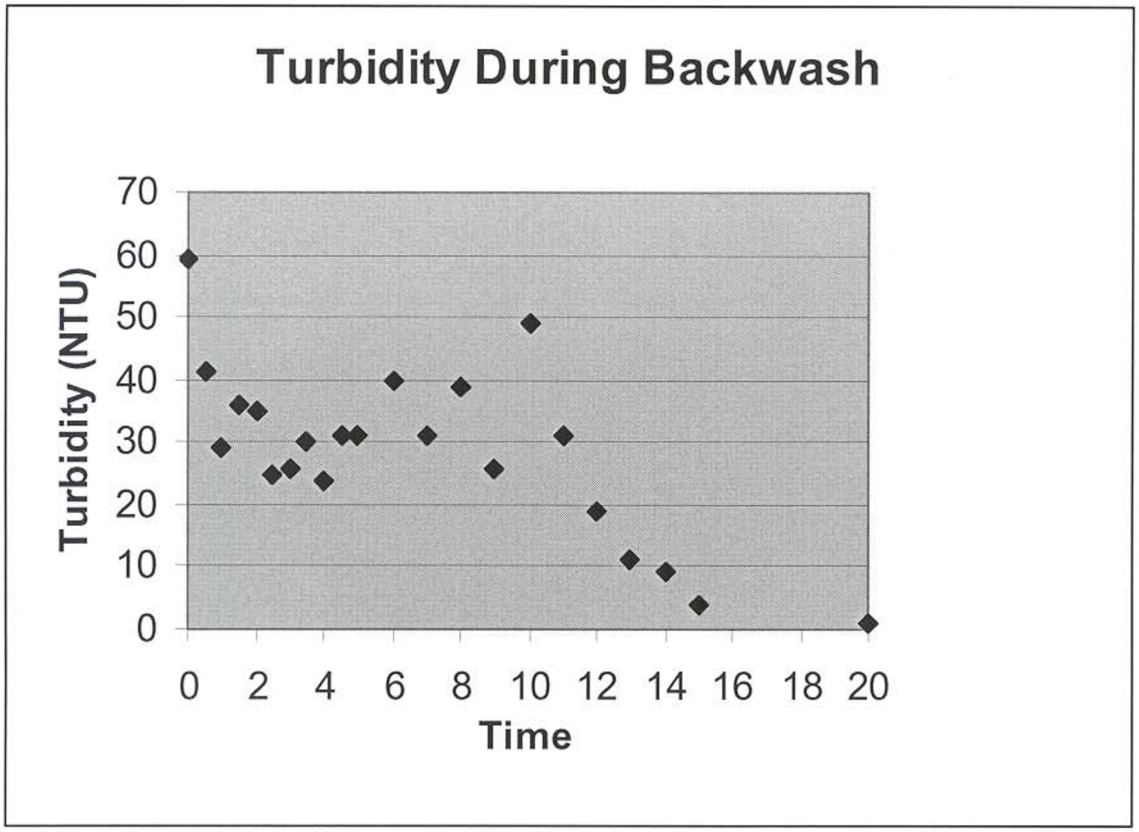
*22" – Top of Gravel Layer

The results of the core sampling and turbidity testing for Location #1 show that filter backwashing caused a substantial reduction in the turbidity that could be shaken off the media in the top 2 inches of the filter, and a considerable reduction in turbidity in the 2-inch to 6-inch layer. Below 6 inches most of the turbidity-causing floc remained on the media even though the media had been backwashed. These results indicate that floc is not sufficiently removed from the filter bed during the backwash process, especially at filter depths from 6 inches to the bottom of the bed. This is believed to be caused by inadequate bed expansion during the backwash cycle. When a filter is not adequately cleaned during backwash, mudballs can form and clog the filter bed. These clogged areas will then contract as head loss increases and lead to cracks in the filter media. Cracks in the filter media then cause short-circuiting during filtration which leads to a decline in filtered water quality.

The core sampling also indicated that the Ilmenite sand layer, in theory located between the gravels and Silica Sand layer, no longer exists as a distinct, identifiable layer within the filter bed. The absence of a distinct Ilmenite layer may cause a decline in overall filtered water quality if the Ilmenite has been lost from the bed. Alternatively, Ilmenite might be intermixed through sand and coal in the bed. If this has happened, some of the filtering benefit of the fine layer of Ilmenite would still be obtained although the intermixing would tend to cause higher clean bed headloss.



The following graph presents the results of the turbidity testing during the backwash cycle. The filter was washed at a low backwash rate for the first 8.5 minutes. At approximately 8.5 minutes, the backwash rate was increased to the maximum rate and the surface wash was started. At approximately 13 minutes, the surface wash was turned off and the maximum backwash rate continued for the remainder of the backwash cycle. Although the turbidity at the end of the backwash cycle is low, there is not a significant peak in the turbidity during the backwash cycle. This is typically indicative of ineffective backwashing or of backwashing a filter with only a small amount of floc accumulated in the bed. The bed expansion appeared to be minimal (perhaps only up to the surface sweeps) so in this instance the backwashing is considered to be ineffective.



Recommendations:

The AwwaRF Report #90908, Filter Maintenance and Operations Guidance Manual, which was ordered by the City of Grants Pass, shows a device that is used to measure filter bed expansion during backwash. With the photos of the bed expansion tool as a guide, it is recommended that staff at the plant construct this device and then use it to measure the bed expansion of all filters during backwash, with surface sweeps off.

Water Quality & Treatment, 5th Ed. recommends that filter bed expansion should be between 15% and 30%. Based on backwashing observed on May 28, we recommend an expansion of 25% to 30%. For 20 inches of media, this would be 5 to 6 inches of bed expansion. If this amount of expansion can not be attained, a higher rate of flow for backwashing is recommended. This recommendation applies to all filters including Filter No. 3.

When bed expansion during backwashing is adequate, backwashing filters until the spent washwater turbidity is about 10 ntu should be sufficient. Little additional benefit is gained by backwashing when only a small amount of additional floc is being removed by washing. After turbidity of spent washwater is as low as 10 ntu, not much can be gained by further washing and the return on the finished water used for filter washing is then minimal.

Conduct the core sampling process and turbidity measurements during backwash for all filters to determine the backwash effectiveness for each filter. Make adjustments to the backwashing process on all filters as described above to ensure adequate bed expansion, and spent washwater turbidity is approximately 10 ntu.

ANTHRATECH U.S.
9500 South 500 West, Suite 210
Sandy, Utah 84070
PHONE/FAX (801) 566-1700/(801) 566-1722

AWI leaders in FILTER OPTIMIZATION

Note:

TO: Jude Daniel Grounds

DATE: August 21, 2003

COMPANY: Montgomery Watson Harza

TOTAL PAGES: 15

FAX NO: 503-226-0023

COPIES:

FROM: Tad Bassett

SUBJECT: Sieve Analysis Summary for Grants Pass

Mr. Grounds:

Attached are the sieve results for the samples received at our facility on August 7. The table below provides a summary of the results.

Sample	Anthracite			Sand			Gravel
	ES (mm)	UC	Wt. %	ES (mm)	UC	Wt. %	Wt. %
Filter 1 Anthracite	1.03	1.37	96.5	0.43	1.32	3.5	n/a
Filter 1 Sand	0.26	5.54	23.6	0.54	1.29	76.4	n/a
Filter 5 Anthracite	1.06	1.29	94.4	0.43	1.27	5.6	n/a
Filter 5 Sand	1.09	1.33	12.1	0.54	1.32	87.6	0.3
Filter 7 Anthracite	1.15	1.35	96.6	0.43	1.20	3.4	n/a
Filter 7 Sand	1.23	1.41	56.1	0.46	1.21	43.9	n/a
Replacement Anthracite	1.08	1.31	100.0	n/a	n/a	n/a	n/a

It is difficult to draw conclusions about the results without knowing the original specification for the media that was placed in the filters; however, it appears that the sand may not have been skimmed prior to placement of the anthracite.

There was a great deal of fine black material which appeared to be anthracite, in the sand sample from filter 1. It is possible that this fine material may not be anthracite, or the presence of these fines could indicate that the sample collection point is not being backwashed adequately.

The anthracite in filter 7 clearly has a larger effective size than filters 1 and 5. This explains the greater degree of intermixing of anthracite in the sand sample from filter 7.

Please feel free to contact us at (801) 566-1700 if you have questions or comments regarding the sieve analysis results. A copy of these results along with the specific gravity results from filter 5 will be sent to you by mail sometime next week.

Tad

AWI
... leaders in
FILTER OPTIMIZATION

9500 South 500 West, Suite 210
Sandy, Utah 84070
(801) 566-1700 Fax (801) 566-1722

INVOICE

INVOICE NO: 1465
DATE: 8/19/03

Sold To:
Montgomery Watson Harza
111 S.W. 5th Avenue, Suite 1770
Portland, OR 97204-3604

Ship To:
Grants Pass, Oregon
111 S.W. 5th Avenue, Suite 1770
Portland, OR 97204-3604

SALESPERSON	P.O. NUMBER	DATE SHIPPED	SHIPPED VIA	F.O.B. POINT	TERMS
Austin	Ltr of 8/6/03	8/19/03	U.S. Mail	SLC, UT	Net 30

QUANTITY	DESCRIPTION	UNIT PRICE	AMOUNT
4 Each	Anthracite Analysis – Effective Size & Uniformity Coefficient	$55.00	$220.00
3 Each	Sand Analysis – Effective Size & Uniformity Coefficient	$55.00	$165.00
1 Each	Anthracite Anaylsis – Apparent Specific Gravity	$75.00	$75.00
1 Each	Sand Anaylsis – Apparent Specific Gravity	$75.00	$75.00
		SUBTOTAL	$535.00
		SALES TAX	$0.00
		SHIPPING & HANDLING	$0.00
		TOTAL DUE	$535.00

Make all checks payable to: Anthratech US Inc.
If you have any questions concerning this invoice, call: (801) 566-1700.

THANK YOU FOR YOUR BUSINESS!

testing engineers international

AMEC EARTH & ENVIRONMENTAL
4137 SOUTH 500 WEST
SALT LAKE CITY UT 84123
USA

Analysis No.	TS-A-01056
Report Date	18 August 2003
Date Sampled	11 August 2003
Time Sampled	17:15
Where	AWI Anthracite
Sampled By	Client
Phone	(801) 266-0720
Facsimile	(801) 266-0727

This is to certify that we have examined samples identified: AWI Anthracite, Job #1-819-001524

and found when tested to the requirements of:

AWWA B100-96 "AWWA Standard for Filtering Material" Section 5.3.3 Specific Gravity

Apparent Specific Gravity	
Sample Identification	Results
3650-2 Anthracite Analysis Per AWWA B100	1.43

True Specific Gravity	
Sample Identification	Results
3650-2 Anthracite Analysis Per AWWA B100	1.49

Porosity	
Sample Identification	Results
3650-2 Anthracite Analysis Per AWWA B100	4.03

USEPA Laboratory I.D. U00930

Rocky Mitchell Baum, M.S. – Director Analytical Testing amec

PROJECT: AWI - FILTER OPTIMIZATION
LOCATION: AS RECEIVED
MATERIAL: SILICA SAND
SAMPLE SOURCE GRANTS PASS, OR

JOB NO: 1-815-001524
WORK ORDER NO: T 3650
LAB NO:
DATE SAMPLED: 08/11/03

SPECIFIC GRAVITY AND ABSORPTION OF FINE AGGREGATE (ASTM C128)

WEIGHT OF S.S.D. AGGREGATE (g)	500.1
WEIGHT OF FLASK & WATER (g)	1468.6
WEIGHT OF OVEN DRY SAMPLE IN AIR (g)	495.3
WEIGHT OF FLASK & WATER & S.S.D. (g)	1775.9
BULK SPECIFIC GRAVITY	2.569
BULK SPECIFIC GRAVITY (S.S.D.)	2.594
APPARENT SPECIFIC GRAVITY	2.636
ABSORPTION (%)	0.99

Reviewed By: Juan Madrigal

AWI ...leaders in FILTER OPTIMIZATION

Client MWH

Date 11-Aug-03

Job Name

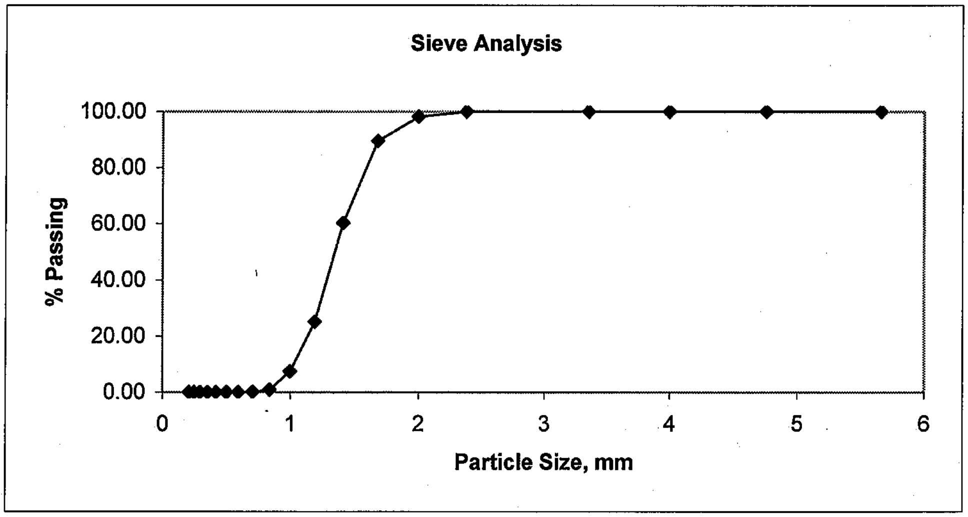
Filter # Filter 1 Anthracite

Anthracite

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38	0.20	0.05	0.05	99.95
10	2	7.70	1.79	1.84	98.16
12	1.68	37.40	8.70	10.53	89.47
14	1.41	125.69	29.23	39.77	60.23
16	1.19	151.09	35.14	74.90	25.10
18	1	75.90	17.65	92.55	7.45
20	0.841	27.90	6.49	99.04	0.96
25	0.707	3.59	0.83	99.88	0.12
30	0.595	0.43	0.10	99.97	0.03
35	0.5	0.11	0.03	100.00	0.00
40	0.42		0.00	100.00	0.00
45	0.354		0.00	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		430.01			

D10 =	1.03	Effective Size	1.03
D50 =	1.35	Mean Size	1.35
D60 =	1.41	Uniformity Coefficient	1.37
D90 =	1.70	Backwash rate GPM/ft2	13.6829



Client MWH

Job Name

Filter # Filter 1 Anthracite

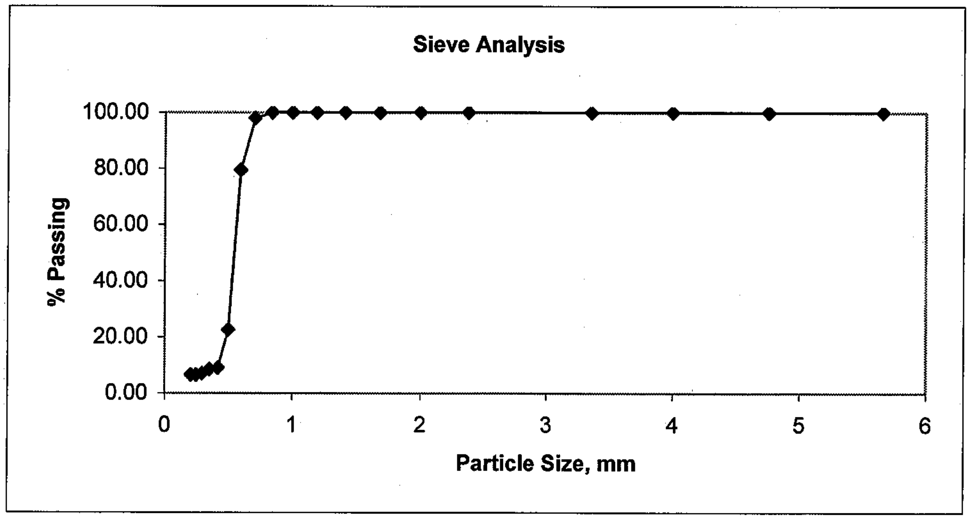
Sample Location

Date 11-Aug-03

Sand

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1		0.00	0.00	100.00
20	0.841		0.00	0.00	100.00
25	0.707	0.31	2.02	2.02	97.98
30	0.595	2.87	18.57	20.58	79.42
35	0.5	8.79	56.80	77.38	22.62
40	0.42	2.10	13.57	90.95	9.05
45	0.354	0.10	0.65	91.60	8.40
50	0.297	0.20	1.29	92.89	7.11
60	0.25	0.10	0.65	93.54	6.46
70	0.21		0.00	93.54	6.46
Pan	#N/A	1.00	6.46	100.00	0.00
		15.48			

D10 =	0.43	Effective Size	0.43
D50 =	0.55	Mean Size	0.55
D60 =	0.56	Uniformity Coefficient	1.32
D90 =	0.66	Backwash rate GPM/ft2	10.4232



AWI ...leaders in FILTER OPTIMIZATION

Client MWH

Date 11-Aug-03

Job Name

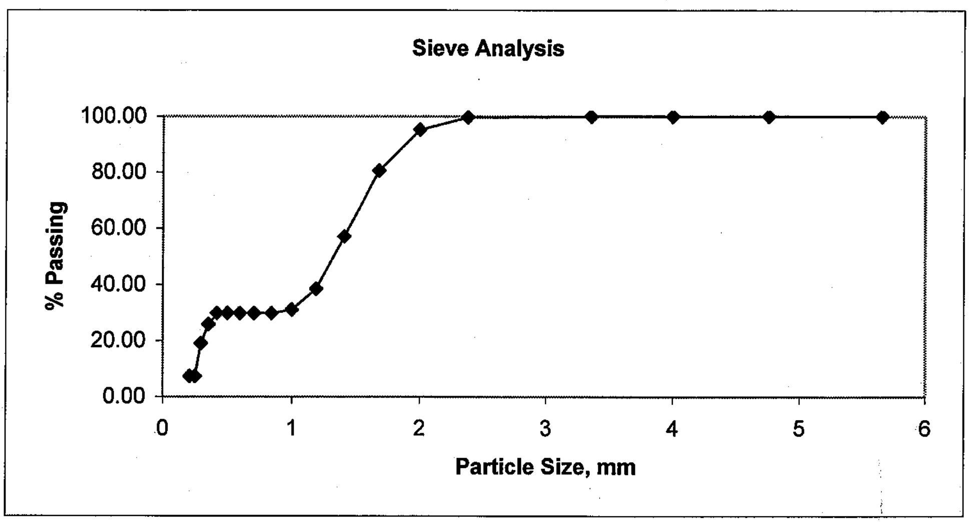
Filter # Filter 1 Sand

Anthracite

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38	0.60	0.36	0.36	99.64
10	2	7.30	4.40	4.76	95.24
12	1.68	24.40	14.70	19.46	80.54
14	1.41	38.90	23.43	42.89	57.11
16	1.19	31.00	18.68	61.57	38.43
18	1	12.30	7.41	68.98	31.02
20	0.841	2.10	1.26	70.24	29.76
25	0.707		0.00	70.24	29.76
30	0.595		0.00	70.24	29.76
35	0.5		0.00	70.24	29.76
40	0.42		0.00	70.24	29.76
45	0.354	6.50	3.92	74.16	25.84
50	0.297	11.20	6.75	80.90	19.10
60	0.25	19.50	11.75	92.65	7.35
70	0.21		0.00	92.65	7.35
Pan	#N/A	12.20	7.35	100.00	0.00
		165.99			

D10 =	0.26	Effective Size	0.26
D50 =	1.33	Mean Size	1.33
D60 =	1.44	Uniformity Coefficient	5.54
D90 =	1.89	Backwash rate GPM/ft2	14.0209



AWI
Leaders in
FILTER OPTIMIZATION

Client MWH **Date** 11-Aug-03

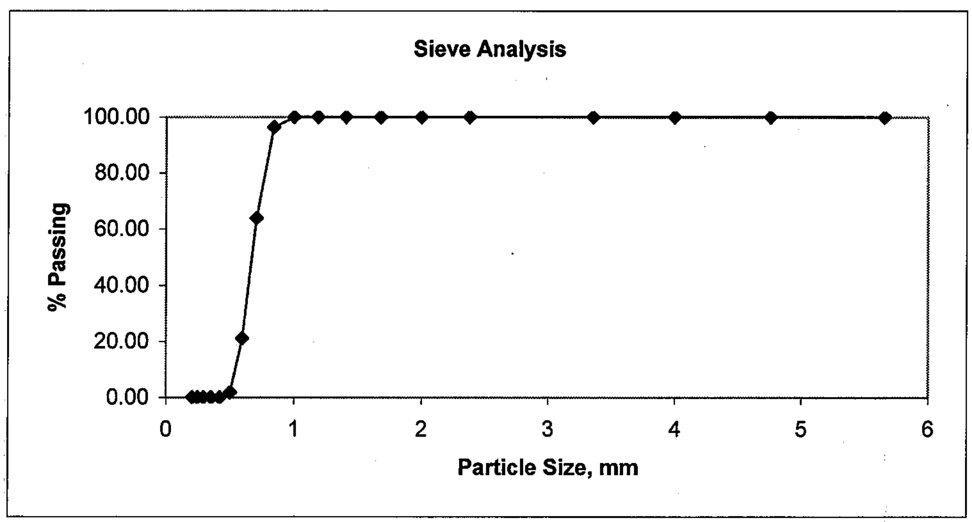
Job Name

Filter # Filter 1 Sand **Sand**

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1		0.00	0.00	100.00
20	0.841	19.90	3.70	3.70	96.30
25	0.707	174.20	32.36	36.06	63.94
30	0.595	230.80	42.88	78.93	21.07
35	0.5	103.30	19.19	98.12	1.88
40	0.42	10.10	1.88	100.00	0.00
45	0.354		0.00	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		538.30			

D10 =	0.54	Effective Size	0.54
D50 =	0.67	Mean Size	0.67
D60 =	0.70	Uniformity Coefficient	1.29
D90 =	0.81	Backwash rate GPM/ft2	12.9095



Client MWH **Date** 11-Aug-03

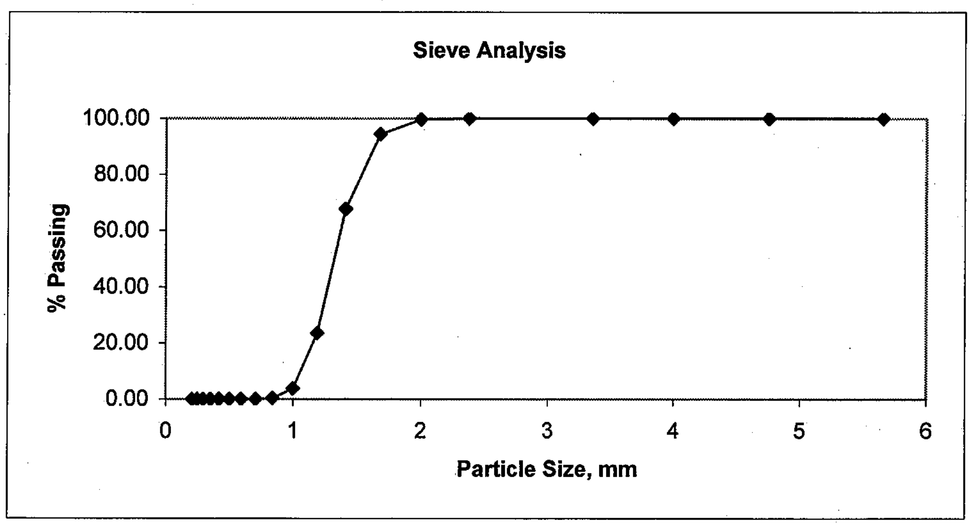
Job Name

Filter # Filter 5 Anthracite **Anthracite**

Sample Location

Mesh	Sieve, mm	Retained, g	% Retained	Cumulative % Retained	Cumulative % Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2	1.20	0.33	0.33	99.67
12	1.68	18.80	5.24	5.58	94.42
14	1.41	95.80	26.72	32.30	67.70
16	1.19	158.29	44.15	76.45	23.55
18	1	70.90	19.77	96.22	3.78
20	0.841	12.40	3.46	99.68	0.32
25	0.707	1.02	0.29	99.97	0.03
30	0.595	0.12	0.03	100.00	0.00
35	0.5		0.00	100.00	0.00
40	0.42		0.00	100.00	0.00
45	0.354		0.00	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		358.52			

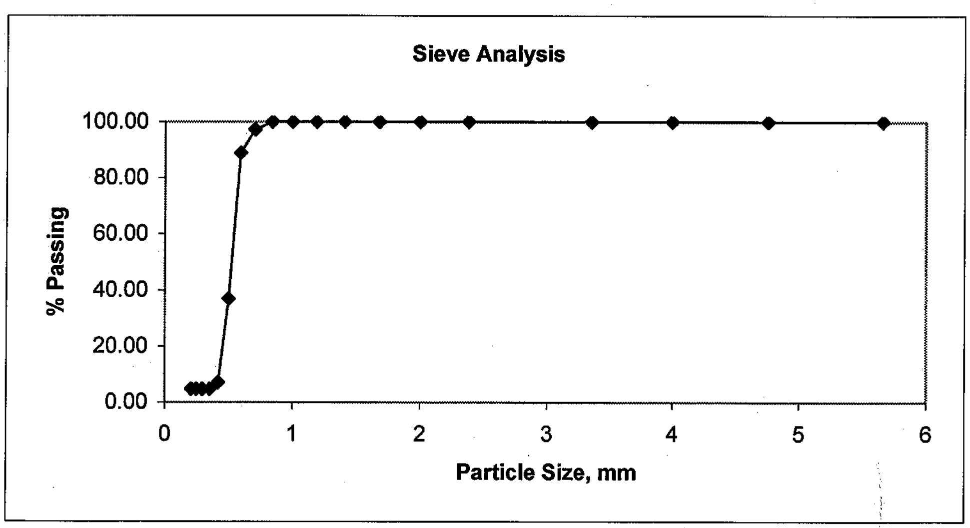
D10 =	1.06	Effective Size	1.06
D50 =	1.32	Mean Size	1.32
D60 =	1.37	Uniformity Coefficient	1.29
D90 =	1.64	Backwash rate GPM/ft2	13.3243



Client	**MWH**	**Date**	11-Aug-03
Job Name			
Filter #	**Filter 5 Anthracite**		**Sand**
Sample Location			

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1		0.00	0.00	100.00
20	0.841		0.00	0.00	100.00
25	0.707	0.58	2.73	2.73	97.27
30	0.595	1.78	8.43	11.16	88.84
35	0.5	11.00	51.98	63.14	36.86
40	0.42	6.30	29.77	92.91	7.09
45	0.354	0.50	2.36	95.27	4.73
50	0.297		0.00	95.27	4.73
60	0.25		0.00	95.27	4.73
70	0.21		0.00	95.27	4.73
Pan	#N/A	1.00	4.73	100.00	0.00
		21.16			

D10 =	0.43	Effective Size	0.43
D50 =	0.52	Mean Size	0.52
D60 =	0.54	Uniformity Coefficient	1.27
D90 =	0.61	Backwash rate GPM/ft2	10.0483



Client MWH

Date 11-Aug-03

Job Name

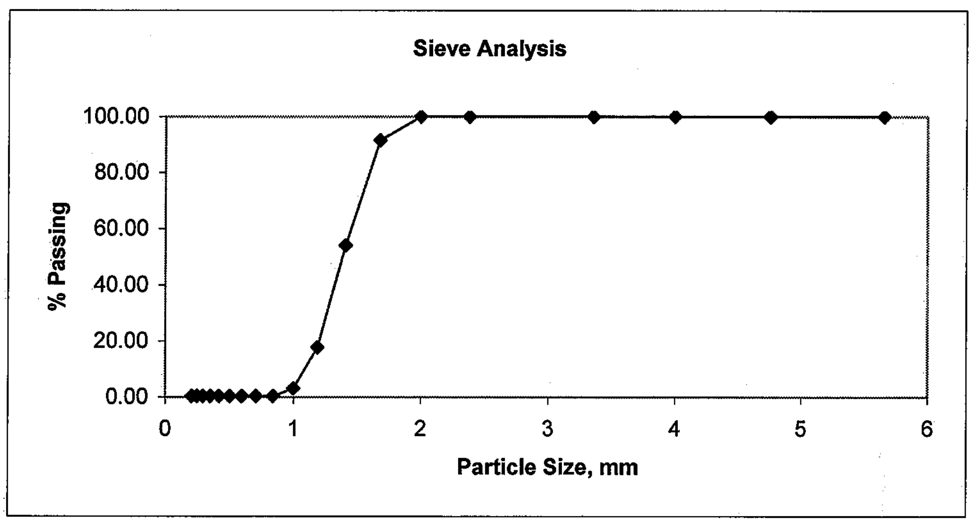
Filter # Filter 5 Sand

Anthracite

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68	7.80	8.49	8.49	91.51
14	1.41	34.50	37.56	46.05	53.95
16	1.19	33.30	36.25	82.30	17.70
18	1	13.43	14.62	96.92	3.08
20	0.841	2.53	2.75	99.67	0.33
25	0.707		0.00	99.67	0.33
30	0.595		0.00	99.67	0.33
35	0.5		0.00	99.67	0.33
40	0.42		0.00	99.67	0.33
45	0.354		0.00	99.67	0.33
50	0.297		0.00	99.67	0.33
60	0.25		0.00	99.67	0.33
70	0.21		0.00	99.67	0.33
Pan	#N/A	0.30	0.33	100.00	0.00
		91.86			

D10 =	1.09	Effective Size	1.09
D50 =	1.39	Mean Size	1.39
D60 =	1.45	Uniformity Coefficient	1.33
D90 =	1.67	Backwash rate GPM/ft2	14.1195



AWI leaders in FILTER OPTIMIZATION

Client MWH **Date** 11-Aug-03

Job Name

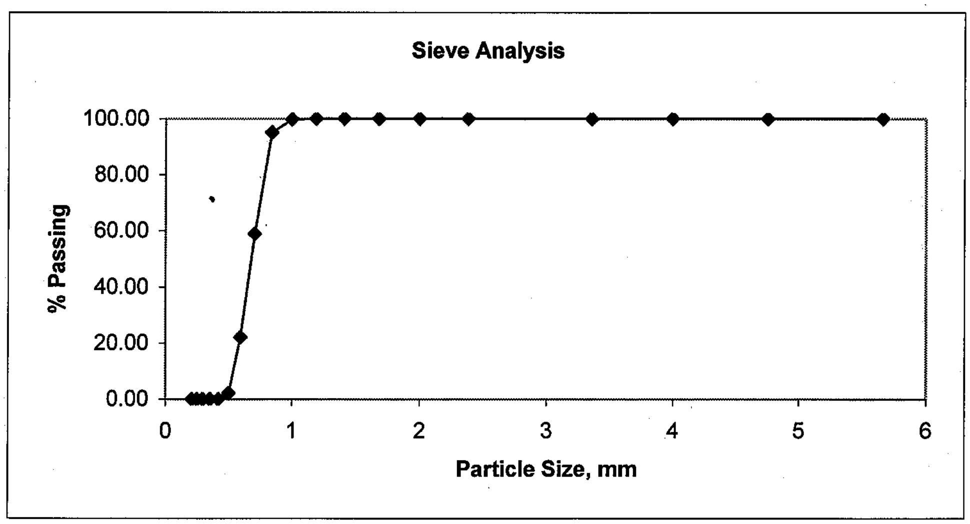
Filter # Filter 5 Sand

Sample Location

Sand

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1	1.17	0.18	0.18	99.82
20	0.841	31.27	4.70	4.87	95.13
25	0.707	241.60	36.28	41.15	58.85
30	0.595	245.00	36.79	77.94	22.06
35	0.5	133.30	20.02	97.96	2.04
40	0.42	13.40	2.01	99.97	0.03
45	0.354	0.20	0.03	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		665.94			

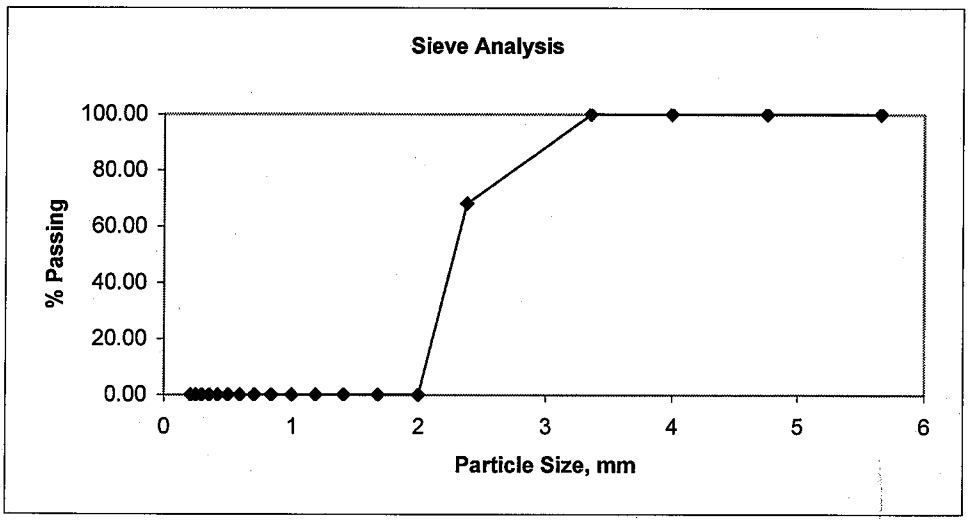
D10 =	0.54	Effective Size	0.54
D50 =	0.68	Mean Size	0.68
D60 =	0.71	Uniformity Coefficient	1.32
D90 =	0.82	Backwash rate GPM/ft2	13.1790



Client	0	Date	11-Aug-03
Job Name	0		
Filter #	Filter 5 Sand		Gravel
Sample Location	0		

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38	0.70	31.82	31.82	68.18
10	2	1.50	68.18	100.00	0.00
12	1.68		0.00	100.00	0.00
14	1.41		0.00	100.00	0.00
16	1.19		0.00	100.00	0.00
18	1		0.00	100.00	0.00
20	0.841		0.00	100.00	0.00
25	0.707		0.00	100.00	0.00
30	0.595		0.00	100.00	0.00
35	0.5		0.00	100.00	0.00
40	0.42		0.00	100.00	0.00
45	0.354		0.00	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		2.20			

D10 =	2.06	Effective Size	2.06
D50 =	2.28	Mean Size	2.28
D60 =	2.33	Uniformity Coefficient	1.14
D90 =	3.05	Backwash rate GPM/ft2	63.4957



Client MWH **Date** 11-Aug-03

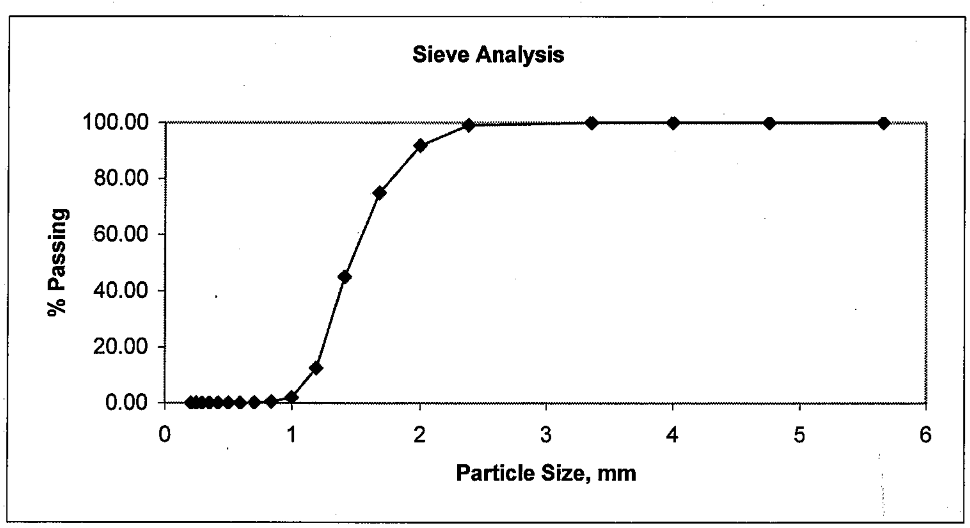
Job Name

Filter # 7 - Anthracite **Anthracite**

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38	3.30	0.95	0.95	99.05
10	2	25.20	7.28	8.24	91.76
12	1.68	58.50	16.90	25.14	74.86
14	1.41	103.50	29.91	55.05	44.95
16	1.19	112.30	32.45	87.50	12.50
18	1	36.60	10.58	98.08	1.92
20	0.841	5.20	1.50	99.58	0.42
25	0.707	1.20	0.35	99.93	0.07
30	0.595	0.23	0.07	99.99	0.01
35	0.5	0.03	0.01	100.00	0.00
40	0.42		0.00	100.00	0.00
45	0.354		0.00	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		346.04			

D10 =	1.15	Effective Size	1.15
D50 =	1.46	Mean Size	1.46
D60 =	1.55	Uniformity Coefficient	1.35
D90 =	1.97	Backwash rate GPM/ft2	15.0169



AWI ...leaders in FILTER OPTIMIZATION

Client MWH **Date** 11-Aug-03

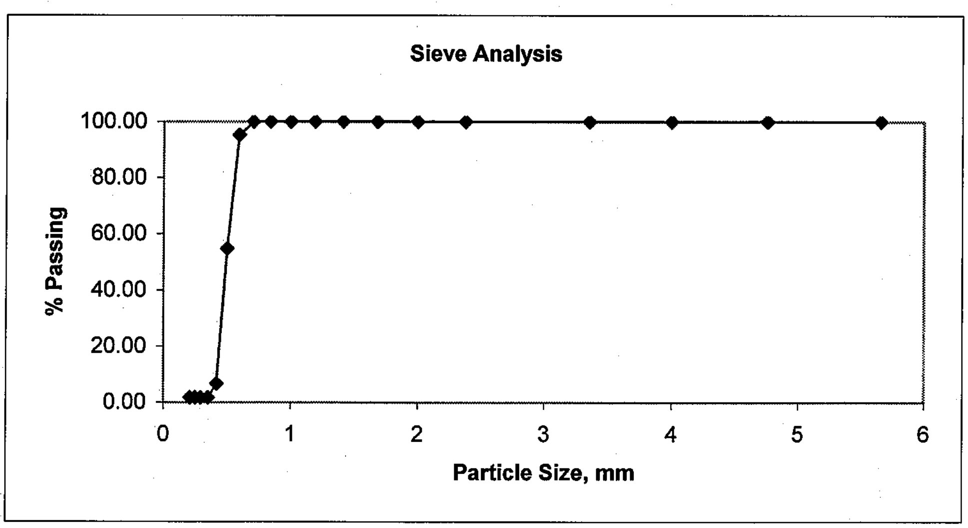
Job Name

Filter # 7 - Anthracite **Sand**

Sample Location

Mesh	Sieve, mm	Retained, g	% Retained	Cumulative % Retained	Cumulative % Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1		0.00	0.00	100.00
20	0.841		0.00	0.00	100.00
25	0.707		0.00	0.00	100.00
30	0.595	0.57	4.77	4.77	95.23
35	0.5	4.87	40.43	45.20	54.80
40	0.42	5.80	48.16	93.36	6.64
45	0.354	0.60	4.98	98.34	1.66
50	0.297		0.00	98.34	1.66
60	0.25		0.00	98.34	1.66
70	0.21		0.00	98.34	1.66
Pan	#N/A	0.20	1.66	100.00	0.00
		12.04			

D10 =	0.43	Effective Size	0.43
D50 =	0.49	Mean Size	0.49
D60 =	0.51	Uniformity Coefficient	1.20
D90 =	0.58	Backwash rate GPM/ft2	9.4911



AWI ...leaders in FILTER OPTIMIZATION

Client MWH

Date 11-Aug-03

Job Name

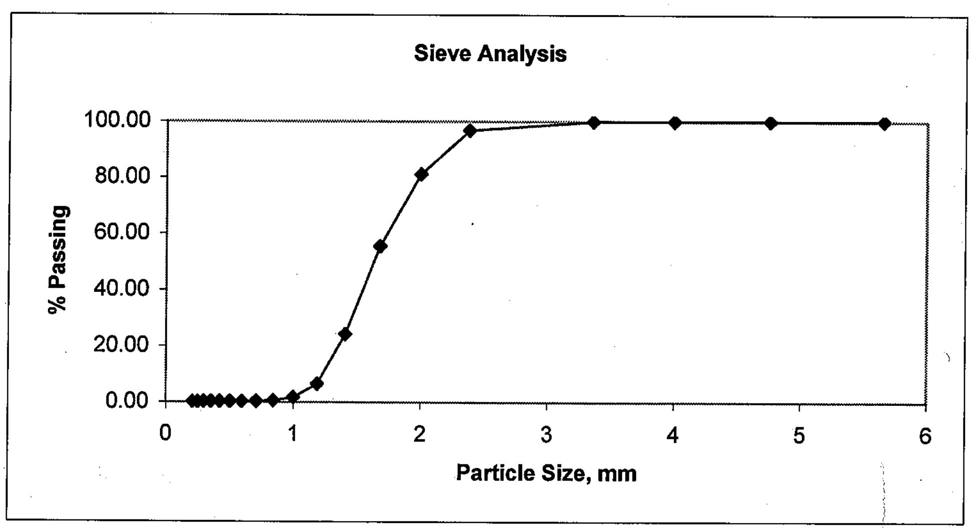
Filter # 7 - Sand

Anthracite

Sample Location

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38	17.90	3.20	3.20	96.80
10	2	87.30	15.62	18.82	81.18
12	1.68	143.49	25.68	44.50	55.50
14	1.41	174.69	31.26	75.76	24.24
16	1.19	99.30	17.77	93.53	6.47
18	1	26.66	4.77	98.30	1.70
20	0.841	6.39	1.14	99.44	0.56
25	0.707	1.02	0.18	99.63	0.37
30	0.595	0.69	0.12	99.75	0.25
35	0.5		0.00	99.75	0.25
40	0.42		0.00	99.75	0.25
45	0.354		0.00	99.75	0.25
50	0.297	0.30	0.05	99.80	0.20
60	0.25	0.20	0.04	99.84	0.16
70	0.21		0.00	99.84	0.16
Pan	#N/A	0.90	0.16	100.00	0.00
		558.83			

D10 =	1.23	Effective Size	1.23
D50 =	1.63	Mean Size	1.63
D60 =	1.74	Uniformity Coefficient	1.41
D90 =	2.21	Backwash rate GPM/ft2	16.8650



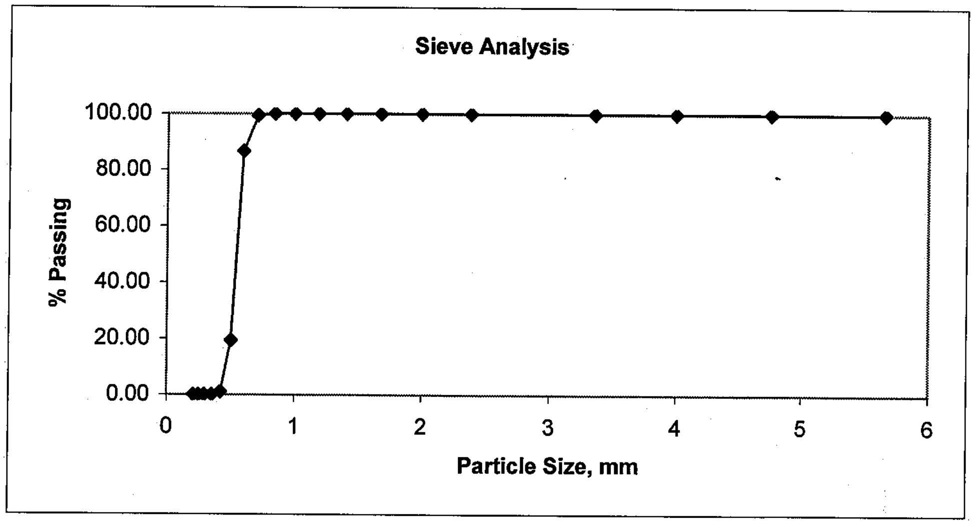
Client	MWH
Job Name	
Filter #	7 - Sand
Sample Location	

Date 11-Aug-03

Sand

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2		0.00	0.00	100.00
12	1.68		0.00	0.00	100.00
14	1.41		0.00	0.00	100.00
16	1.19		0.00	0.00	100.00
18	1		0.00	0.00	100.00
20	0.841	0.21	0.05	0.05	99.95
25	0.707	2.58	0.59	0.64	99.36
30	0.595	55.41	12.69	13.33	86.67
35	0.5	294.10	67.33	80.66	19.34
40	0.42	80.00	18.31	98.97	1.03
45	0.354	4.50	1.03	100.00	0.00
50	0.297		0.00	100.00	0.00
60	0.25		0.00	100.00	0.00
70	0.21		0.00	100.00	0.00
Pan	#N/A		0.00	100.00	0.00
		436.81			

D10 =	0.46	Effective Size	0.46
D50 =	0.54	Mean Size	0.54
D60 =	0.56	Uniformity Coefficient	1.21
D90 =	0.62	Backwash rate GPM/ft2	10.3276

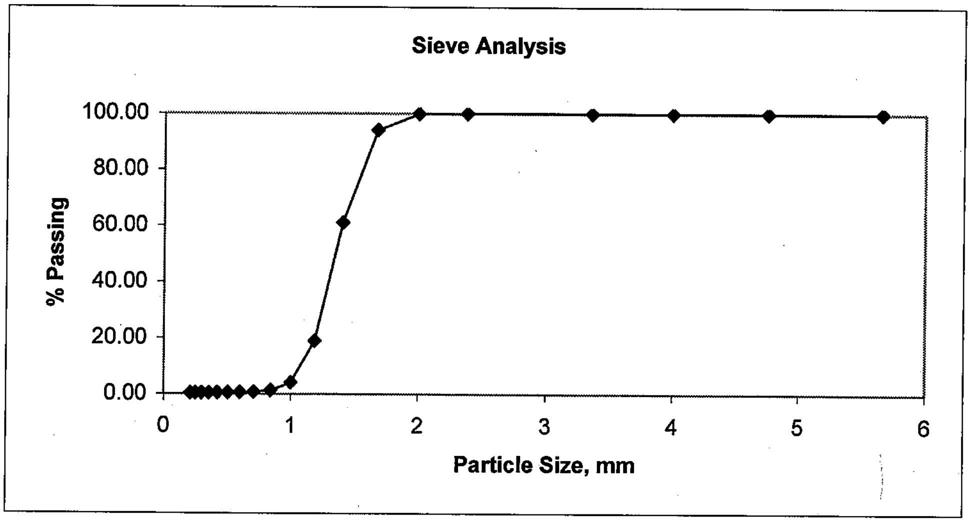


FILTER OPTIMIZATION

Client	**MWH**	**Date**	11-Aug-03
Job Name			
Filter #	**Replacement Anthracite**		**Anthracite**
Sample Location			

				Cumulative	
Mesh	Sieve, mm	Retained, g	% Retained	% Retained	% Passing
3.5	5.66		0.00	0.00	100.00
4	4.76		0.00	0.00	100.00
5	4		0.00	0.00	100.00
6	3.36		0.00	0.00	100.00
8	2.38		0.00	0.00	100.00
10	2	0.60	0.11	0.11	99.89
12	1.68	31.60	5.83	5.94	94.06
14	1.41	178.99	33.03	38.97	61.03
16	1.19	228.19	42.10	81.07	18.93
18	1	80.30	14.82	95.89	4.11
20	0.841	15.80	2.92	98.80	1.20
25	0.707	2.70	0.50	99.30	0.70
30	0.595	0.70	0.13	99.43	0.57
35	0.5	0.50	0.09	99.52	0.48
40	0.42	0.30	0.06	99.58	0.42
45	0.354	0.30	0.06	99.63	0.37
50	0.297		0.00	99.63	0.37
60	0.25		0.00	99.63	0.37
70	0.21		0.00	99.63	0.37
Pan	#N/A	2.00	0.37	100.00	0.00
		541.98			

D10 =	1.08	Effective Size	1.08
D50 =	1.35	Mean Size	1.35
D60 =	1.40	Uniformity Coefficient	1.31
D90 =	1.65	Backwash rate GPM/ft2	13.6447



M E M O R A N D U M

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CITY OF GRANTS PASS
WTP FACILITY PLAN

To:	Rohel Amundson, Jason Canady	**Date:**	August 19, 2003 Sept. 31, 2003 – Rev. 1
From:	Pete Kreft	**Reference:**	City of Grants Pass 1530536.010101
Reviewed By:	Dennis Dorratcague		
Subject:	Review of Rogue River Intake and Pumping Station		

As part of the WTP Facility Plan, the existing Rogue River intake and pumping station is being evaluated to determine possible improvement needs. The existing intake was constructed in the early 1980's when the plant was expanded and upgraded to replace an older intake located immediately upstream. The intake is equipped with 4 identical vertical turbine pumps, capable of delivering approximately 20 mgd to the WTP with all 4 pumps operating. 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 travelling screen is reported to require significant maintenance and/or replacement due to its age. Repair of the screen has reportedly been deferred until a long-term plan for the intake is developed. The existing single intake opening appears to be too small to meet the current maximum approach velocity requirements to protect juvenile salmonid fish species, when pumping at rates greater than 10 mgd. The City is contemplating the addition of a new variable frequency drive (VFD) to one of its raw water pumps to better control plant flows. Hence, this is an opportune time to review potential improvement options.

Attached are photographs of the existing intake and copies of the original intake construction drawings showing general plan and section information. Also included is an original shop drawing of the existing travelling screen.

From the drawings, the floor of the intake wetwell is at elevation 874.00 and tapers to 873.00 where the travelling screen sits. The bottom of the intake openings is at elevation 875.00 and the top of the openings is at elevation 886.00 for an opening height of 11.0 feet. The width of each opening is 5.5 feet. Therefore, each opening has an area of 60.5 square feet.

According to records and discussions with staff, the lower part of the intake was inundated with gravel, silt and debris within 1 to 2 years after construction. A significant amount of material was deposited in front of the intake and this material also entered the intake and damaged pumps, screens and other equipment. As a result of these events, the lower part of each opening was blocked off with stop logs. As best as we can determine, the top of the stop logs are 4.5 feet above the bottom of the openings (at elevation 879.50). The river bed appears to have stabilized at this elevation, although detailed surveying with divers is required to determine the exact conditions.

Therefore, the actual openings that currently allow water to enter the intake are only 6.5 feet high instead of 11.0 feet as originally intended. Each opening only has an effective area of 35.75 square feet, which is almost 40% less than what was designed. These conditions create higher approach velocities of water entering the intake through the screen and result in greater challenges for modifying the intake to meet current and future requirements.

During a site visit on August 15, 2003, the water level in front of the intake was almost at the top of the openings (elevation 886.00) and the water depth was approximately 6 to 7 feet. Hence, it appears that the riverbed is now near the top of the stop logs. In this case, the lower part of the openings are no longer available for installing screens and the net available screen area is significantly reduced.

Silt, sand, gravel and debris collect inside the pumping wetwell and need to be periodically removed. The intake is equipped with a “de-silting” system which scours the wetwell floor with high-velocity water and allows the raw water pumps to move this material into the basins. The existing travelling screen system has gaps along its sides and bottoms which allow the larger material to enter. In addition, these gaps do not meet juvenile fish screening criteria. The intake is not inspected by divers on a routine basis.

The riverbank adjacent to the WTP and upstream of the intake was recently improved and protected via a jointly-funded project between the Army Corps of Engineers (COE) and the City. The bank was in a deteriorated/unstable condition which caused concern about the long-term viability of the plant site. While the bank improvements were successful, it created a significant back-eddy in front of the intake. Actually, prior to the improvements, there was a noticeable back-eddy, but not as significant as exists today. The debris removed by the travelling screen falls from the intake deck back into the river just downstream of the intake and some of this material now flows back to the screens. This condition also needs to be addressed during evaluation of improvement options.

INTAKE AND SCREENING REQUIREMENTS

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. Included in these requirements are:

- Maximum screen opening size
  - 3/32-inch (2.38 mm) for woven wire or perforated plate screens
  - 0.0689-inch (1.75 mm) for profile wire screens
- Minimum 27% open area
- Maximum approach velocity = 0.40 fps; no "hot spots" on/through the screens
- Sweeping velocity past the screens equal to or greater than the approach velocity
- Screens should be "flush" with structures and should not be recessed
- Screens equipped with reliable cleaning system to keep the full screen area open

There are numerous older municipal intakes and screening systems throughout Oregon and the Pacific Northwest which have similar challenges and do not meet the current regulations as stated above. Many of these systems have been modified, or are in the process of being modified, and the regulatory agencies responsible for approving the improvements have shown flexibility in complying with the requirements on a case-by-case basis.

EVALUATION OF EXISTING INTAKE AND SCREEN SYSTEM

As mentioned above, the two openings in the intake are each 11 feet high by 5.5 feet wide, for a total opening area of 121 sf (60.5 sf per opening), but only the upper 6.5 feet +/- are open for water entry as the lower 4.5 feet are blocked off with stop logs and silted in by the riverbed. Hence, the actual useable opening area is 71.5 sf (35.75 sf per opening) assuming the water level is at or above the tops of the openings. The openings are equipped with bar screens to keep large debris from entering, which might damage the travelling screen and/or pumps. A slide gate is located along the face of the intake to close off the screened opening if so desired, but the gate cannot be moved without significant effort. A 2.5-feet x 2.5-feet square opening and gate are also located in the divider wall between the two interior wetwell sections, but the gate is currently inoperable due to damage to the operator and stem extension. As mentioned above, the existing travelling screen is in need of repair/replacement according to plant operators, and could potentially fail at any time. Repairs have been postponed until a long-term plan for the intake is formulated.

Based on this information, the following summarizes current intake characteristics with respect to approach velocity through the existing single screened opening assuming it is completely submerged with the lower 4.5 feet blocked off which provides a 6.5-feet(H) x 5.5-feet(W) clear opening:

- At 5 mgd (7.75 cfs), the approach velocity = 0.22 fps
- At 10 mgd (15.5 cfs), the approach velocity = 0.43 fps
- At 15 mgd (23.2 cfs), the approach velocity = 0.65 fps
- At 20 mgd (27.8 cfs), the approach velocity = 0.87 fps

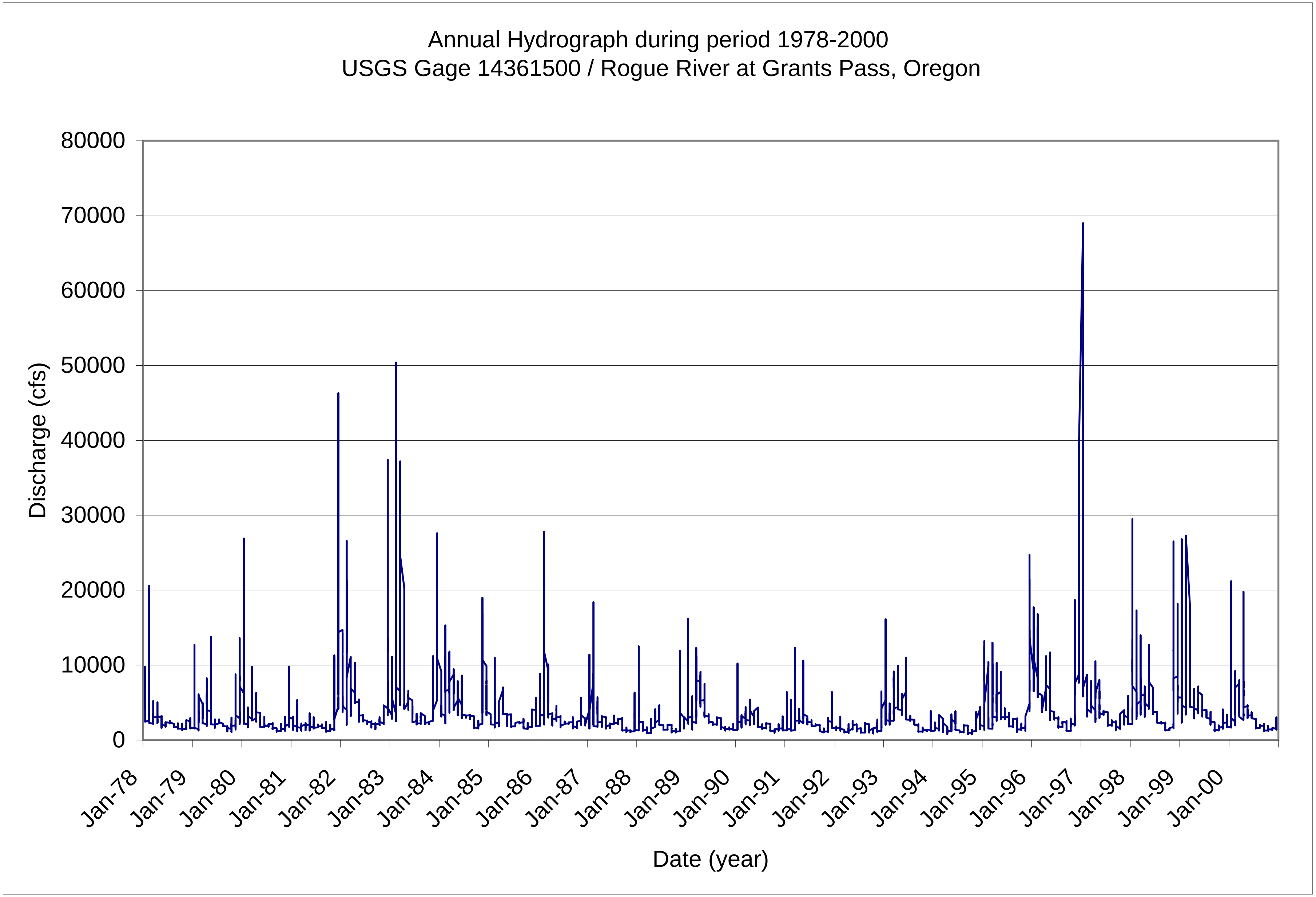
Therefore, when passing more than 9.2 mgd through the single screened intake opening, the 0.40 fps approach velocity criteria is exceeded. This also assumes that the top of the opening is always submerged (ie, the LWL is always greater than elevation 886). Since the plant rarely operates at instantaneous rates less than 10 mgd (2 pumps running), the approach velocity criteria is always exceeded.

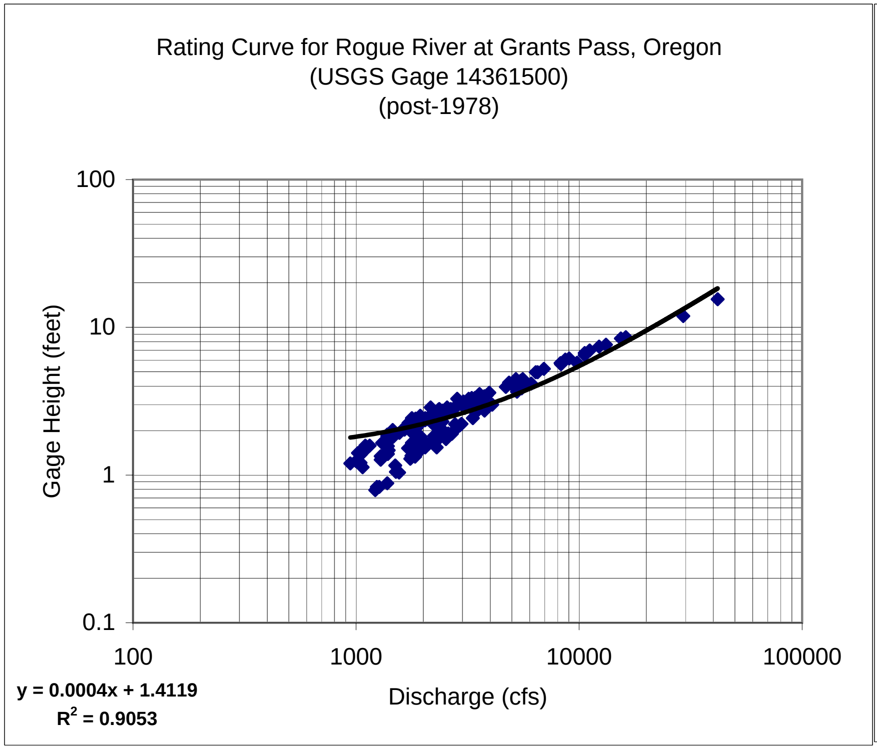
Water Levels and Flows

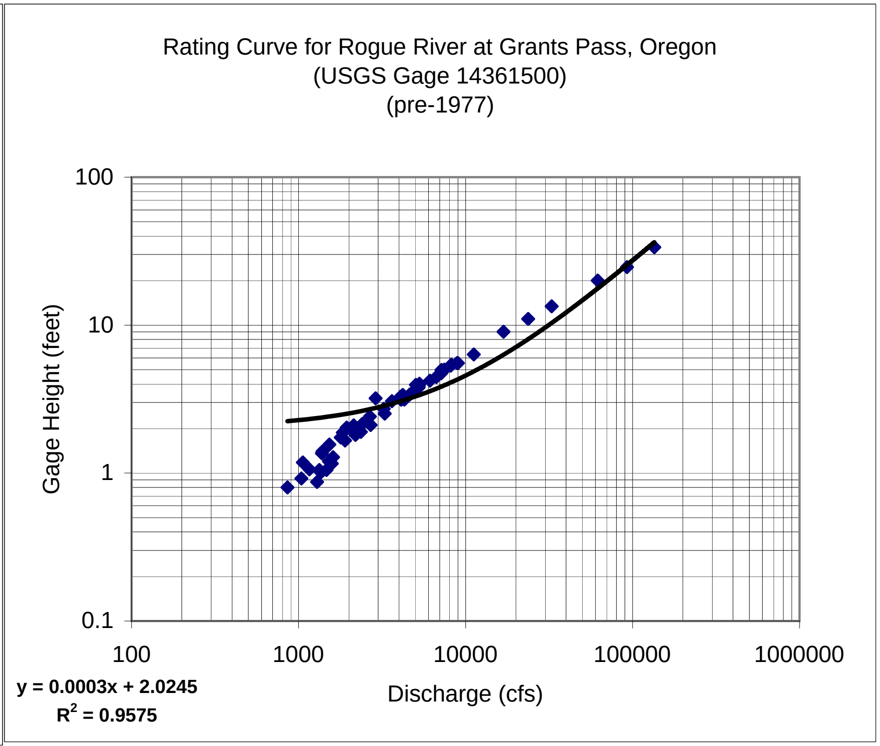
The 1980 construction drawings indicate a low water level (LWL) of 886.0 feet, corresponding to the top of the intake openings. A review of recent historical river flow and elevation data from the gaging station at the old plant intake indicate that occasional low flows of approximately 1,600 cfs or less result in river levels at the intake which are below the top of the openings, perhaps by 2 to 6 inches. On September 24, 2003, the USGS river gage elevation was 0.83 feet and the water level at the intake was approximately 3-inches below the top of the opening, according to plant staff. These low flows can occur during summer high-demand periods when the plant typically operates with 3 or 4 pumps (15 or 20 mgd). Therefore, the full area of the existing openings is not always available for approach velocity calculations.

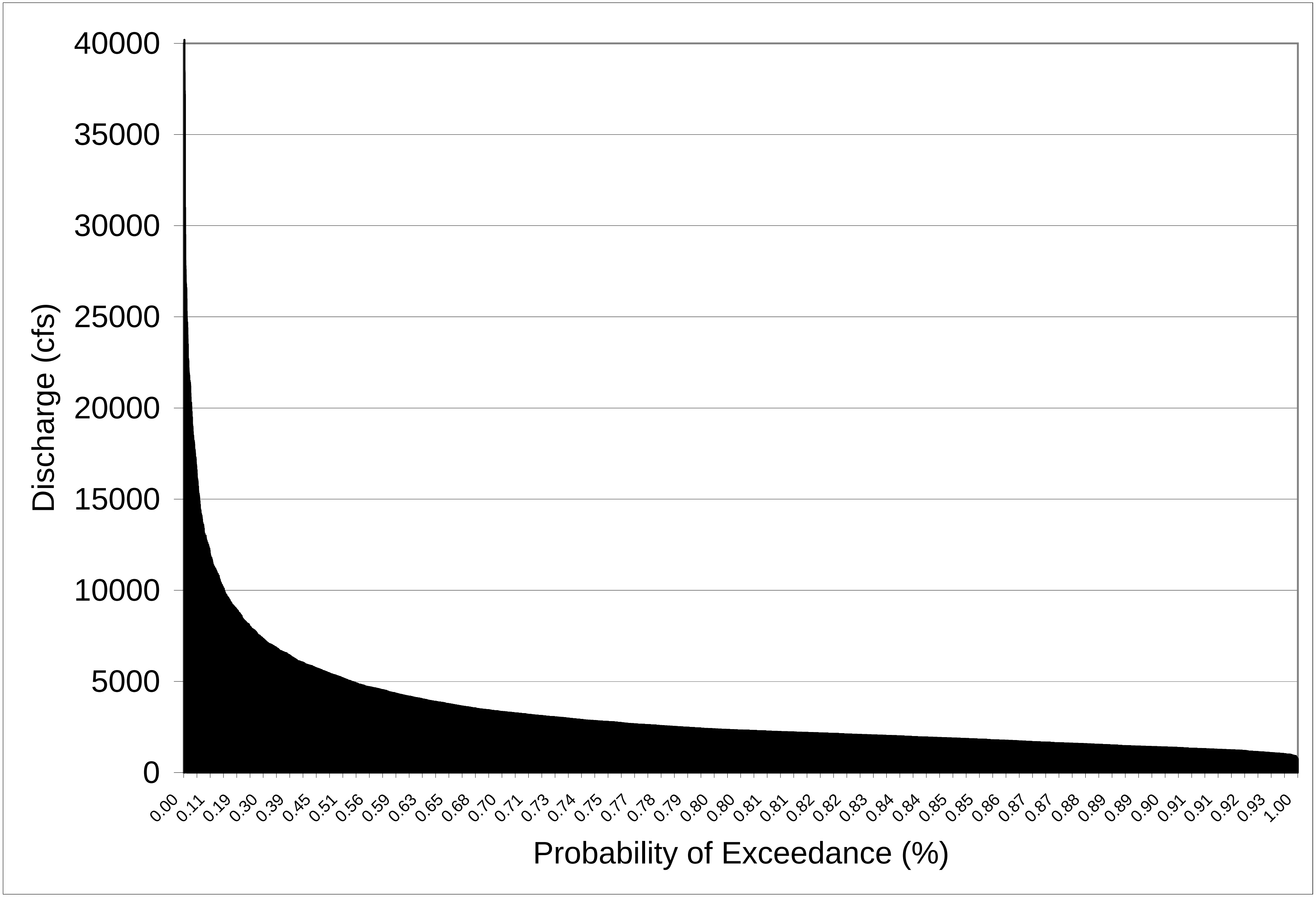
The attached figures represent historical river flows and gage levels for the USGS gage station at the old WTP intake, both before and after construction of the Lost Creek Reservoir. Operations of this dam and reservoir can significantly affect flow and water quality in the lower Rogue River. Also, the Savage Rapids Dam, immediately upstream of Grants Pass, can impact flow and water quality in the lower Rogue River, but to a lesser extent. Short-term and possible longer-term water quality and water level impacts may be observed in Rogue River if the Savage Rapids Dam is eventually removed as has been proposed and discussed over the past few years.

According to shop drawings and visual inspections, the existing travelling screen size openings may not meet the 3/32-inch (2.38 mm) criteria. The clear open space between the woven screen mesh is approximately 1/8 x 1/8 inch, and this is larger than 3/32 inch. The face of the travelling screen is set back approximately 12-inches from the face of the intake structure, and there are gaps along the sides and bottoms of the screens which exceed the minimum opening size.


Annual Hydrograph during period 1978-2000
USGS Gage 14361500 / Rogue River at Grants Pass, Oregon
Discharge (cfs)
80000
70000
60000
50000
40000
30000
20000
10000
0
Jan-78
Jan-79
Jan-80
Jan-81
Jan-82
Jan-83
Jan-84
Jan-85
Jan-86
Jan-87
Jan-88
Jan-89
Jan-90
Jan-91
Jan-92
Jan-93
Jan-94
Jan-95
Jan-96
Jan-97
Jan-98
Jan-99
Jan-00
Date (year)


Rating Curve for Rogue River at Grants Pass, Oregon
(USGS Gage 14361500)
(post-1978)
Gage Height (feet)
100
10
1
0.1
100
1000
10000
100000
Discharge (cfs)
y = 0.0004x + 1.4119
R2 = 0.9053


Rating Curve for Rogue River at Grants Pass, Oregon
(USGS Gage 14361500)
(pre-1977)
Gage Height (feet)
100
10
1
0.1
100
1000
10000
100000
1000000
Discharge (cfs)
y = 0.0003x + 2.0245
R2 = 0.9575



Discharge (cfs)
40000
35000
30000
25000
20000
15000
10000
5000
0
0.00
0.11
0.19
0.30
0.39
0.45
0.51
0.56
0.59
0.63
0.65
0.68
0.70
0.71
0.73
0.74
0.75
0.77
0.78
0.79
0.80
0.80
0.81
0.81
0.82
0.82
0.83
0.84
0.84
0.85
0.85
0.86
0.87
0.87
0.88
0.89
0.89
0.90
0.91
0.91
0.92
0.93
1.00
Probability of Exceedance (%)

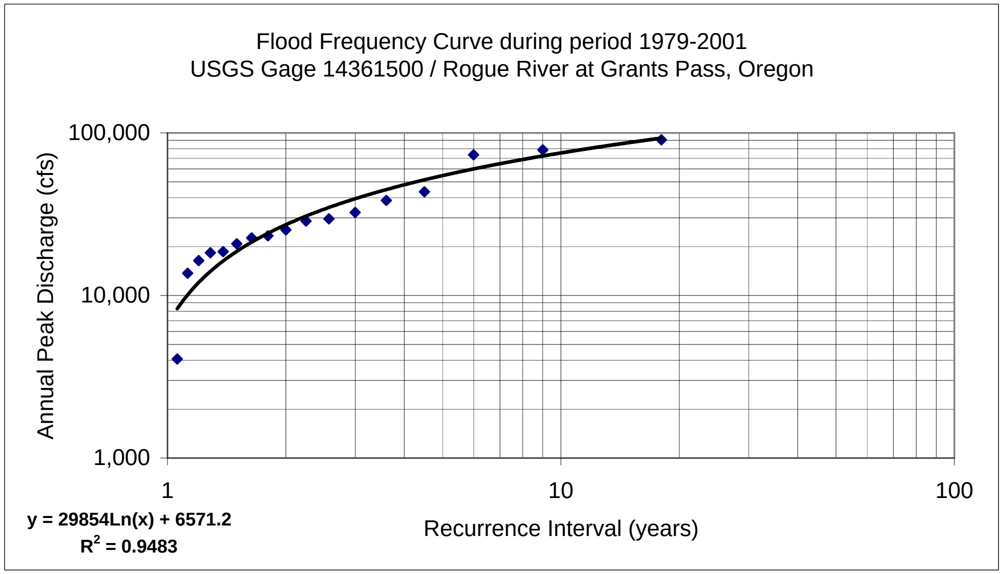
TLB, MWH, August 20, 2003

U.S. Geological Survey
National Water Information System
Retrieved: 2003-08-19 17:27:38 EDT
#
---------------------WARNING---------------------
The data you have obtained from this automated
U.S. Geological Survey database have not received
Director's approval and as such are provisional
and subject to revision. The data are released
on the condition that neither the USGS nor the
United States Government may be held liable for
any damages resulting from its use.
#
This file contains the annual peak streamflow data.
#
This information includes the following fields:
#
agency_cd Agency Code
site_no USGS station number
peak_dt format YYYY-MM-DD
peak_va Annual peak streamflow value in cfs
peak_cd Peak Discharge-Qualification codes (see explanation below)
gage_ht Gage height for the associated peak streamflow in feet
gage_ht_cd Gage height qualification codes
year_last_pk Peak streamflow reported is the highest since this year
ag_dt Date of maximum gage-height for water year (if not concurrent with peak)
ag_tm Time of maximum gage-height for water year (if not concurrent with peak
ag_gage_ht maximum Gage height for water year in feet (if not concurrent with peak
ag_gage_ht_cd maximum Gage height code
#
Sites in this file include:
USGS 14361500 ROGUE RIVER AT GRANTS PASS, OR
#
Peak Streamflow-Qualification Codes(peak_cd):
1 ... Discharge is a Maximum Daily Average
2 ... Discharge is an Estimate
3 ... Discharge affected by Dam Failure
4 ... Discharge less than indicated value,
which is Minimum Recordable Discharge at this site
5 ... Discharge affected to unknown degree by
Regulation or Diversion
6 ... Discharge affected by Regulation or Diversion
7 ... Discharge is an Historic Peak
8 ... Discharge actually greater than indicated value
9 ... Discharge due to Snowmelt, Hurricane,

Post-Dam

Date	**Year**	**Annual Peak Flow (cfs)**	**Rank**	**Recurrence Interval**
5/31/2001	2001	4,070	17	1
1/8/1990	1990	13,700	16	1
1/15/1988	1988	16,400	15	1
3/5/1991	1991	18,300	14	1
1/11/1979	1979	18,600	13	1
1/20/1993	1993	20,800	12	2
2/2/1987	1987	22,600	11	2
11/12/1984	1984	23,300	10	2
1/10/1989	1989	25,300	9	2
12/15/1995	1995	28,700	8	2
4/18/2000	2000	29,600	7	3
2/18/1986	1986	32,400	6	3
1/13/1980	1980	38,400	5	4
11/21/1998	1998	43,400	4	5
2/18/1983	1983	73,300	3	6
12/20/1981	1981	78,700	2	9
1/1/1997	1997	90,800	1	18

Notes: 1) Missing data for yea
2) Flood Frequency Cu



Recent improvements to stabilize the riverbank below the WTP and adjacent to the intake have created a significant “back eddy” which hinders the ability to achieve a sweeping velocity across the face of the screens. The flow in front of the intake is very turbulent depending on river flow. At current low river flows, the back eddy is not as pronounced as it was during Spring 2003 based on visual observations. As mentioned previously, debris discharged from the travelling screen now gets partially “re-entrained” into the screen when it falls back into the river.

Based on this review of existing intake conditions, it is clear that a number of the screening requirements are not being met and that improvements are required to bring the intake into compliance.

EVALUATION OF RAW WATER PUMPS

The intake is equipped with 4 pumps, each 75 Hp, rated at 3,500 gpm (5 mgd) at __ TDH. These pumps were installed in the early 1980’s when the new intake was built. Space was provided for 6 pumps total, presumably all at 5 mgd, for a total pumping capacity of 30 mgd with all 6 pumps running. Therefore, for purposes of this discussion, the intake should be considered to have a maximum pumping capacity of 30 mgd and this matches with the estimate that 30 mgd of treatment capacity can be accommodated on the existing site.

The plant currently operates for significant periods of each year with 4 pumps running for an instantaneous flow of 20 mgd. During the peak demand season, the plant is running at either 15 mgd (3 pumps) or 20 mgd (4 pumps) most of each day to meet the demands during an operating period of 12 to 16 hours per day. The plant is only staffed and operated for 8 to 16 hours per day and is operated on a start/stop basis as a means of reducing plant operating (labor) costs. The plant staff are considering the addition of a new variable frequency drive (VFD) to one of the raw water pumps to provide optimized flow control. MWH supports the decision to add at least one VFD to the raw water pumping system.

Based on the current installation, we consider the reliable/firm raw water pumping capacity to be 15 mgd, based on industry standards, if the largest pump is out of service. The City needs to consider this “risk” if one of the pumps is ever out of service for an extended period of time. However, since the peak day demand is currently 10.5 mgd, having one of the raw water pumps out of service just means that the plant would have to operate for a longer period of time to produce the required volume of water.

Therefore, no changes to the current raw water pumping system would be recommended until the City’s peak day water demand approaches 15 mgd, which may occur in the next 7 to 12 years based on current projections. At that time, various pumping options should be evaluated including:

1. Add one new 5 mgd pump to increase total capacity to 25 mgd, with reliable/firm capacity of 20 mgd, or

2. Add two new 5 mgd pumps to increase total capacity to 30 mgd, with firm/reliable capacity of 25 mgd, or
3. Add two new 10 mgd pumps to increase total capacity to 40 mgd, with firm/reliable capacity of 30 mgd

The ultimate decision needs to be based on economics, reliability/flexibility, whether the existing intake structure will continue to serve the WTP or if a new structure will be constructed, and how much capacity the existing electrical system can support without major cost impacts. For the purposes of this planning effort, it will be assumed that two new 5 mgd pumps will be added to the existing intake within the 20-year planning horizon.

If at all possible, intake improvements to meet fish protection requirements and to provide 30 mgd of intake capacity should be implemented in such a way that the existing raw water pump station continues to deliver raw water to the WTP. This will provide the most economical long-term solution that best utilizes existing infrastructure.

RELEVANT DESIGN CRITERIA AND CONSTRAINTS

The City of Grants Pass has approximately 53 mgd (82 cfs) of water rights on the Rogue River. The City is in the process of perfecting 20 mgd (31 cfs) of these rights based on historical usage and pumping rates. Based on other efforts being performed for the WTP Facility Plan, it is anticipated that the plant can be expanded to approximately 30 mgd on the existing site. Therefore, any proposed intake modifications should be considered for potential expansion, either now or in the future, to support a withdrawal rate of 30 mgd. Based on a cursory review of the intake construction drawings, it appears as though it was designed for expansion to at least 30 mgd. Unfortunately, having the lower part of both intake openings blocked off due to riverbed conditions, along with the presence of a significant back-eddy in front of the intake, creates challenges for bringing the intake into compliance.

Based on current juvenile fish protection criteria, the maximum allowable approach velocity for screening systems is 0.4 feet per second (fps). Hydraulics of the intake and screen design are important considerations to ensure that the flow velocity up to and through the screens is uniform across the entire surface area to avoid potential “hot spots” (locations of high velocity) which could trap/impinge small fish. The screens must also be designed with a maximum opening size of approximately 3/32-inch (2.38 mm) for woven wire or perforated plate screens, or 0.0689 inch (1.75 mm) for profile wire screens, with a minimum 27% open area. The screens shall be equipped with a reliable cleaning system to allow the full surface area of the screens to remain open for water flow through it. Based on the 0.4 fps maximum approach velocity criteria, the following minimum screen surface areas are required for different intake capacities under consideration by the City:

Capacity (mgd)	Design Capacity (cfs)	Minimum Screen Area (sf)
20 (existing)	31.0	77.5

25	38.75	96.88
30	46.5	116.25

As stated previously, the existing opening that is equipped with the travelling screen has an available surface area of 35.75 sf with the lower 4.5 feet blocked off and the opening submerged. The two openings in the existing intake have a total surface area of 71.5 sf. Hence, the single opening is too small for the existing 20 mgd capacity, and even the two openings combined appear to have slightly less area to support a maximum withdrawal rate of 20 mgd. If the full depth of both openings were not blocked off and were available for flow entry (total 121 sf of area), then approximately 30 mgd of capacity could be supported by the existing structure.

Design of a new/modified screen system needs to allow withdrawal of the design capacity under almost any foreseeable water level condition in the river adjacent to the intake. For most municipal water systems, maximum water usage is highest during the summer months when river flows are at their lowest. Peak usage events (usually in July and August) can coincide with drought conditions when rivers are at their historical low levels. Review of historical flows in the Rogue River indicate that minimum flows can occasionally drop below 1,600 cfs and can extend into mid-October during low flow/drought years. As mentioned previously, the City has observed water levels in the river which have dropped below the top of the existing opening by 2 to 6 inches during late summer. This means that the available surface area of the openings decreases by 2 to 3 square feet each. It will be important to allow for these low water levels in the ultimate screen system design.

The Rogue River, and consequently the City’s intake, is subject to a wide variety of debris loads including logs, branches, sticks and twigs, algae, stringy grasses, sand, gravel and large rocks/boulders. Any improvement to the intake needs to address the wide variety of materials which can collect on the screens, which must be removed by the cleaning system, as well as materials which can potentially damage the screens/intake and/or enter the intake.

The existing intake structure accumulates silt, sand, gravel and debris inside the wetwell on a seasonal basis and is not routinely cleaned using a diver. Rather, the intake is equipped with a de-silting system which stirs up collected material on the bottom of the intake and then allows the raw water pumps to deliver this material to the basins. The basins are cleaned on a bi-annual basis in the spring and fall and the river materials ultimately end up in the sludge lagoon.

The travelling screen system currently in use has gaps along its sides and bottom, along with slightly-large mesh opening size, which allows the gravel and debris to enter the wetwell during periods of the year when river materials are being carried downstream (usually during high-flow winter conditions). With the installation of new fish screens which can have no gaps and have smaller openings, the accumulation of gravel and larger debris should cease and the only significant material which will then accumulate is silt and sand small enough to pass through the screen openings. The City should expect to remove less material from the wetwell in the future compared to current conditions, and will likely reduce the amount of material which collects within the basins. As part of the intake improvements, the existing de-silting system should be evaluated to determine if it is doing an adequate job of scouring the intake floor and if there are improvements which can made to improve cleaning operations, perhaps to eliminate the delivery of sand and heavier silts to the WTP basins, but instead return them to the river on an annual basis.

The current intake structure has a “slide gate” along the riverside screened opening which can presumably be used to completely dewater the intake. Plant staff indicate that the gate is very difficult to operate and they have no need to operate the slide gate since they have never attempted to completely dewater the entire intake wetwell. To work on a portion of the intake “in the dry”, there is an internal 2.5-feet (H) x 2.5-feet (W) gate which currently can not be operated properly due to actuator and extension problems. Based on initial visual observations, the external slide gate appears to be structurally inadequate to dewater the sump at anything above minimum water levels. A structural evaluation would be recommended if the gate remains.

It is recommended that this screen modification project not include features which can allow complete dewatering of the wetwell. The costs and installation challenges of such gates and apparatus appear to outweigh the possible benefits, especially considering the second opening which would be screened and the need to provide a slide gate for it also. If complete dewatering of the wetwell is required in the future, then it appears that installation of temporary bulkheads/caps on the outside of the structure over the screens, may be a more cost-effective solution. The installation of the new screens on the existing structure may require use of similar type systems, and the City may be able to save these systems, if used for construction, at the treatment plant for possible future use.

Any screen/intake improvement option also needs to consider the need for the WTP to continue producing potable water during construction. Depending on the type of construction and the permit requirements, it may be necessary to perform work during the limited in-river work period during July and August according to ODFW and NMFS guidelines. This 6-to-8 week period coincides with the City’s maximum water demand period and would require very careful planning and coordination in order to ensure that adequate water is available to the City at all times to meet its demands. Ideally, the City could perform some construction during the off-peak periods of the year when it has lower water demands and more flexibility regarding pumping and treatment operations, but this may not be allowed and could possibly create a need to complete construction over multiple in-river periods.

If some or all of the construction can only be performed during the summer period, and this requires significant shutdowns and/or reduced pumping rates, the City may have to develop an Interim Supply Operating Plan until construction of the proposed improvements is completed. Some aspects of the Plan may actively promote conservation, perhaps implement a curtailment program, discontinue sales to wholesale customers that have other supply sources, and ensure that alternative supply(ies) are available to supplement what can be withdrawn from the Intake. Hence, the planning, permitting, design and construction of the intake modifications needs to carefully address sequencing, methodology and coordination to ensure that impacts to the City’s water supply system are minimized.

The City may also want to take advantage of this “opportunity” to make structural modifications to protect the intake against damage during a severe earthquake. Since the intake was designed and constructed, seismic vulnerability concerns have increased in the Pacific Northwest and critical water supply facilities are often designed to resist significant damage as much as possible.

REVIEW OF INTAKE/SCREEN MODIFICATIONS OPTIONS

The City needs to consider all of the issues mentioned above with respect to future plans for the intake. There are a number of options to bring the intake into compliance with regulations, ranging from construction of a new intake to modifications of the existing structure, and the City needs to carefully weigh the pros and cons and costs of each option. The ultimate decision will also depend on how comfortable regulatory agencies are with the City’s preferred option, as they may not be as concerned about the cost impacts as the City would be. It is very important to strategically engage the various regulatory agencies at key points in the decision-making process, to enhance the City’s ability to implement the most cost-effective and long-term solution without being forced into an undesirable decision by the regulators.

The City also needs to acknowledge its current risk and potential liability with respect to on-going non-compliance when deciding how quickly to proceed with improvements, and whether short-term modifications to its operating policies (such as operating at a lower rate to keep the approach velocity below 0.4 fps) are warranted.

The City should view this as an opportunity to implement a long-term solution for the intake in terms of capacity and water rights, considering that it has the capability to treat flows greater than the current 20 mgd maximum on the existing WTP site. The City’s water rights may also be able to be “firmed up” via this process, if careful planning and discussions with regulators are conducted, and if so desired by the City.

With respect to intake capacity, the City can either choose to make improvements to maintain the existing 20 mgd capacity to serve existing needs or consider improvements to serve the ultimate site capacity of 30 mgd. Preliminary discussions with City staff indicate a preference to make improvements that will allow 30 mgd withdrawal capacity, as this will likely be more-cost effective considering the probable significant mobilization costs to construct the improvements no matter what capacity is intended. Also, improving the intake to allow 30 mgd now will reduce the permitting requirements and construction constraints in the future, when it may be more-challenging to get such work accomplished assuming increasingly-stringent environmental regulations.

It is acknowledged that travelling screens are not a viable long-term improvement option since this type of screen can’t meet all fish-protection criteria because of the screen “set-back”, and gaps along the edges of the openings. Therefore, the preferred screen type is a fixed screen, either using “flat-plate” design attached to the face of the structure, or a submerged, cylindrical screen which projects into the water away from the structure. Fixed screens have a proven track record in many applications throughout the country

The back-eddy in front of the existing intake does not allow sweeping velocity criteria to be met under almost all flow conditions and either has to be eliminated or the screens have to project further into the river to avoid the eddy. The City is reluctant to consider modifications to the bank which could eliminate the back-eddy, since this may undermine the bank stabilization improvements recently made, and there is no guarantee that the eddy will be completely eliminated.

These decisions and constraints lead to the following intake improvement options for consideration by the City:

1. Construct a new 30 mgd intake, at a different location; keep existing intake operational until new intake is operational

2. Modify the existing intake for as much capacity as feasible (approximately 15 mgd) without adding to the structure and build a new intake at another location to provide the additional 15 mgd.

3. Install a series of submerged cylindrical intake screens in the river for 30 mgd, connected with pipe to the existing intake, and far enough away from the existing intake to avoid the eddy

4. Modify and extend the existing structure out into the river to avoid the eddy and install flat-plate screens on the face of the extended structure

Other options which were given consideration but not carried forward for detailed comparison include construction of a subsurface collector well system, either parallel to the riverbank or projecting under the riverbed. These types of systems have been constructed for smaller capacity intakes under different geological and hydrogeological conditions. Collector wells are not considered technically or economically viable for this application.

Each of the four short-listed options is discussed further in the following sections. A range of preliminary capital costs is presented for each option which include contingencies, engineering, permitting and administration. The estimates do not include costs for increasing the pumping capacity. These cost ranges are presented for comparison with other alternatives to assist in the evaluation process.

Option 1 - Construct New 30 mgd Intake

This option includes construction of a new intake further downstream within the City’s WTP property and would divert raw water by gravity to the existing intake for pumping to the WTP. The existing intake and travelling screens would be abandoned by sealing the existing intake openings. The intake, screens and piping would be designed for a 30 mgd withdrawal rate. The intake would use flat-plate screens located along the face of the new structure and the face would project into the river far enough to achieve acceptable sweeping velocities. The eddy which is created in front of the existing intake would be avoided.

The screens would be designed for a maximum approach velocity of 0.4 fps under minimum water level conditions. The maximum height of the screens would be approximately 6.0 feet; the top of the screens would be at elevation 886.0 and the bottom of the screens would be at elevation 880.0. The total screen length would be at least 20 feet to achieve the minimum required screen area of 116 square feet for 30 mgd. This design will keep the screens above the normal riverbed elevation as currently understood.

The screens could be cleaned by a number of methods including mechanical rakes, a water jet system with nozzles located behind the screens, or use of air-burst with water jets if the screens are sloped far enough away from the vertical position.

This option would allow continued use of the existing intake/pump station during construction and would not impede the plant's ability to produce water during the peak summer demand period. Some short-term shutdowns might be required to connect piping to the existing intake and for construction purposes.

The range of estimated capital costs for this option is from $3.0 to $4.0 million.

Option 2 - Modify Existing Intake and Construct New 15 mgd Intake

The existing intake has two openings, with approximately 71.5 square feet of available opening above the riverbed. It is conceivable to use this area for new flat-plate screens for an approximate withdrawal rate of 15 to 20 mgd, with some structural modifications necessary to achieve 20 mgd to widen the existing openings. Then, a new intake would be required to provide the additional 10 to 15 mgd of additional withdrawal capacity to achieve 30 mgd total.

This option is only feasible if the back-eddy in front of the existing intake can be eliminated or ignored. As discussed above, the City is reluctant to consider modifications to the bank which could eliminate the back-eddy, since this may undermine the bank stabilization improvements recently made, and there is no guarantee that the eddy will be completely eliminated. If no eddy improvements are available, then the only way this option is viable is if the regulatory agencies involved in permitting intakes and fish screens "waive" the requirement for a sweeping velocity in front of the existing intake's screens. At this time, there is no guarantee that this would occur.

If the existing intake can be modified via this approach, then the City could proceed with these improvements now and choose to defer construction of the new intake until the WTP is expanded to 30 mgd, assuming the existing intake could be permitted for 20 mgd withdrawal rate.

This option would allow continued use of the existing intake/pump station during construction of the new intake and would not impede the plant's ability to produce water during the peak summer demand period (assuming all work is done concurrently). Some short-term shutdowns might be required to connect piping to the existing intake and for construction purposes. Modifications to the existing intake would be completed after the new intake is operational. If the City decides to defer construction of the new intake, then the existing intake would have to be modified in such a way as to ensure that at least 10 mgd of intake and pumping capacity is always available during construction.

The range of estimated capital costs for this option is from $2.5 to $3.5 million.

Option 3 - Submerged Cylindrical Screens for 30 mgd

This option would add multiple cylindrical screen intakes in front of the existing intake, away from the existing back-eddy, and deliver raw water to the existing intake wetwell via a piped connection (36-inch approximate pipe size). At least two cylinders manifolded together, each 24-inch diameter by 10 feet long, would be required for 30 mgd withdrawal capacity. The existing intake and travelling screens would be abandoned by sealing the existing intake openings.

The cylinders would probably be installed 15 feet away from the existing intake and "parallel" to the river current with the tops would be located at least 3 feet below the minimum water surface (at approximate elevation 883.0). The limited water depth during low flows will determine the actual cylinder size and elevations, as they require a minimum submergence, as well as a minimum dimension above the riverbed. The cylinders would likely be surrounded by posts or other means of protecting the screens from damage from boulders and river debris, as well as to advise boaters and swimmers of their location.

The submerged cylindrical screens would be cleaned by an air-burst system via piping and a compressor and air receiver tank located near the existing intake. The compressor and tank would require approximately 800 square feet of space and requires a shelter to protect the equipment. There would be violent agitation and air release above the screens during cleaning, probably enough to be dangerous to a boat or swimmer which may be near the screens. The location of these screens would also impact recreational boating and fishing which currently occurs in front of the intake by creating a navigational hazard and a potential underwater "snag".

This option would likely require construction of a cofferdam to dewater the area where the screens and pipe would be installed, probably all the way back to the intake. Therefore, this option requires careful design, planning and coordination with the Contractor to ensure continued use of the existing intake/pump station during construction to provide approximately 10 mgd withdrawal rate. Some short-term shutdowns might be required to connect piping to the existing intake and for construction purposes. Installation of temporary pumps to deliver water from the river to the intake wetwell may be required if the cofferdam can't be constructed in such a way to avoid this.

The range of estimated capital costs for this option is from $2.0 to $2.5 million.

Option 4 - Modify and Extend the Existing Intake with Flat-Plate Screens for 30 mgd

This option would modify the existing intake by constructing an "extension" from the existing intake out into the river to allow installation of flat-plate screens in a location to avoid the existing back-eddy. It is estimated that the face of the screens would need to be at least 10 feet further into the river from the existing intake openings to avoid the eddy.

The screens would be designed for a maximum approach velocity of 0.4 fps under minimum water level conditions. The maximum height of the screens would be approximately 6.0 feet; the top of the screens would be at elevation 886.0 and the bottom of the screens would be at elevation 880.0. The total screen length would be at least 20 feet to achieve the minimum required screen area of 116 square feet for 30 mgd. This design will keep the screens above the normal riverbed elevation as currently understood.

The screens could be cleaned by a number of methods including mechanical rakes, a water jet system with nozzles located behind the screens, or use of air-burst with water jets if the screens are sloped far enough away from the vertical position.

The intake structure would be physically connected to the existing intake and designed/built in such a way to resist structural, hydrodynamic and seismic forces. It is likely that the floor of this structural extension would match the floor of the existing intake (at elevation 870 +/-) and project into the river to the screen face, even though the lower part of the structure would not have screens. This extension may have to be supported with piles assuming that bedrock may not be able to fully support the loads (this applies to all alternatives). The top of the extension at the screen face would be at elevation 887 +/- and then slope back to the existing intake to avoid creating a "bench or shelf" which would be accessible to boaters and swimmers under low water level conditions.

This option would likely require construction of a cofferdam to dewater the area in front of the intake to allow construction of the extension and screens. Therefore, this option requires careful design, planning and coordination with the Contractor to ensure continued use of the existing intake/pump station during construction to provide approximately 10 mgd withdrawal rate. Some short-term shutdowns might be required for construction purposes. Installation of temporary pumps to deliver water from the river to the intake wetwell may be required if the cofferdam can't be constructed in such a way to avoid this.

The range of estimated capital costs for this option is from $1.5 to $2.0 million.

Comparison of Options

Costs and a number of non-cost criteria were used to evaluate and compare the 4 options described above. Table 1 presents the comparison in a semi-quantitative manner. In addition to being one of the lowest cost approaches, Option 4 has the least number of "negative" attributes for the identified criteria. Therefore, Option 4 is recommended for planning purposes as the preferred intake modification option. One of the biggest challenges for Option 4 is keeping the existing intake/pump station operational during construction to allow the WTP to produce adequate water during the peak demand season. This should be a key focus item during planning and design of this project. The attached figure represents the conceptual improvement plan for Option 4.

PERMITTING REQUIREMENTS

Various permits and environmental documentation are required prior to construction of the selected improvements. Following is a listing of the major permitting requirements for modifications to the City's Rogue River Intake.

Army Corps of Engineers (COE) – Requires a 404 permit in consultation with NOAA/NMFS; biological assessment and/or opinion may be required to assess ESA compliance. May need DEQ 401 certification, dependent on type of 404 permit required by COE.

Division of State Lands (DSL) – Fill/Removal Permit will be required for in-river work; consult with ODFW regarding in-river work period and construction methods; resolve property ownership issues if there are any questions.

City – Land use permit to address floodway impacts and construction noise issues; should consult with County regarding floodway impacts.

No contacts have been made with any of the agencies/representatives responsible for administering the permitting process. It is recommended that the City avoid formal contacts regarding this project until an acceptable improvement plan, schedule and the permitting strategy is developed. Formal permit application preparation, including development of supporting documentation, should begin following the City's acceptance of a preferred conceptual design and selection of a preferred improvement approach. When the City decides to begin detailed design, the City should request agency review comments and conditions as appropriate early in the design phase.

TABLE 1

CITY OF GRANTS PASS
WATER TREATMENT PLANT FACILITY PLAN
ROGUE RIVER INTAKE MODIFICATIONS

RELATIVE COMPARISON OF OPTIONS FOR 30 MGD CAPACITY

Criteria	**Option 1** **New 30 mgd Intake**	**Option 2** **Modify Existing Intake with New 10 mgd Intake**	**Option 3** **New Submerged Cylindrical Screens**	**Option 4** **Extend Existing Intake with Flat-Plate Screens**
Construction Cost	-	-	o	o
Operating Cost	o	-	o	o
Long-Term Environmental Impacts to River	o	-	o	o
Short-Term Environmental Impacts during Construction	o	o	-	o
Impacts to Recreation and Navigation	o	o	-	+
Floodway Impacts	-	-	o	o
Reliability and Ease of Cleaning	o	o	-	o
Risk of Damage during High Flows	o	o	-	o
Impacts to Pumping During Construction	+	o	-	-
Pumping/ Hydraulics	-	-	o	+
Liklihood of Completing Construction in 1 Season	-	-	-	o
Flexibility for Future Unknowns	+	o	o	o
Impact of Very Low Water Levels	+	o	-	o
Overall Ease of Permitting	-	-	o	o

\+ means option is favorable for the criteria compared to other options
o means option is acceptable/neutral compared to other options
\- means option is less favorable compared to other options

SCHEDULE

It is likely that the permitting process will take 12 to 18 months to complete based on current COE and NMFS backlog and based on MWH's recent experience with similar projects in Oregon. Therefore, it is unlikely that construction could be completed during summer 2005 (unless the City decides to expedite this project) and the early target for in-river construction, if required, should be summer 2006. To meet this schedule, preliminary design and permitting should begin as soon as the City decides to proceed with the project, as a construction contractor would have to be hired approximately 6 months prior to the summer of 2006 (say by end of 2005) to allow planning and procurement time before in-river construction begins. The overall project schedule will last approximately two (2) years from initiation of preliminary design and permitting to closeout of construction.

RECOMMENDATIONS AND SUMMARY

The preferred approach for modifying the existing Intake structure to meet fish protection criteria and to achieve 30 mgd withdrawal capacity is Option 4 – "Modify and Extend the Existing Intake with Flat-Plate Screens". This approach appears to be the most feasible, can meet the needs of the City, and is a low-cost approach compared to other options considered.

Summarized below are major work efforts and considerations to be completed for this project as the City moves forward:

- Make informal contact with regulatory agencies about preferred improvements
    - Discuss preferred option to meet approach velocity, as well as present other options that were considered
    - Discuss back eddy/sweeping velocity issues
    - Discuss screen cleaning options
    - Discuss potential capacity expansion >20 mgd
    - Determine when improvements can/should be made
        - Is there any work which can possibly be performed outside of the in-river work period?
- Determine preferred schedule for design, permitting and construction – as discussed above, the earliest probable in-river construction window is summer 2005 based on the length of the permitting process
- Begin preliminary design after receiving verbal input and approval by regulatory agencies
- Initiate permit application and approval process during/after predesign is completed
- Develop detailed construction sequencing plan and determine methods for maintaining as much of capacity during summer as possible
- Develop an Interim Operating and Supply Plan to address possible production shortfalls during construction
- Remove and dispose of existing travelling screen system
- Implement permitting and regulatory agency conditions and requirements when these are established

The City should therefore minimize investments to its existing intake and travelling screen system until the modified intake is designed and constructed. The City should take a low-cost approach to ensuring the reliable operation of the existing travelling screen for the next few years.

PRELIMINARY CAPITAL COST ESTIMATE

A planning-level capital cost estimate is presented in Table 2 for the preferred intake modification option. The accuracy of this estimate should be considered + 50%/- 30% for this stage of the planning process. A 50% markup was added to the construction cost estimate for contingencies, engineering, permitting and administration. This markup is higher than for other elements of the WTP Facility Planning project due to the greater uncertainty and risk associated with this type of project.

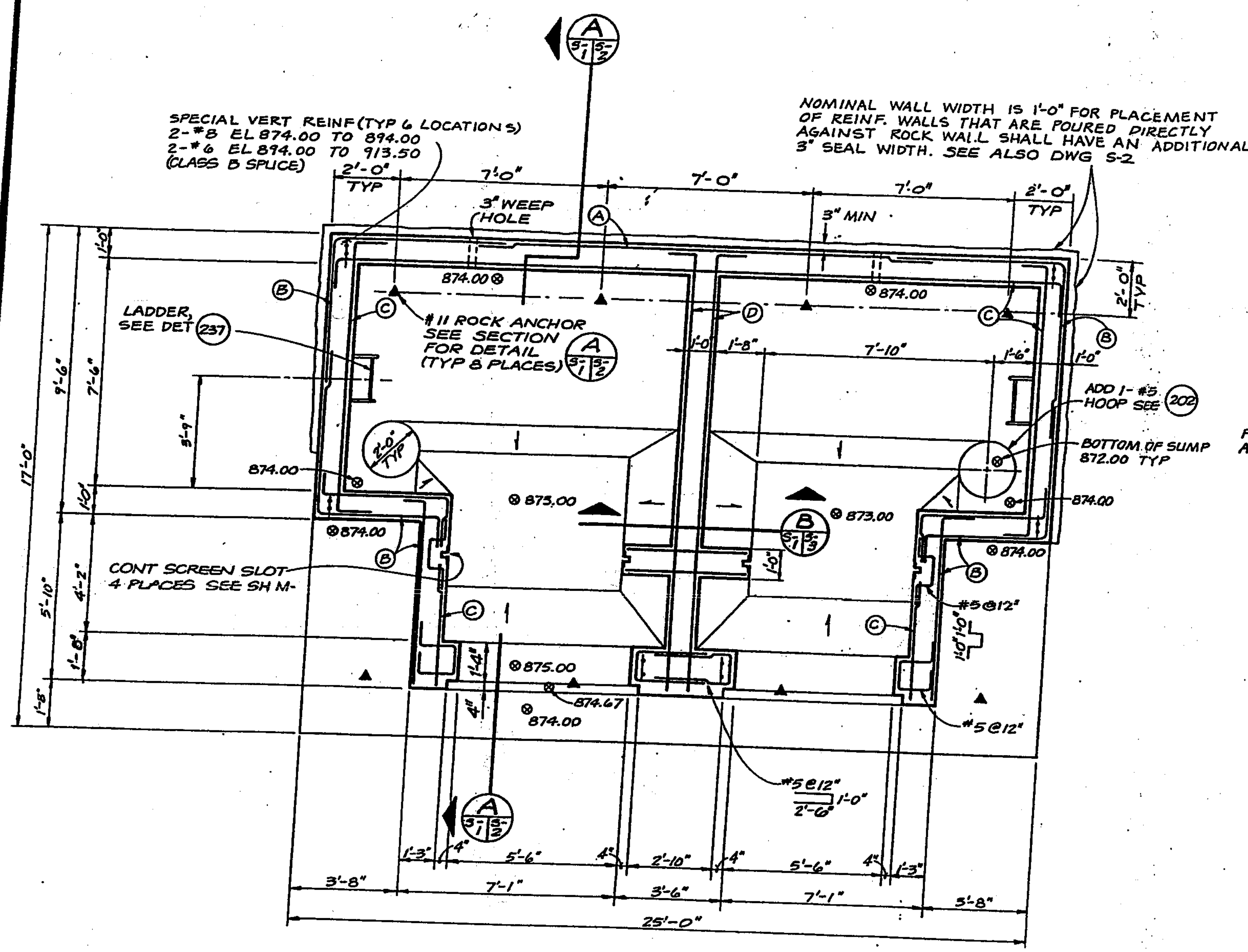
TABLE 2

PLANNING-LEVEL CAPITAL COST ESTIMATE

CITY OF GRANTS PASS ROGUE RIVER INTAKE MODIFICATIONS PROJECT

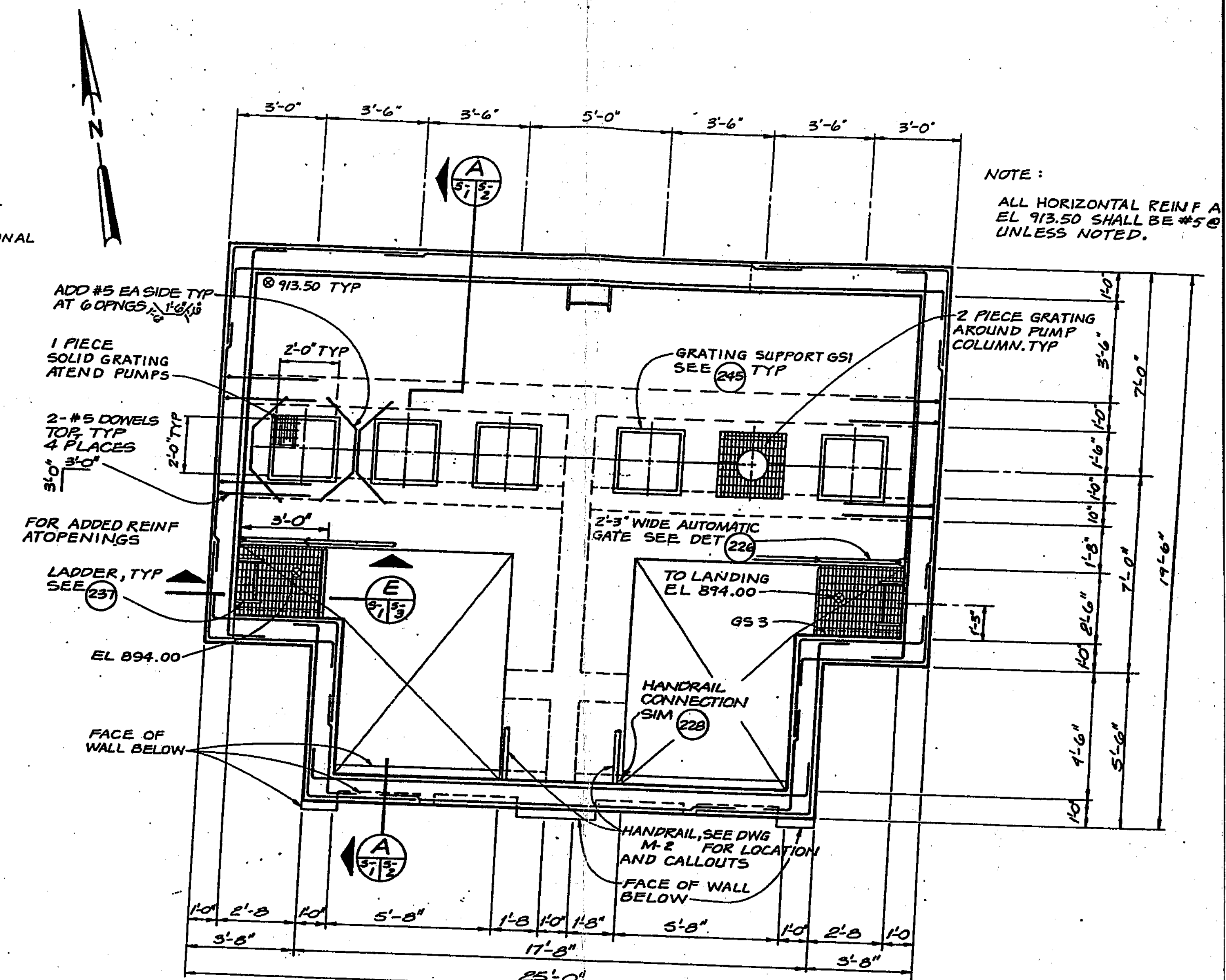
Item	Description	Unit	Quantity	Unit Cost	Total Materials	Installation Percentage	Installation Cost	Installed Cost	Element Cost
	Install/Remove Cofferdam	LS	1	$ 75,000			$ 75,000	$ 75,000	
	Excavation and disposal	LS	1	$ 75,000			$ 75,000	$ 75,000	
	Structural Support/Piles	LS	1	$ 75,000			$ 75,000	$ 75,000	
	Concrete/Structural	LS	1	$ 100,000			$ 100,000	$ 100,000	
	SS Screens and Frames	ea	2	$ 30,000			$ 60,000	$ 60,000	
	Screen Cleaning Systems	ea	2	$ 35,000			$ 70,000	$ 70,000	
	Cleaning System Pump, Piping, Valves	LS	1	$ 45,000			$ 45,000	$ 45,000	
	Temporary Pumps and Piping	LS	1	$ 50,000			$ 50,000	$ 50,000	
	Environmental Protection	LS	1	$ 30,000			$ 30,000	$ 30,000	
	Remove existing Travelling Screen	LS	1	$ 10,000			$ 10,000	$ 10,000	
	Electrical/I&C	LS	1	$ 60,000			$ 60,000	$ 60,000	
	Misc. Work	LS	1	$ 50,000			$ 50,000	$ 50,000	
Construction Sub-Total									**$ 700,000**
Mobilization			**20%**						**$ 140,000**
Bonds and Insurance			**2%**						**$ 14,000**
Testing and Startup			**LS**						**$ 10,000**
Demobilization and Cleanup			**10%**						**$ 70,000**
Subtotal									**$ 934,000**
Contractor Overhead and Profit			**15%**						**$ 140,100**
Subtotal - Construction Cost Estimate									**$ 1,074,000**
Contingency, Engineering, Permitting, Admin.			**50%**						**$ 537,000**
TOTAL ESTIMATED CAPITAL COST									**$ 1,611,000**

Notes: 1) Costs are in 2003 dollars and include no escalation.



PLAN AT EL. 875.00

3/8" = 1'-0"



PLAN AT EL. 913.50

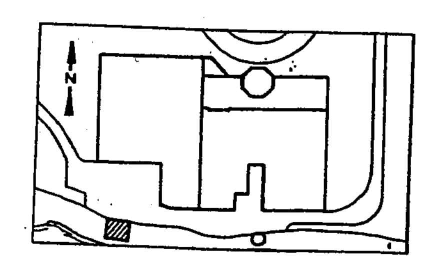
3/8" = 1'-0"

NOTES:

1. FOR STRUCTURAL NOTES, SEE SH S7 & S23
2. SEE MECHANICAL DWGS FOR OPENINGS NOT SHOWN

HORIZ. REINF. SCHED.

BAR TYPE	EL 874.00 TO 884.00	EL 884.00 TO 894.00	REMAINING
A	#7@6"	#7@12"	#5@12"
B	#7@12"	#6@12"	#5@12"
C	#5@12"	#5@12"	#5@12"
D	#6@6"	#6@12"	#5@12"



KEY PLAN

REGISTERED PROFESSIONAL ENGINEER 4713 OREGON JULY 14, 1961 STEVE S. C. LEE

CH2M HILL

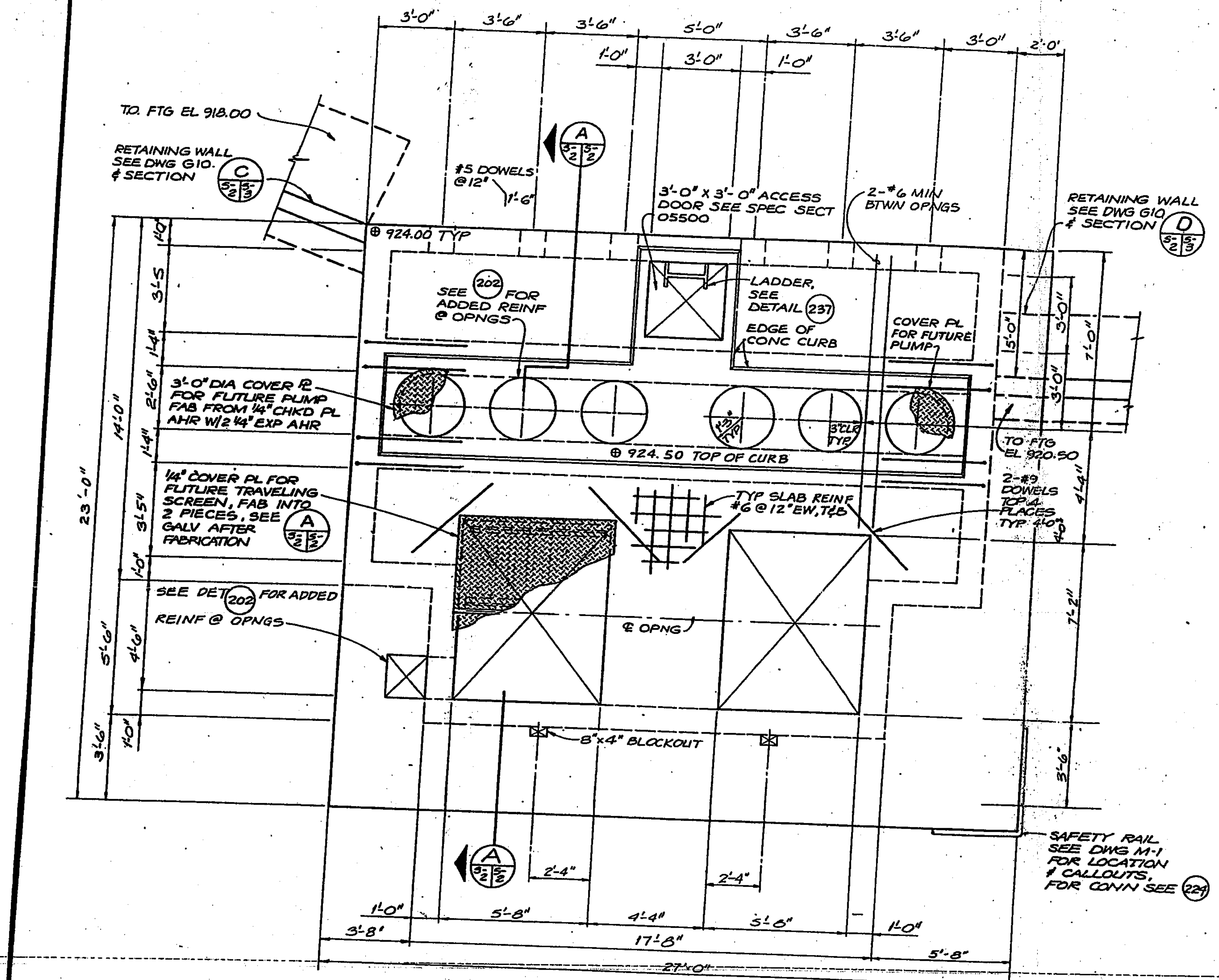
DES. DEF	NO.	DATE	REVISION	BY	APPD.
DR. CRS					
CHK. LJE					
APPD. AEM					

THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE
IF THE SCALE READS: 1" = 1'-0" USE 1/2" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

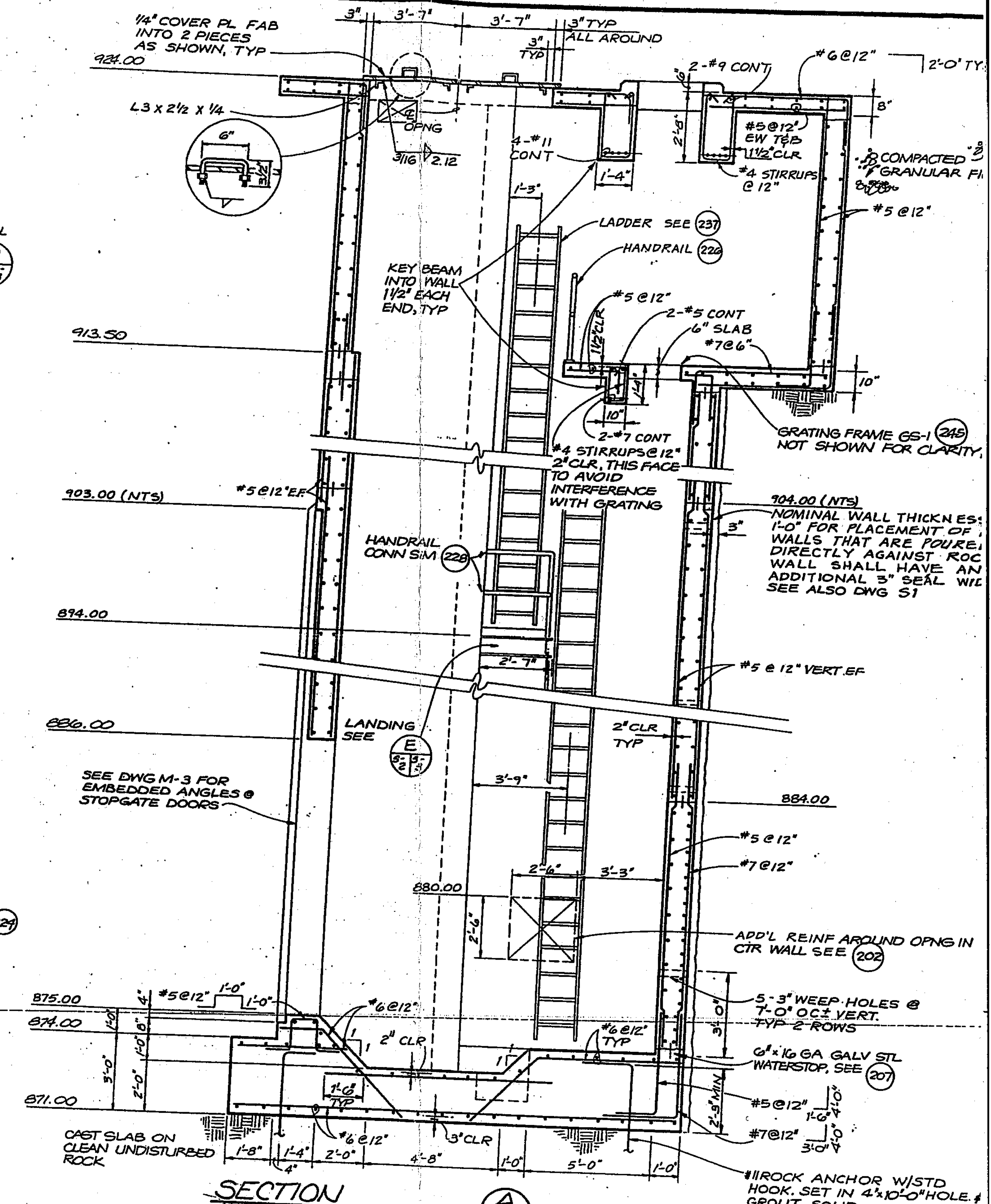
STRUCTURAL
RAW WATER INTAKE PLANS

SH. 28
DWG. NO. S-1
DATE: APR 1980
FILE NO. C13078.T1



PLAN AT EL 924.00

3/8" = 1'-0"



SECTION A

3/8" = 1'-0"

SEE DWG S-1 FOR DETAILS OF HORIZ REINF

NOTES:

1. FOR STRUCTURAL NOTES SEE SH S7 & S23
2. SEE MECHANICAL DWGS FOR OPENINGS NOT SHOWN.

REGISTERED PROFESSIONAL ENGINEER 4713 OREGON JULY 14, 1961 STEVE S. C. LEE

CH2M HILL

DES. DEF	NO.	DATE	REVISION	BY	APPD.
DR. CRS					
CHK. LJE					
APPD. AEM					

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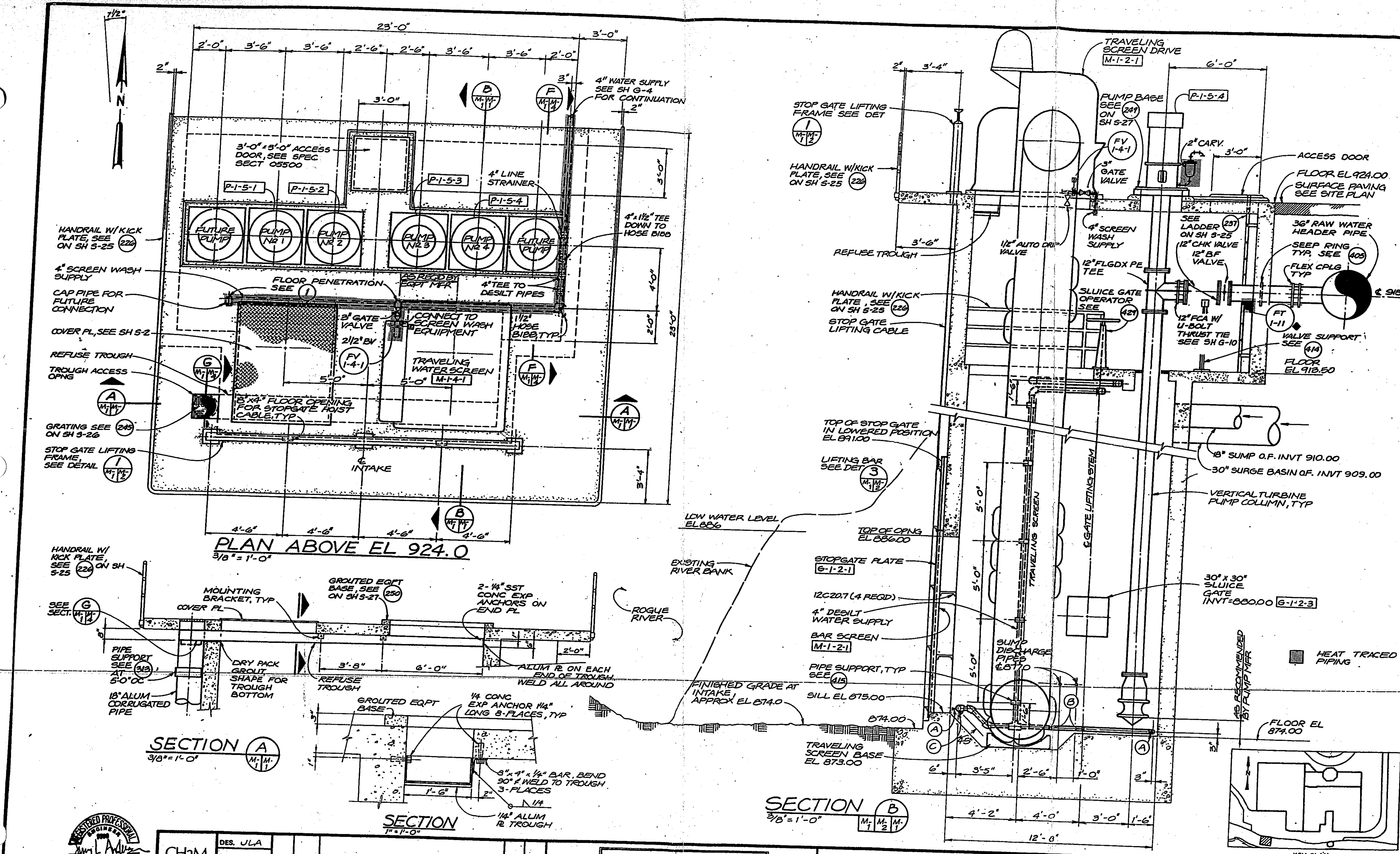
CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

STRUCTURAL
RAW WATER INTAKE PLAN AND SECTION

SH. 29
DWG. NO. S-2
DATE: APR 1980
FILE NO. C13078.T1



REGISTERED PROFESSIONAL ENGINEER, OREGON — JERRY L. ANDERSON

CH2M HILL

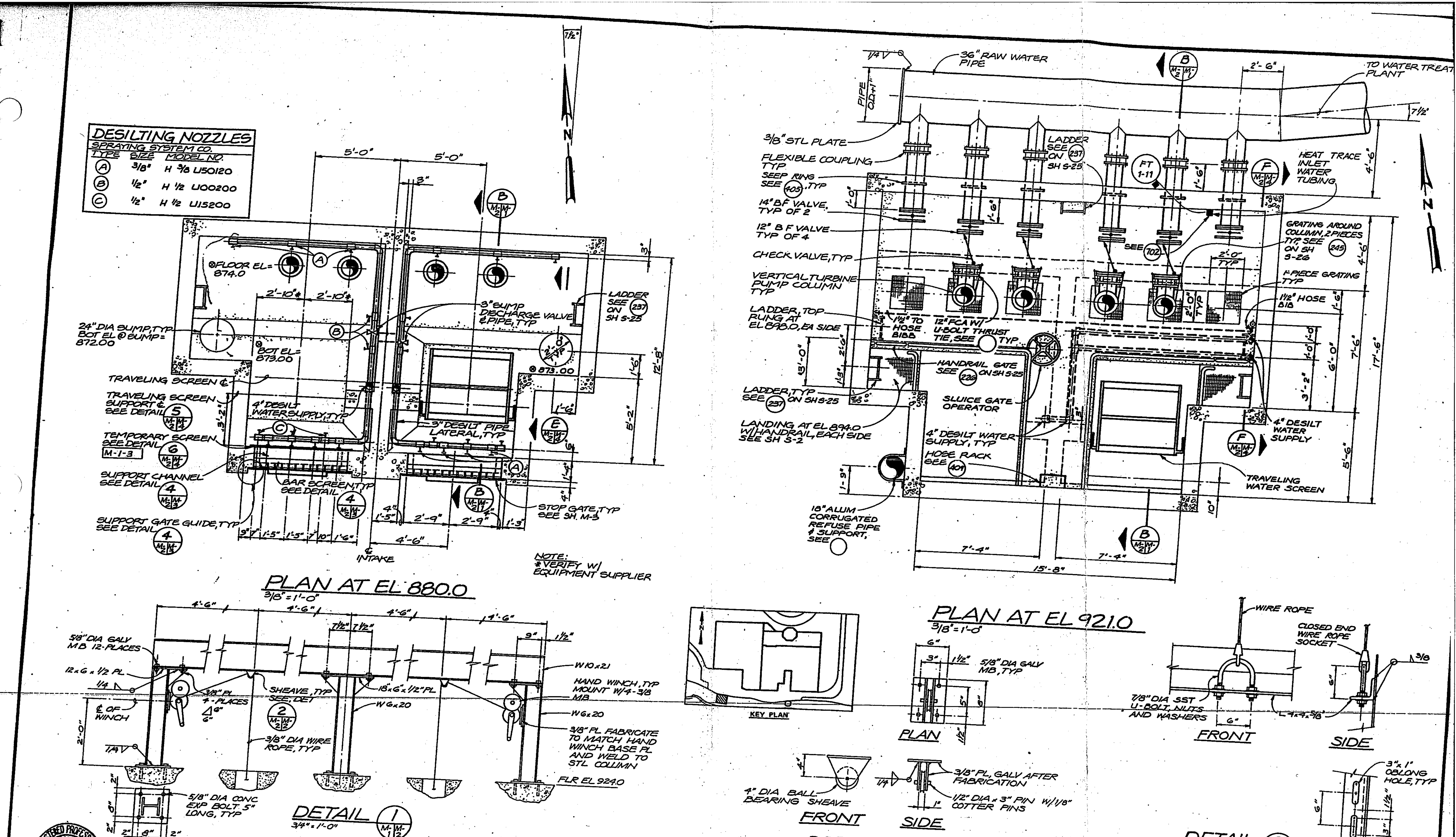
DES. JLA	NO.	DATE	REVISION	BY	APPD.
DR. MAW					
CHK. NCW					
APPD. AEM					

THIS PRINT IS REDUCED TO ONE-HALF OF THE ORIGINAL SCALE
IF THE SCALE READS:
1" = 1'-0" USE ½" = 1'-0" OR 1" = 10' USE 1" = 20'

CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

MECHANICAL
RAW WATER INTAKE PLAN AND SECTIONS

SH. 55
DWG. NO. M-1
DATE: APR 1980
FILE NO. C13078.T1



CH2M HILL	DES. JLA				
	DR. MAW				
	CHK. NCW				
	APPD. AEM	NO.	DATE	REVISION	BY	APPD.

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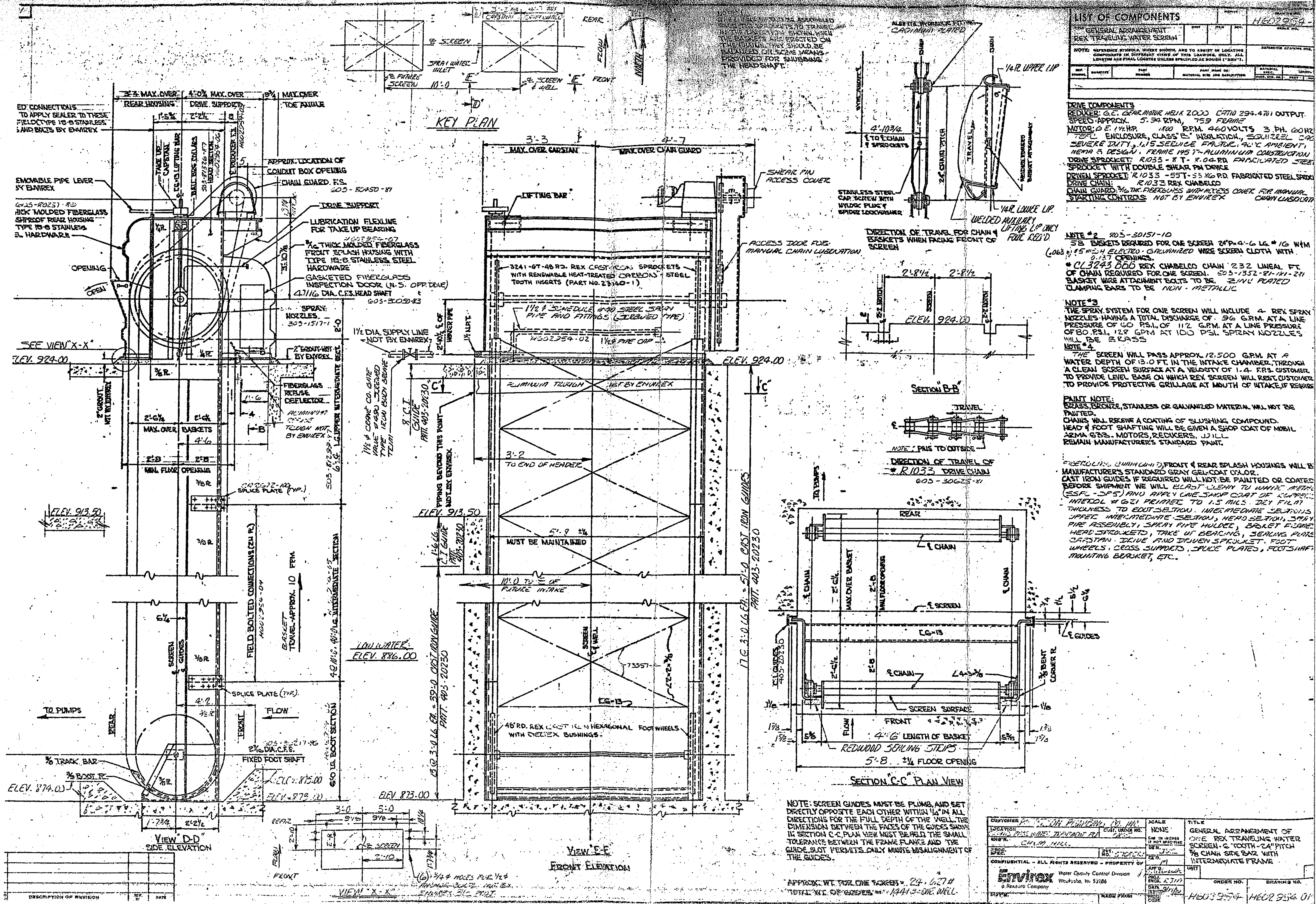
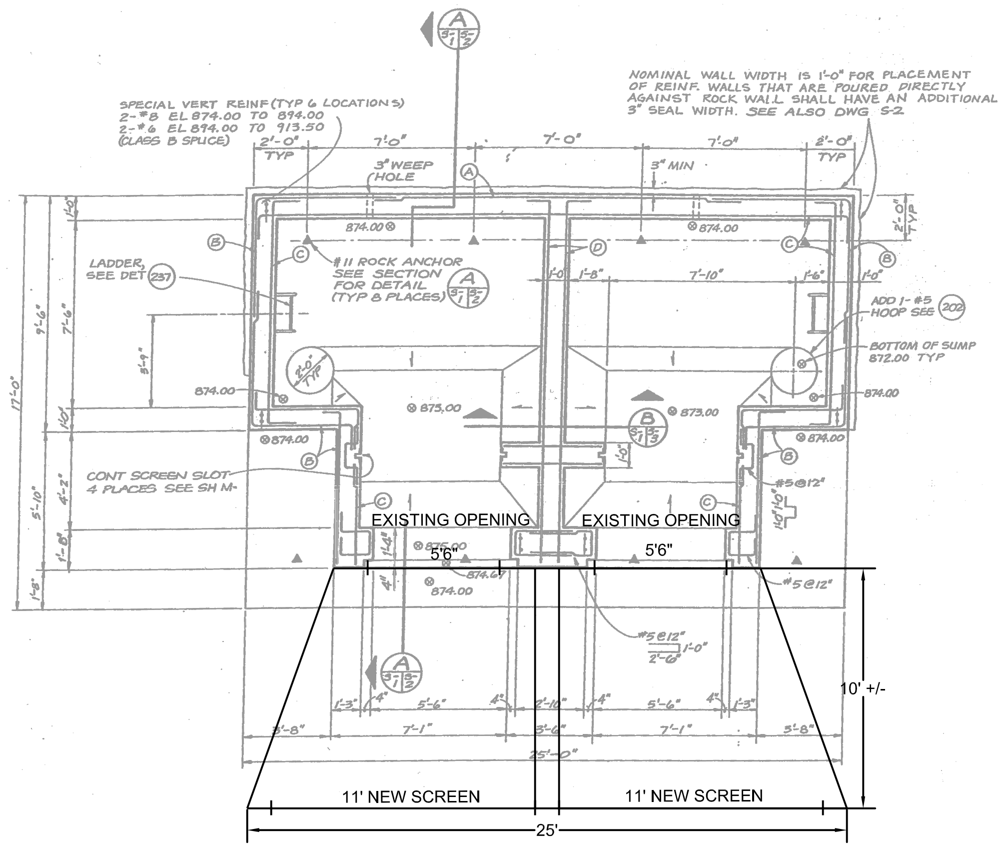
CITY OF GRANTS PASS, OREGON
WATER SYSTEM IMPROVEMENTS

MECHANICAL
RAW WATER INTAKE
PLANS AND DETAILS

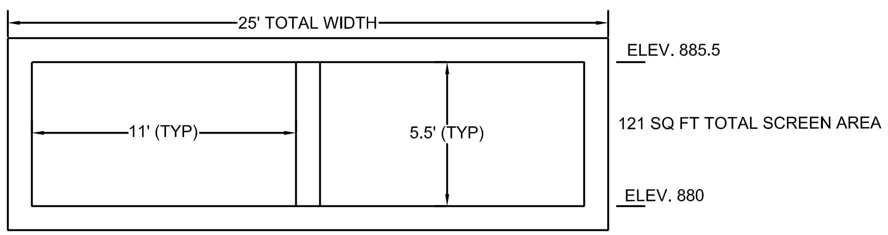
SH. 56
DWG. NO. M-2
DATE: APR 1980
FILE NO. C13078.T1



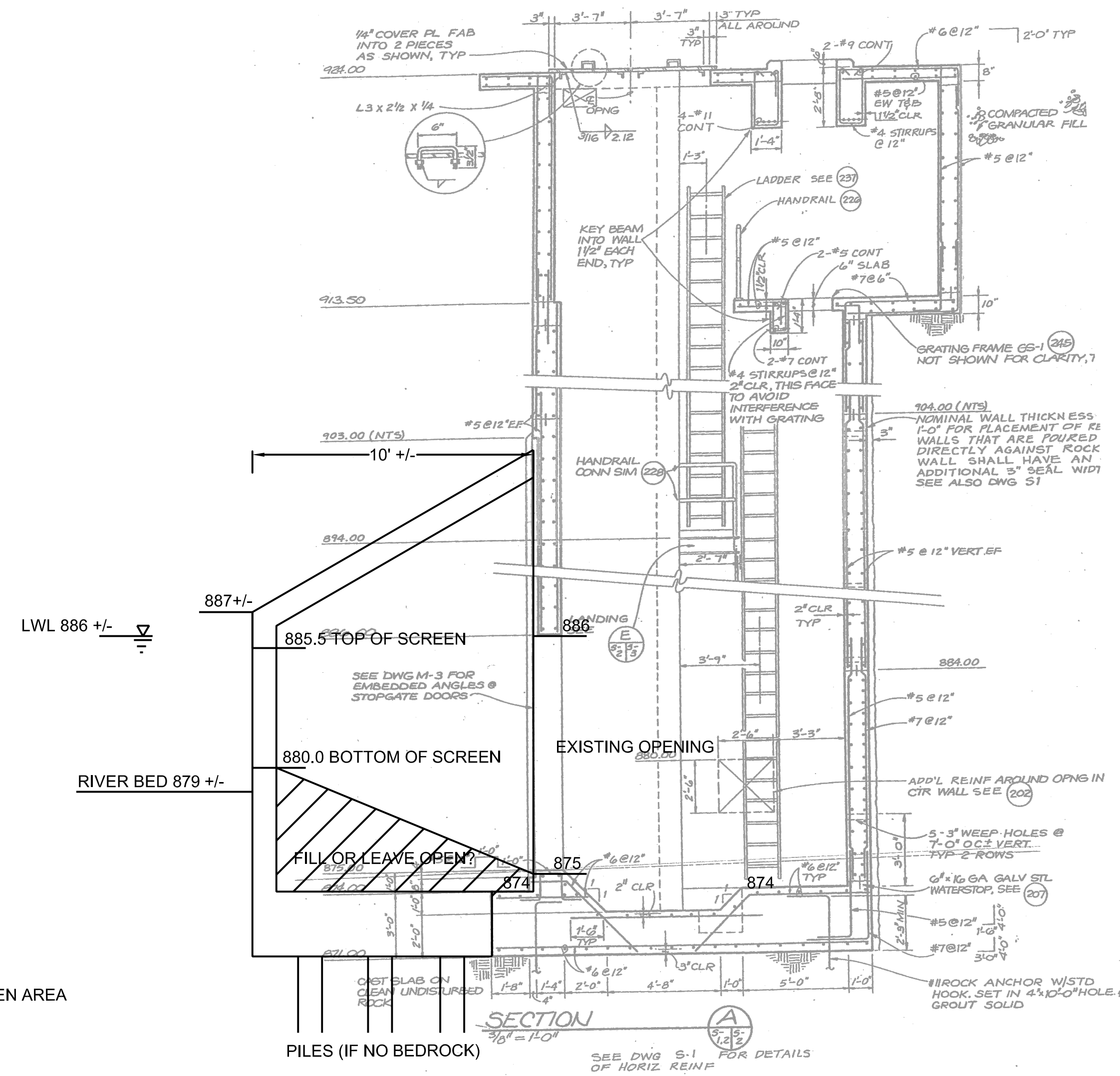
LIST OF COMPONENTS
GENERAL ARRANGEMENT REX TRAVELING WATER SCREEN
H602954
KEY PLAN
DRIVE COMPONENTS
REDUCER: G.E. GEARMOTOR HELIX 2000 CATID 294.4761 OUTPUT SPEED APPROX. 5.94 RPM, 759 FRAME
MOTOR: G.E. 1½ HP. 1800 R.P.M. 460 VOLTS 3 PH. 60 HZ TEFC ENCLOSURE, CLASS "B" INSULATION, SQUIRREL CAGE SEVERE DUTY, 1.15 SERVICE FACTOR, 40°C AMBIENT, NEMA B DESIGN, FRAME 145T - ALUMINUM CONSTRUCTION
DRIVE SPROCKET: R1033 - 8T - 8.04 P.D. FABRICATED STEEL SPROCKET WITH DOUBLE SHEAR PIN DEVICE
DRIVEN SPROCKET: R1033 - 55T - 55.76 P.D. FABRICATED STEEL SPROCKET
DRIVE CHAIN: R1033 REX CHABELCO
CHAIN GUARD: 3/16 THK. FIBERGLASS WITH ACCESS COVER FOR MANUAL CHAIN LUBRICATION
STARTING CONTROLS: NOT BY ENVIREX
NOTE #2 503-30151-10
58 BASKETS REQUIRED FOR ONE SCREEN 24" P × 4'-6 LG. # 16 W&M (.063) × ¼" MESH ELECTRO-GALVANIZED WIRE SCREEN CLOTH WITH 0.187 OPENINGS.
* CL3243 BBB REX CHABELCO CHAIN 232 LINEAL FT. OF CHAIN REQUIRED FOR ONE SCREEN. 503-1352-81-181-281
BASKET WIRE ATTACHMENT BOLTS TO BE ZINC PLATED
CLAMPING BARS TO BE NON-METALLIC
NOTE #3
THE SPRAY SYSTEM FOR ONE SCREEN WILL INCLUDE 4 REX SPRAY NOZZLES HAVING A TOTAL DISCHARGE OF 96 G.P.M. AT A LINE PRESSURE OF 60 P.S.I., OF 112 G.P.M. AT A LINE PRESSURE OF 80 P.S.I., 129 GPM AT 100 PSI. SPRAY NOZZLES WILL BE BRASS
NOTE #4
THE SCREEN WILL PASS APPROX. 12,500 G.P.M. AT A WATER DEPTH OF 13.0 FT. IN THE INTAKE CHAMBER, THROUGH A CLEAN SCREEN SURFACE AT A VELOCITY OF 1.4 F.P.S. CUSTOMER TO PROVIDE LEVEL BASE ON WHICH REX SCREEN WILL REST. CUSTOMER TO PROVIDE PROTECTIVE GRILLAGE AT MOUTH OF INTAKE, IF REQUIRED
PAINT NOTE:
BRASS, BRONZE, STAINLESS OR GALVANIZED MATERIAL WILL NOT BE PAINTED.
CHAINS WILL RECEIVE A COATING OF SLUSHING COMPOUND.
HEAD & FOOT SHAFTING WILL BE GIVEN A SHOP COAT OF MOBIL ARMA 633. MOTORS, REDUCERS, WILL REMAIN MANUFACTURER'S STANDARD PAINT.
FIBERGLASS CHAIN GUARD, FRONT & REAR SPLASH HOUSINGS WILL BE MANUFACTURER'S STANDARD GRAY GEL-COAT COLOR.
CAST IRON GUIDES IF REQUIRED WILL NOT BE PAINTED OR COATED. BEFORE SHIPMENT WE WILL BLAST CLEAN TO WHITE METAL (SSPC-SP5) AND APPLY ONE SHOP COAT OF KOPPERS INERTOL #621 PRIMER TO 1.5 MILS DRY FILM THICKNESS TO BOOT SECTION, INTERMEDIATE SECTIONS, UPPER INTERMEDIATE SECTION, HEAD SECTION, SPRAY PIPE ASSEMBLY, SPRAY PIPE HOLDER, BASKET FRAMES, HEAD SPROCKETS, TAKE UP BEARING, SEALING PLATE, CARSTAN DRIVE AND DRIVEN SPROCKET, FOOT WHEELS, CROSS SUPPORTS, SPLICE PLATES, FOOTSHAFT MOUNTING BRACKET, ETC.
SECTION "B-B"
ELEV. 924.00
DIRECTION OF TRAVEL FOR CHAIN & BASKETS WHEN FACING FRONT OF SCREEN
DIRECTION OF TRAVEL OF * R1033 DRIVE CHAIN
SECTION "C-C" PLAN VIEW
4'-6" LENGTH OF BASKET
REDWOOD SEALING STRIPS
5'-8 ±¼ FLOOR OPENING
NOTE: SCREEN GUIDES MUST BE PLUMB, AND SET DIRECTLY OPPOSITE EACH OTHER WITHIN ¼" IN ALL DIRECTIONS FOR THE FULL DEPTH OF THE WELL. THE DIMENSION BETWEEN THE FACES OF THE GUIDES SHOWN IN SECTION C-C, PLAN VIEW MUST BE HELD. THE SMALL TOLERANCE BETWEEN THE FRAME FLANGE AND THE GUIDE SLOT PERMITS ONLY MINUTE MISALIGNMENT OF THE GUIDES.
APPROX. WT. FOR ONE SCREEN = 24,627 #
VIEW "D-D" SIDE ELEVATION
VIEW "E-E" FRONT ELEVATION
VIEW "X-X"
ELEV. 913.50
LOW WATER ELEV. 886.00
ELEV. 874.0
ELEV. 873.00
3241-6T-48 P.D. REX CAST IRON SPROCKETS WITH RENEWABLE HEAT-TREATED CARBON STEEL TOOTH INSERTS (PART NO. 23140-1)
48" P.D. REX CAST IRON HEXAGONAL FOOT WHEELS WITH DELREX BUSHINGS
MUST BE MAINTAINED
FIXED FOOT SHAFT
TO PUMPS
FLOW
GENERAL ARRANGEMENT OF ONE REX TRAVELING WATER SCREEN - 6 "TOOTH-24" PITCH 5/8 CHAIN SIDE BAR WITH INTERMEDIATE FRAME
Envirex Water Quality Control Division Waukesha, Wi. 53186 a Rexnord Company
CONFIDENTIAL - ALL RIGHTS RESERVED - PROPERTY OF
H602954



INTAKE PLAN



SCREEN ELEVATION



INTAKE SECTION

Not to Scale

MWH

City of Grants Pass
Water Treatment Plant Facility Plan
Suggested Intake Improvements

M E M O R A N D U M

111 SW 5th Avenue, Suite 1770
Portland, OR 97204
(503) 226-7377
(503) 226-0023 facsimile

CITY OF GRANTS PASS
WTP FACILITY PLAN

To:	**Jason Canady**	**Date:**	**05/18/04**
From:	**Jude Grounds**	**Reviewed by:**	**Pete Kreft**
Subject:	**Results from Bench-scale Coagulation Experimentation**	**Reference:**	**1530536.010101**

INTRODUCTION

As part of the work being performed for the City of Grants Pass Water Treatment Plant Facility Plan (WTPFP), MWH has recommended that the City review and optimize the current coagulation strategy at the plant. Potential benefits from an optimized coagulation strategy include:

- Increased solids removal efficiencies in the Basins
- Improved settled water quality and filtered water quality
- Reduced filter aid polymer usage
- Reduced sludge production and pH adjustment requirements
- Longer filter runs and less backwashing
- Increased overall plant efficiencies

To assist the City in identifying alternative coagulation strategies appropriate for pilot/full scale studies, the City requested that MWH perform a series of jar tests. These tests were performed November 3 through 5, 2003. This Technical Memorandum (TM) reviews the coagulation performance at the plant over the past several years, compares the current coagulation strategy with other WTPs in the region, and presents the results from the jar test experiments. This TM is organized as follows:

- Background
- Experimental Approach
- Experimental Plan
- Results
- Discussion/Conclusions

BACKGROUND

The City of Grants Pass WTP draws water from an adjacent intake on the Rogue River. The 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), and seasonally variable turbidity, temperature, and color. This variable raw water quality can significantly impact overall performance of the coagulation, clarification and filtration processes at the plant. Inefficient coagulation performance is exacerbated by the lack of formal flocculation and continuous sludge removal equipment at the plant.

Historically, these treatment challenges have been met using a relatively high dosage of alum compared to plants treating similar raw water qualities in the region. This strategy has resulted in relatively high solids production (putting a "stress" on the existing solids handling facilities by filling up the basins and pond faster than expected after cleaning), depressed pH (corresponding to an increase in pH adjustment chemical usage/costs), higher settled water turbidities, and decreased overall plant efficiencies.

Proposed improvements to the filters and/or basins may serve to improve overall plant efficiencies also. However, optimizing the coagulation strategy at the WTP is an essential, near-term step toward increasing plant efficiencies and minimizing solids production. This section discusses some alternative coagulation strategies for the City's WTP.

Table 1 presents potential alternative coagulation schemes for the City's WTP.

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 6-8 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 Clackamas River Water WTP employs a combination of ACH + alum to decrease alum demands. The Lake Oswego WTP (also along the Clackamas River), uses PACl with alum, but limits the use of PACl to high turbidity events; the Lake Oswego WTP also uses pH adjustment with carbon dioxide to maintain optimal pH during coagulation to minimize alum usage. Similarly, the Medford WTP (on the Rogue River) is currently using alum + cationic polymer, but is considering the use of PACl alone or PACl + cationic polymer to avoid impacts of high alum doses on pH and sludge production. Finally, the City of Roseburg (on the Umpqua River) uses ACH alone at about 2 mg/L compared to alum alone at 8 mg/L (these doses are during low turbidity periods).

TABLE 1: 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 fulls-cale results; PACl may be more "robust" • Proprietary chemicals; relatively expensive
Ferric Chloride/Sulfate	• Performs better than alum in colder water • Performance similar to alum • Sludge more "manageable"; easier to dewater • Solids production similar to alum, but better for disposal to the WWTP
Alum/Poly or Ferric/Poly Blend	• Proprietary chemicals; relatively expensive • Relatively expensive vs. purchasing separately
Multiple Chemicals	
Alum + ACH/PACl	• Less pH depressions than alum alone • Lesssludge production compared to alum alone
Alum + Cationic Polymer	• Less pH depression than alum alone • 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

Though there is potential to optimize the current coagulation strategy at the Grants Pass 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), but settled water turbidity increase when raw water turbidity exceeds 10 NTU. 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, type 801b). Preliminary results from tests conducted April 10 – 19, 2003 (with an average raw water temperature of 50^{o}F and turbidities less than 5 NTU) indicated that settled water turbidity was lower and filter runs were longer compared to the use of alum alone (ACH dose of 10 to 12 mg/L compared to alum dose of 18 to 25 mg/L). However, similar testing performed in July 2003 (average raw water temperature 67^{o}F and turbidities less than 3 NTU) 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, but may have resulted from overdosing of ACH. This is discussed further in the Discussion/Conclusions portion of this TM. This conflicting performance with ACH was one main reason for conducting additional jar tests.

EXPERIMENTAL APPROACH

This bench-scale study was divided into four tasks; experimental methods used during each task are discussed briefly in the following sub-sections.

TASK I: Model Full-scale Plant Performance
TASK II: Optimization of Primary Coagulant Addition
TASK III: Optimization of Polymer Addition
TASK IV: Impacts of pH Analysis

EQUIPMENT REQUIRED:

The equipment required to perform the necessary jar-tests are listed below.

- One Phipps and Bird laboratory stirrer with regular 2-L square jars.
- Whatman No. 1 Filters
- Filterability Test Apparatus
- One bench-top turbidimeter and standard solutions
- pH meter and standard solutions
- Alcohol thermometer
- Stopwatch
- One of each; a 1, 5 and 10 mL adjustable pipette
- 100 disposable 5 mL syringes
- Chemicals: Ferric Sulfate, Ferric Chloride, Alum, ACH, PACl, Alum/Poly Blend, Ferric/Poly Blend, Cationic Polymer, Lime, Sodium Hypochlorite, Potassium Permanganate, Sulfuric Acid

Excepting the chemicals, all equipment was provided by the City. MWH, with assistance from the City, performed all of the jar test experiments. All experiments were performed at the Grants Pass WTP water quality laboratory.

Jar Tests

All jar tests were performed using the standard Phipps and Bird jar test apparatus with the six rectangular 2-liter jars provided by the City. Each of the bench-scale coagulation, flocculation and sedimentation experiments followed the standard procedure described below.

1. Two liters of raw water was added to each jar.
2. The prescribed dose of pre-oxidant(s) was added while mixing at a speed of 100 rpm.
3. The prescribed dose of coagulant was then added while mixing at a speed of 100 rpm.
4. The water was stirred at 100 rpm for about 30 seconds.
5. If a coagulant aid was used, it was added during the first 10 seconds of this rapid mix, primary coagulant was added 20 seconds later. The rapid mix duration was extended to ensure at least 30 seconds of mixing followed the primary coagulant addition.
6. The water was then be flocculated for 15 minutes. The standard flocculation mixing conditions are summarized below.
7. The water was then allowed to quiescently settle for 20 minutes.
8. 500 mL samples of settled water was then collected through a sampling tap, located at a distance of 10 cm below the water surface.

A summary of the mixing conditions to be used during Tasks I though IV of the study is provided below.

Flash Mix – 30 seconds at a mixing speed of 200 rpm (Gt = 6,000)
Stage 1 – 5.0 minutes at a mixing speed of 70 rpm (Gt = 18,000)
Stage 2 – 5.0 minutes at a mixing speed of 40 rpm (Gt = 900)
Stage 3 – 5.0 minutes at a mixing speed of 25 rpm (Gt = 4,500)
Stage 4 – 20.0 minutes of Sedimentation (Gt = 0)

This mixing regimen is designed to impart a total Gt of 2.9×10^4.

Settling Velocities

To establish the settling velocity curve for the coagulated and flocculated samples, the settled water was sampled at 10 cm below the water surface after 2, 5, 10 and 20 minutes (following mixing) in select jars. The sample volume was limited to about 20 mL which is assumed sufficient for turbidity measurement with the turbidimeter provided by the City. The distance between the surface of the settling water and the sampling port was assumed constant. Thus, the sampling times correspond to a settling velocity expressed in centimeters per minute. The sampling times of 2, 5, 10 and 20 minutes correspond to settling velocities of 5, 2, 1.0 and 0.5 cm/min., respectively. These velocities can be converted to surface loading rates of 1.2, 0.5, 0.25 and 0.12 gpm/sf, respectively. The plot for the turbidity values versus the corresponding settling velocity allows the comparison of the solids settleability for various chemical treatments, and also allows the turbidity of the settled water to be determined for a given sedimentation basin loading rate.

Filterability Analysis

Filterability of selected settled water samples was determined by filtering ~100 mL of the sample through 10um filter paper. The filter paper was rinsed with distilled water prior to the experiment to remove dust and fibers. The time required to filter 50 mL of the sample, as well as the filtered water turbidity was measured and used for alternative comparison.

Chemicals

A number of coagulants and polymers were evaluated during the bench-scale work. Their characteristics are summarized below. Pre-oxidants and pH adjustment chemicals (sulfuric acid), when used, were added prior to the start of the jar test. The primary coagulant(s) and cationic polymer (serving as the coagulant aid) addition were “lagged” during rapid mix; in general, cationic polymer addition occurred prior to addition of primary coagulant. ***Table 2*** summarizes the chemicals used throughout the course of this bench-scale study.

TABLE 2: LIST OF CHEMICALS USED DURING JAR TESTS

Product	Description	Specific Gravity	Function
Alum	Aluminum Sulfate	n/a why?	Primary Coagulant
Ferric Chloride	Iron Salt	n/awhy?	Primary Coagulant
801B ACH	Aluminum Chlorohydrate	1.34	Primary Coagulant
8187 ACH	Aluminum Chlorohydrate	1.34	Primary Coagulant
8157 PACl	Poly-aluminum Chloride	1.26	Primary Coagulant
8158 "Sulfated"	Poly-aluminum Chloride	1.21	Primary Coagulant
71264 Ferric/EPI-DMA	Ferric Chloride /Cat Poly Blend	1.4	Primary Coagulant
8185 ACH/EPI-DMA	ACH/Cat Poly Blend	1.24	Primary Coagulant
8105 EPI-DMA	Cationic Polymer	1.15	Coagulant Aid
2490 Amphoteric	Cationic/Anionic Polymer	1.08	Coagulant Aid
Bleach	Sodium Hypochlorite	1.00	Pre-Oxidation
Potassium Permanganate		n/a	Pre-Oxidation
Lime	Calcium Hydroxide	n/a	pH Adjustment
Sulfuric Acid		1.07	pH Adjustment

TASK I: MODEL FULL-SCALE PLANT PERFORMANCE

To model the existing chemical regimen at the Grants Pass WTP, one jar test was performed. Results of this jar test were used as a "baseline" for comparison with alternative chemical regimens analyzed during the evaluation. Chemicals and their corresponding concentrations used for this evaluation were similar to those used at the WTP. Mixing energies considered during this experiment are summarized below:

- Rapid Mix (Energy) – 200 and 100 rpm
- Rapid Mix Duration – 30 and 60 seconds
- Settling Time – adjusted based on full-scale conditions

To better understand the existing plant conditions, WTP basin influent and effluent samples were collected from two basins to determine the effects of additional mixing on the settled water. Filterability tests of plant settled water were performed and compared to the plant filter effluent to establish a "baseline" for future filterability tests.

During each Task I jar test, coagulant performance was evaluated using the following parameters: pH, temperature, settled water turbidity, floc formation and settling velocities.

TASK II: EVALUATE THE IMPACT OF PRIMARY COAGULANT

A series of jar tests were performed in order to evaluate the impact of coagulant type and dose on the raw water settleability. Mixing conditions presented in the Experiment Approach were used throughout Task II experiments. The coagulant(s) analyzed were:

Single Chemical
- ACH alone
- PACl alone
- Ferric Chloride alone
- Ferric Sulfate alone

Multiple Chemicals
- Alum + ACH
- Alum + PACl

Coagulant performance was evaluated using the following parameters: pH, temperature, settled water turbidity, floc formation and settling velocities.

This first round of experiments ("single chemical") was performed at coagulant doses proven effective at similar plants treating similar raw waters in the region. Based on the results of these initial tests, as well as "regional" experience with alum and ferric coagulants, a(n) optimal coagulant(s) was identified, and considered for future experiments**.**

TASK III: EVALUATE THE IMPACT OF COAGULANT AID

A series of jar tests was performed to evaluate the impact of polymer type and dose on the raw water settleability. Coagulant performance in each of these tests was evaluated using the following parameters: pH, temperature, floc formation and settleability. The chemical configurations considered for this evaluation were:

- Alum + Cationic Polymer
- Ferric + Cationic Polymer
- ACH + Cationic Polymer
- Alum + Cationic/Anionic Polymer
- Ferric + Cationic/Anionic Polymer
- Alum/Poly Blend
- Ferric/Poly Blend

When used in conjunction with an organic polymer, the optimal dose for the inorganic coagulant is typically less than the optimum observed from when the primary coagulant is used alone. For the next series of jar tests, an optimal dose of the polymer(s) was selected (and fixed), and used with varying concentrations of the inorganic coagulants optimized in Task 2; proprietary "blends" were also analyzed. Coagulant performance was evaluated using the following parameters: pH, temperature, settled water turbidity, floc formation and settling velocities.

TASK IV: IMPACTS OF pH ADJUSTMENT

Based on the findings from Task II and III, optimal chemical configurations/dosages were analyzed over a range of pH values to better quantify the impacts of diurnal swings in pH on plant performance. Coagulant performance in each of these tests was evaluated using the following parameters: pH, temperature, floc formation and settleability. Filtered water turbs?

RESULTS

EXPERIMENTAL RESULTS – TASKS I – IV

This sub-section presents a review of raw water quality data observed during the jar tests in addition to the results from jar tests performed as part of Task I through IV.

Raw Water Quality

Though attempts were made to schedule the jar testing during "challenging" water treatment conditions (i.e. elevated turbidities), raw water turbidities observed during the experiments were relatively low (less than 3 NTU), thereby limiting the ability to extrapolate full-scale implications from experimental jar test results. However, comparisons between the relative effectiveness of the various coagulation alternatives can be drawn. ***Table 3*** summarizes the raw water quality parameters observed during the jar testing.

TABLE 3: RAW WATER QUALITY OBSERVED DURING JAR TESTS

Parameter	Unit	Average	St. Dev[1]
Temperature	^{o} C	7.8	±0.0
Turbidity	NTU	1.9	±0.9
pH		7.7	±0.4
Alkalinity	mg/L (as $CaCO_3$)	39.6	±2.7

[1]Values reported at the 95th-percentile confidence interval

Results-Task I

Water quality samples from various points throughout the the full-scale plant were initially taken and used to calibrate bench-top equipment, as well as to provide a baseline for optimizing jar test experimental conditions and evaluating results. ***Table 4*** presents the full-scale water quality observed during performance of Task I at various points throughout the full-scale plant.

TABLE 4: FULL-SCALE PLANT WATER QUALITY DURING TASK I-PRELIMINARY TESTING

Sample Location	Unit	On-line Measurement	Bench-top Analysis
Raw Water	NTU	1.72	1.60
Basin 1 Influent Turbidity	NTU		2.75
Basin 1 Effluent Turbidity	NTU	0.78	0.85
Filters 1 – 3 Turbidity	NTU	0.02[1]	
Basin 3 Influent Turbidity	NTU		2.43
Basin 3 Effluent Turbidity	NTU	1.31	1.43
Filters 6 – 8 Turbidity	NTU	0.02[1]	

[1]Values represent the average of on-line turbidimeter readings.

To model the full-scale plant performance, several jar tests were performed to optimize the mixing energy, chemical addition sequence and settling times used throughout the remaining jar tests. The final testing protocol resulting from these experiments is outlined in the experimental approach portion of this TM.

Additional experiments used to verify the benefits of adding flocculation to the full-scale plant were also performed. These tests involved performing the jar tests on water samples collected from various points throughout the full-scale plant. Results from these experiments are presented in ***Table 5***.

TABLE 5: PRELIMINARY TESTING – JAR TESTING FULL-SCALE SAMPLES

Sample Location	Unit	Settled Water Turbidity[1]	Filtered Water Turbidity[2]
Basin 1 Influent Turbidity	NTU	1.58	
Basin 1 Effluent Turbidity	NTU	0.61	
Basin 3 Influent Turbidity	NTU	0.85	
Basin 3 Effluent Turbidity	NTU	1.15	0.12

[1]Following a Jar Test, as defined in the Experimental Approach
[2]Following a Filterability Analysis, as defined in the Experimental Approach

Comparing these results with those presented in ***Table 4***, a measurable benefit (lower turbidity) was observed from the additional mixing, thereby supporting the recommendation to add formal flocculation to the basins as a major Capital Improvement. Though the difference appears slight, the impact of additional mixing will likely increase with increasing raw water turbidities. Also, the results from the filterability test are significantly higher than those observed in the full-scale filters. This test can be used as an indicator to gauge relative "filterability" of settled waters, but should not be used to predict full-scale filtered water turbidities resulting from the various chemical configurations tested.

Results-Task II

Results from jar tests performed to evaluate the impact of coagulant type and dose on the raw water settleability are presented below in ***Table 6***. Only the "optimal" results from each series of jar tests are summarized; complete experimental results can be found in the ***Appendix***.

TABLE 6: IMPACTS OF COAGULANT TYPE AND DOSE ON RAW WATER SETTLEABILITY

Coagulant Type	Dose (mg/L)	Settled Water Turbidity (NTU)[1]	Filtered Water Turbidity (NTU)[2]
Full-Scale Plant (Alum)	19	0.85 – 1.43	0.02
Simulation of Full-Scale (Alum)	19	1.6	0.18
ACH (801b)	7.0	1.5	0.15
ACH (8187)	9.0	1.5	0.26
PACl (8157)	25	0.83	0.14
PACl (8158)	25	0.53	0.16
$FeCl_3$	10	0.98	0.12
Alum + ACH	6.0 + 3.0	1.3	0.24
Alum + PACl (8158)	10 + 10	0.65	0.16

[1]Following a Jar Test, as defined in the Experimental Approach
[2]Following a Filterability Analysis, as defined in the Experimental Approach

In comparing the results from Task II experiments with that of the full-scale plant performance and the jar-simulated plant performance, most chemical combinations were able to achieve desirable settled water turbidities. However, the coagulant dosages required were relatively high.

Results-Task III

Results from jar tests performed to evaluate the impact of polymer type and dose on the raw water settleability are summarized below in ***Table 7***. Only the "optimal" results from each series of jar tests are summarized; complete experimental results can be found in the ***Appendix***.

TABLE 7: IMPACTS OF COAGULANT TYPE AND DOSE ON RAW WATER SETTLEABILITY

Coagulant Type	Dose (mg/L)	Settled Water Turbidity (NTU)[1]	Filtered Water Turbidity (NTU)[2]
Full-Scale Plant (Alum)	19	0.85 – 1.43	0.02
Simulation of Full-Scale (Alum)	19	1.6	0.18
Alum + Cat Poly (8105)	14 + 0.8	3.6	0.19
Ferric + Cat Poly (8105)	10 + 1	1.1	0.23
Ferric/Cat Poly Blend (71264)	3 - 18	No floc formation	N/A
ACH/Cat Poly Blend (8185)	6 - 21	No floc formation	N/A
Alum + Cat/Anionic Poly (2490)	14 + 0.8	2.0	0.12
Ferric + Cat/Anionic Poly (2490)	10 + 0.6	0.81	0.09

[1]Following a Jar Test, as defined in the Experimental Approach
[2]Following a Filterability Analysis, as defined in the Experimental Approach

Though desirable settled water quality was achieved using several of the chemical configurations, none effectively reduced the overall dose of primary coagulant. Of the chemical configurations tested, the ferric/cationic polymer showed the most potential.

Results-Task IV

Four series of jar tests were performed using differing chemical configurations over a broad range of pH, and results from these tests were inconclusive. The relatively low raw water turbidities made discerning subtle differences in settled water turbidities difficult. Results from these tests are presented in the ***Appendix***. It is recommended that these tests be performed during periods of "challenging" raw water quality.

DISCUSSION/CONCLUSIONS

The results of the jar testing were mostly inconclusive which perhaps illustrates limitations with jar testing under certain water quality and chemical conditions as much as anything. Jar testing is an excellent tool for evaluating settled water quality under a wide range of chemical conditions if the raw water turbidity is high enough. It is difficult to mimic filter performance using jars and filter paper.

However, full-scale testing by plant staff with alternative coagulants, specifically ACH (Pelican Chemicals, type 801b), in April 2003 (before the jar testing) and in January 2004 (after the jar testing) have determined that use of a different coagulation scheme offers an opportunity to optimize treatment performance and potentially reduce operating costs. Other regional utilities with similar sources, including Roseburg and Clackamas River Water, have been successfully using a similar chemical for as long as 3 years. It is quite possible that the unsuccessful performance of ACH during full-scale tests in the summer 2003 was the result of overdosing, based on results observed during the January 2004 testing.

Based on the positive results observed with ACH during the two successful full-scale plant trials, the following economic analysis is presented below using an annualized approach. The average annual plant production was assumed to be 5.0 mgd based on recent records.

- Current annual alum cost = $32,000/year based on average dose of 24 mg/L at a unit cost of $0.09/lb

- Projected annual cost using ACH = $65,000/year based on average dose of 10 mg/L at a unit cost of $0.45/lb

- Projected annual <u>reduction</u> in lime costs = $3,000/year based on current average lime dose of 4 mg/L and future lime dose of 1 mg/L when using ACH, at a unit cost of $0.075/lb

- Projected annual <u>reduction</u> in filter aid polymer costs = $300/year based on current average polymer dose of 0.03 mg/L and a future polymer dose of 0.015 mg/L when using ACH, at a unit cost of $1.25/lb

- Projected annual <u>reduction</u> in power costs due to less-frequent backwash pumping (due to longer filter runs between backwashes when using ACH) = $1,000/year based on current average of 6 backwashes per day and a future 5 backwashes per day when using ACH, at a unit power cost of $0.05/kw-hr.

- Projected annual <u>reduction</u> in plant operating costs due to less raw water pumping and treatment to produce the required volume of finished water (due to longer filter runs and less backwash water usage when using ACH) = $25,000/year,based on an incremental unit production cost of $0.10/1,000 gallons and a 15% increase in filter production efficiencies.

- Projected annual <u>reduction</u> in solids handling and removal costs due to reduced solids volume and easier sludge dewatering when using ACH = $8,000/year, based 20% solids reduction and estimated current annual sludge handling and removal costs of $40,000/year (using dredge and Geo-Tube approach to be initiated in Spring 2004).

Based on this preliminary analysis, the use of ACH to replace alum as the primary coagulant offers the ability to <u>reduce plant operating costs by a few thousand dollars per year</u> and perhaps more if the actual purchase price of the ACH is less than $0.45/lb used in this analysis, and/or the average ACH dose can be lowered below 10 mg/L.

To fully understand the possible benefits and cost impact of using alternative coagulants, additional 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.

It is therefore recommended that the plant continue to experiment with ACH and/or PACl, either using pilot-scale filters or with the full-scale plant, with the goal of minimizing solids production, reducing settled water turbidities, lengthening filter runs, reducing the lime dose required for pH adjustment, and perhaps reducing filter aid polymer dose. Until the plant has had the ability to demonstrate acceptable performance with ACH and/or PACl during all seasons of the year, under variable water quality conditions, the plant should retain its alum storage and feed capabilities.

Results Summary

			Chemicals		Settled Water		Filtered Water	
Trial #	**Jar #**	**Description**	**Coag. Dose** (mg/L)	**Poly Dose** (mg/L)	**Turbidity** (NTU)	**pH**	**Turbidity** (NTU)	**Remarks**
TASK I								
1		**Preliminary Test**						
	1	Basin 1 Effluent			0.61	6.98		
	2	Mixing Basin			1.58	6.98		
	3	Basin 3 Effluent			1.15	6.98		
	4	Basin 3 Influent			0.85	6.98	0.12	
2		**Simulation of Full-Scale**						
	1	Alum Only	13		2.1			Pinpoint/Small (0.5 - 0.75 mm)
	2	Alum Only	15		2			Medium/Small (1.0 - 1.5 mm)
	3	Alum Only	17		1.7			Medium (1.5 - 2.0 mm)
	4	Alum Only	19		1.6		0.18	Medium (1.5 - 2.0 mm)
	5	Alum Only	21		1.7			Medium (1.5 - 2.0 mm)
	6	Alum Only	23		1.6			0
TASK II								
3		**Prev. Tested ACH**						
	1	ACH	2		1.5	7.8		no visible floc, no settling
	2	ACH	3		1.7	7.8		Pinpoint (0.3 - 0.5 mm), no settling
	3	ACH	4		1.7	7.79		Pinpoint (0.3 - 0.5 mm), no settling
	4	ACH	5		1.7	7.57		Pinpoint (0.3 - 0.5 mm)
	5	ACH	6		1.6	7.63	0.21	Small (0.75 - 1.0 mm)
	6	ACH	7		1.5	7.7	0.15	Medium/Small (1.0 - 1.5 mm)
4		**8187 (ACH) Alone**						
	1	ACH	4		1.6	7.64		no visible floc, no settling
	2	ACH	5		1.6	7.66		Pinpoint (0.3 - 0.5 mm), no settling
	3	ACH	6		1.6	7.84		Pinpoint (0.3 - 0.5 mm), no settling
	4	ACH	7		1.6	7.78	0.74	Small (0.75 - 1.0 mm)
	5	ACH	8		1.6	7.7	0.61	Small (0.75 - 1.0 mm)
	6	ACH	9		1.5	7.85	0.26	Medium/Small (1.0 - 1.5 mm)
5		**8157 (PACl) Alone**						
	1	PACl	5					no visible floc, no settling
	2	PACl	7					no visible floc, no settling
	3	PACl	9					no visible floc, no settling
	4	PACl	11					no visible floc, no settling
	5	PACl	13					no visible floc, no settling
	6	PACl	15					no visible floc, no settling
6		**Ferric Chloride Alone**						
	1	FeCl3	10		0.98	6.7	0.12	Medium
	2	FeCl3	12		0.55	6.82	0.13	Medium/Large
	3	FeCl3	14		0.44	6.84	0.11	Large
	4	FeCl3	16		0.38	6.79		Large
	5	FeCl3	18		0.28	6.77		Large
	6	FeCl3	20		0.24	6.67		Large
7		**Alum + ACH**						
	1	Alum/ACH	6	1	1.7	6.83		no visible flock formed
	2	Alum/ACH	6	2	1.6	7.2		medium/small
	3	Alum/ACH	6	3	1.3	7.29	0.24	medium
	4	Alum/ACH	4	1.75	1.7	7.23		pinpoint floc
	5	Alum/ACH	8	1.25	1.6	7.2		medium/small
	6	Alum/ACH	10	1	1.9	7.15		small
TASK III								
8		**Alum + CatPoly**						
	1	Alum/8105	10	0.8	3.7	6.68		Pinpoint; minimal floc formation
	2	Alum/8105	14	0.8	3.6	7.06	0.19	Small; similar to jar 3
	3	Alum/8105	18	0.8	3.9	7.14	0.19	Small
	4	Alum/8105	10	1.2	3.9	7.02		Pinpoint; minimal floc formation
	5	Alum/8105	14	1.2	3.6	7.1		Small; similar to jar 3
	6	Alum/8105	18	1.2	4.3	7.02		Small
9		**Ferric/CatPoly Blend (Proprietary)**						
	1	71264	3					No Visible Floc Formed, test aborted
	2	71264	6					No Visible Floc Formed, test aborted
	3	71264	9					No Visible Floc Formed, test aborted
	4	71264	12					No Visible Floc Formed, test aborted
	5	71264	15					No Visible Floc Formed, test aborted
	6	71264	18					No Visible Floc Formed, test aborted
10		**Ferric/CatPOly**						
	1	Ferric/8105	4	1	3	6.99		No visible floc formed
	2	Ferric/8105	6	1	3.3	7.11		Pinpoint floc; minimal floc formed
	3	Ferric/8105	8	1	3.4	7.09	0.33	Pinpoint floc; no signs of settling
	4	Ferric/8105	10	1	1.1	6.68	0.23	Medium/Large floc
	5	Ferric/8105	12	1	0.6	6.55		Large floc
	6	Ferric/8105	14	1	0.6	6.5		Large floc
11		**ACH/CatPoly Blend (Proprietary)**						
	1	8185	6					No Visible Floc Formed, test aborted
	2	8185	9					No Visible Floc Formed, test aborted
	3	8185	12					No Visible Floc Formed, test aborted
	4	8185	15					No Visible Floc Formed, test aborted
	5	8185	18					No Visible Floc Formed, test aborted
	6	8185	21					No Visible Floc Formed, test aborted
12		**Alum + Cationic/Anionic Poly**						
	1	Alum/2490	10	0.8	2.2	7.19		Pinpoint
	2	Alum/2490	12	0.8	2.4	7.17	0.63	Small; settling ok
	3	Alum/2490	14	0.8	2	7.15	0.12	Small; settling ok
	4	Alum/2490	16	0.8	2.2	7.1		Small; settling ok
	5	Alum/2490	18	0.8	2.3	7		Small; settling ok
	6	Alum/2490	20	0.8	2.2	6.9		Small; settling ok
13		**Ferric + Cationic/Anionic Poly**						
	1	Ferric/2490	8	0.6	3.15	7		No visible floc formation
	2	Ferric/2490	9	0.6	1.72	7		Medium/small (1.0 - 1.5 mm)
	3	Ferric/2490	10	0.6	0.81	7	0.09	Medium (1.5 - 2.25 mm)
	4	Ferric/2490	8	1	1.85	7		Medium/small (1.0 - 1.5 mm)

Results Summary

	5	Ferric/2490	9	1	0.88	7	0.13	Medium (1.5 - 2.25 mm)
	6	Ferric/2490	10	1	0.62	7		Medium (1.5 - 2.25 mm)
14	**8157 (PACl) Alone**							
	1	PACl	15		2.22	7.45		No floc formation
	2	PACl	20		1.43	7.44		Pinpoint/Small
	3	PACl	25		0.83	7.44		Small
	4	PACl	30		0.61	7.41	0.14	Medium/Small
	5	PACl	35		0.55	7.37	0.25	Medium
	6	PACl	40		0.28	7.34		Medium
15	**8187 (ACH) Alone**							
	1	ACH	10					Pinpoint; no settling, test aborted
	2	ACH	15					Pinpoint; no settling, test aborted
	3	ACH	20					Pinpoint; no settling, test aborted
	4	ACH	25					No visible floc formation; test aborted
	5	ACH	30					No visible floc formation; test aborted
	6	ACH	35					No visible floc formation; test aborted
16	**8158 ("Sulfated" PACl) Alone**							
	1	PACl	15		2.3	7.51		No apparent floc formation
	2	PACl	20		1.9	7.41		Pinpoint
	3	PACl	25		0.53	7.34	0.16	Medium/Small
	4	PACl	30		0.29	7.31	0.08	Medium
	5	PACl	35		0.31	7.32		Medium
	6	PACl	40		0.25	7.22		Large
17	**Alum + 8158 ("Sulfated" PACl)**							
	1	Alum/8158	6	15	0.54	6.18		Large; superior settling
	2	Alum/8158	8	15	0.54	7.09		Large; acceptable settling
	3	Alum/8158	12	15	0.44	6.98		Large; good settling
	4	Alum/8158	10	10	0.65	6.88	0.16	Large; acceptable settling
	5	Alum/8158	10	15	0.52	6.86		Large; good settling
	6	Alum/8158	10	20	0.34	6.89		Large; superior settling
18	**Alum + 8158 ("Sulfated" PACl)**							
	1	Alum/8158	6	6	1.9	7.2		No visible floc formation
	2	Alum/8158	10	6	1.6	7.02	0.2	Pinpoint/Small
	3	Alum/8158	14	6	0.96	6.75		Medium/Small
	4	Alum/8158	10	4	2	7.11		Pinpoint/Small
	5	Alum/8158	10	8	0.7	7.11	0.22	Medium/Small
	6	Alum/8158	10	10	0.6	7.12		Medium
19	**Alum + 8158 ("Sulfated" PACl) - pH Variant**							
	1	pH = 6.9	10	7	0.76	6.67		Medium
	2	pH = 7.14	10	7	0.7	6.92		Medium
	3	pH = 7.38	10	7	0.9	7		Medium
	4	pH = 7.62	10	7	0.72	7.06		Medium
	5	pH = 7.86	10	7	0.89	7.08		Medium
	6	pH = 8.1	10	7	0.9	7.1		Medium/Small
20	**Alum Alone - pH Variant**							
	1	pH = 6.9	18		2	6.8		Small; unsettleable
	2	pH = 7.14	18		2.5	6.9		Small; unsettleable
	3	pH = 7.38	18		2.1	6.9		Small; unsettleable
	4	pH = 7.62	18		2	6.9		Small; unsettleable
	5	pH = 7.86	18		2	6.95		Small; unsettleable
	6	pH = 8.1	18		2.2	7.04		Small; unsettleable
21	**8158 ("Sulfated" PACl) Alone - pH Variant**							
	1	pH = 6.9	25		0.5	7		Medium; very settleable
	2	pH = 7.14	25		0.5	7.13		Medium; very settleable
	3	pH = 7.38	25		0.5	7.28		Medium; very settleable
	4	pH = 7.62	25		0.5	7.3		Medium; very settleable
	5	pH = 8.1	25		0.5	7.36		Medium; very settleable
	6	pH = 8.3	25		0.5	7.43		Medium; very settleable
22	**Ferric + Cationic/Anionic Poly**							
	1	pH = 6.9	10	0.5	0	0		Medium/Small
	2	pH = 7.14	10	0.5	0	0		Medium/Small
	3	pH = 7.38	10	0.5	0	0		Medium
	4	pH = 7.62	10	0.5	0	0		Medium
	5	pH = 8.1	10	0.5	0	0		Pinpoint/almost no visible floc (missed chemical?)
	6	pH = 8.3	10	0.5	0	0		Medium

Preliminary Testing/Task I

Experiment # **1**

Date	11/3/2003
Time	9:45
Operator	J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	100	0.5	3000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	40	0
Total			26400

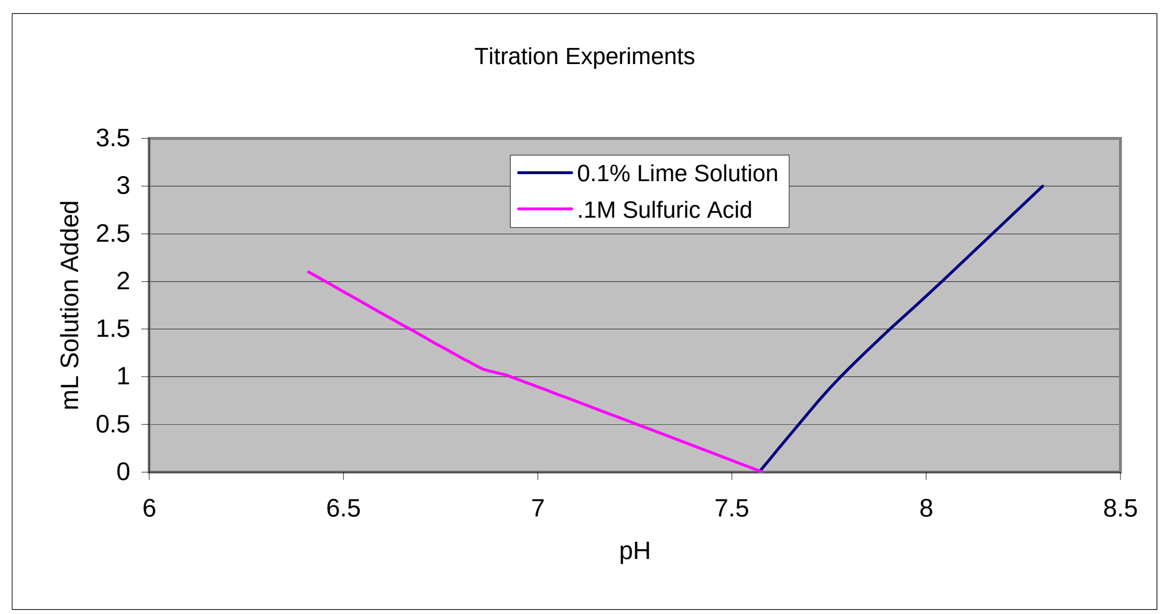
Raw Water Quality		
Temperatui	7.77	(deg C)
pH	7.5	
Alkalinity	37.9	mg/L as CaCO3

Jar Results		"Unmixed" Turbidity (NTU)	pH (mg/L)	"Mixed" Turbidity (NTU)	"Filtered" Turbidity (NTU)	On-line Data Turbidity	Remarks
No 1							
No 2	Basin 1 Effluent	0.85	6.98	0.61		0.78	Trace of pin floc (0.5 - 0.75 mm); little settling
No 3	Mixing Basin	2.75	6.98	1.58			Medium floc, settleable
No 4	Basin 3 Effluent	1.43	6.98	1.15		1.31	Medium/small floc; not particularly settleable
No 5	Basin 3 Influent	2.43	6.98	0.85	0.12	0.02	Medium floc, settleable
No 6							

Titration Experiments

pH	mL 0.1M Sulfuric Acid
7.58	0
6.93	1
6.85	1.1
6.41	2.1

pH	mL 0.1% lime Solution
7.57	0
7.78	1
8.04	2
8.3	3



Task II Results

Experiment # **2** Existing Conditions

Date	11/3/2003
Time	11:15
Operator	J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	100	0.5	3000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			26400

Jar Results	Raw Water					Chemicals					Settled Water			Filterability
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	Turbidity (NTU)
No 1	7.77	1.60	7.7	37.9		4.5	0.23	13			2.1		Pinpoint/Small (0.5 - 0.75 mm)	
No 2	7.77	1.60	7.7	37.9		4.5	0.23	15			2		Medium/Small (1.0 - 1.5 mm)	
No 3	7.77	1.60	7.7	37.9		4.5	0.23	17			1.7		Medium (1.5 - 2.0 mm)	
No 4	7.77	1.60	7.7	37.9		4.5	0.23	19			1.6	7.1	Medium (1.5 - 2.0 mm)	0.18
No 5	7.77	1.60	7.7	37.9		4.5	0.23	21			1.7		Medium (1.5 - 2.0 mm)	
No 6	7.77	1.60	7.7	37.9		4.5	0.23	23			1.6			

NOTE: Settling time reduced to 20 minutes for this experiment.

Experiment # **3** 801B Alone

Date	11/3/2003
Time	1:20
Operator	J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	100	0.5	3000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			26400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	801B (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.60	7.7	37.9		4.5	0.09	2			1.5	7.8	no visible floc, no settling				
No 2	7.77	1.60	7.7	37.9		4.5	0.09	3			1.7	7.8	Pinpoint (0.3 - 0.5 mm), no settling				
No 3	7.77	1.60	7.7	37.9		4.5	0.09	4			1.7	7.79	Pinpoint (0.3 - 0.5 mm), no settling				
No 4	7.77	1.60	7.7	37.9		4.5	0.09	5			1.7	7.57	Pinpoint (0.3 - 0.5 mm)	1.8	1.8	1.7	
No 5	7.77	1.60	7.7	37.9		4.5	0.09	6			1.6	7.63	Small (0.75 - 1.0 mm)	1.8	1.8	1.7	0.21
No 6	7.77	1.60	7.7	37.9		4.5	0.09	7			1.5	7.7	Medium/Small (1.0 - 1.5 mm)	1.8	1.7	1.7	0.15

Experiment # **4** 8187 Alone

Date	11/3/2003
Time	3:00
Operator	J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	100	0.5	3000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			26400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8187 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.60	7.7	37.9		4.5	0.09	4			1.6	7.64	no visible floc, no settling				
No 2	7.77	1.60	7.7	37.9		4.5	0.09	5			1.6	7.66	Pinpoint (0.3 - 0.5 mm), no settling				
No 3	7.77	1.60	7.7	37.9		4.5	0.09	6			1.6	7.84	Pinpoint (0.3 - 0.5 mm), no settling				
No 4	7.77	1.60	7.7	37.9		4.5	0.09	7			1.6	7.78	Small (0.75 - 1.0 mm)	1.7	1.7	1.6	0.74
No 5	7.77	1.60	7.7	37.9		4.5	0.09	8			1.6	7.7	Small (0.75 - 1.0 mm)	1.7	1.7	1.6	0.61
No 6	7.77	1.60	7.7	37.9		4.5	0.09	9			1.5	7.85	Medium/Small (1.0 - 1.5 mm)	1.7	1.6	1.6	0.26

Task II Results

Experiment # **5** 8157 (PACl) Alone

Date 11/3/2003
Time 4:15
Operator J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	0.5	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8157 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.58	7.7	37.9		3.5	0.09	5					no visible floc, no settling				
No 2	7.77	1.58	7.7	37.9		3.5	0.09	7					no visible floc, no settling				
No 3	7.77	1.58	7.7	37.9		3.5	0.09	9					no visible floc, no settling				
No 4	7.77	1.58	7.7	37.9		3.5	0.09	11					no visible floc, no settling				
No 5	7.77	1.58	7.7	37.9		3.5	0.09	13					no visible floc, no settling				
No 6	7.77	1.58	7.7	37.9		3.5	0.09	15					no visible floc, no settling				

Experiment # **6** Ferric Chloride Alone

Date 11/3/2003
Time 5:30
Operator J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	0.5	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Ferric Cl (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.60	7.7	37.9		2.5	0.09	10			0.98	6.7	Medium	2.67	1.6	1.3	0.12
No 2	7.77	1.60	7.7	37.9		2.5	0.09	12			0.55	6.82	Medium/Large	2.64	1.22	0.7	0.13
No 3	7.77	1.60	7.7	37.9		2.5	0.09	14			0.44	6.84	Large	2.7	1.09	0.62	0.11
No 4	7.77	1.60	7.7	37.9		2.5	0.09	16			0.38	6.79	Large				
No 5	7.77	1.60	7.7	37.9		2.5	0.09	18			0.28	6.77	Large				
No 6	7.77	1.60	7.7	37.9		2.5	0.09	20			0.24	6.67	Large				

Experiment # **7** Alum + ACH

Date 11/3/2003
Time 5:30
Operator J/J

Mixing Conditions	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	0.5	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	ACH (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.60	7.8	37.9		2.5	0.09	6	1		1.7	6.83	no visible flock formed				
No 2	7.77	1.60	7.8	37.9		2.5	0.09	6	2		1.6	7.2	medium/small				
No 3	7.77	1.60	7.8	37.9		2.5	0.09	6	3		1.3	7.29	medium				0.24
No 4	7.77	1.60	7.8	37.9		2.5	0.09	4	1.75		1.7	7.23	pinpoint floc				
No 5	7.77	1.60	7.8	37.9		2.5	0.09	8	1.25		1.6	7.2	medium/small				
No 6	7.77	1.60	7.8	37.9		2.5	0.09	10	1		1.9	7.15	small				

Task IIi Results

Experiment # **8** Alum + CatPoly

Date 11/4/2003
Time 8:30
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

CatPoly at T0, Alum at T30

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered	
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	8105 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)	
No 1	7.77	3.00	7.5	39.8		2.5	0.09	10	0.8		3.7	6.68	Pinpoint; minimal floc formation					NOTE: All jars accidentally rapidly mixed prior to sedimentation;
No 2	7.77	3.00	7.5	39.8		2.5	0.09	14	0.8		3.6	7.06	Small; similar to jar 3	3.9	3.8	3.8	0.19	floc was sheared, but was allowed to resettle.
No 3	7.77	3.00	7.5	39.8		2.5	0.09	18	0.8		3.9	7.14	Small				0.19	NOTE: Jar 2 seemed to filter the same as Jar 3; performance similar with ~4mg/L less alun
No 4	7.77	3.00	7.5	39.8		2.5	0.09	10	1.2		3.9	7.02	Pinpoint; minimal floc formation					
No 5	7.77	3.00	7.5	39.8		2.5	0.09	14	1.2		3.6	7.1	Small; similar to jar 3	4.3	4	3.9		
No 6	7.77	3.00	7.5	39.8		2.5	0.09	18	1.2		4.3	7.02	Small					

Experiment # **9** 71264

Date 11/4/2003
Time 10:00
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	71264 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	2.71	7.5	39.8		2.5	0.09	3					No Visible Floc Formed, test aborted				
No 2	7.77	2.71	7.5	39.8		2.5	0.09	6					No Visible Floc Formed, test aborted				
No 3	7.77	2.71	7.5	39.8		2.5	0.09	9					No Visible Floc Formed, test aborted				
No 4	7.77	2.71	7.5	39.8		2.5	0.09	12					No Visible Floc Formed, test aborted				
No 5	7.77	2.71	7.5	39.8		2.5	0.09	15					No Visible Floc Formed, test aborted				
No 6	7.77	2.71	7.5	39.8		2.5	0.09	18					No Visible Floc Formed, test aborted				

Experiment # **10** Ferric + Cat Poly

Date 11/4/2003
Time 10:30
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

CatPoly at T0, Ferric at T30

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered	
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	FeCl3 (mg/L)	8105 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)	
No 1	7.77	2.51	7.5	39.8		2.5	0.09	4	1		3	6.99	No visible floc formed					
No 2	7.77	2.51	7.5	39.8		2.5	0.09	6	1		3.3	7.11	Pinpoint floc; minimal floc formed					
No 3	7.77	2.51	7.5	39.8		2.5	0.09	8	1		3.4	7.09	Pinpoint floc; no signs of settling	4.1	3.9	3.6	0.33	NOTE: No visible signs of settling
No 4	7.77	2.51	7.5	39.8		2.5	0.09	10	1		1.1	6.68	Medium/Large floc	2.7	1.9	1.4	0.23	NOTE: Settled water is "clear"
No 5	7.77	2.51	7.5	39.8		2.5	0.09	12	1		0.6	6.55	Large floc	3.6	0.9	1.1		NOTE: Iron carryover in the settled water; slight yellow color
No 6	7.77	2.51	7.5	39.8		2.5	0.09	14	1		0.6	6.5	Large floc					

Experiment # **11** 8185

Date 11/4/2003
Time 1:30
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000

Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8185 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	2.07	7.5	39.5		2.5	0.09	6					No Visible Floc Formed, test aborted				
No 2	7.77	2.07	7.5	39.5		2.5	0.09	9					No Visible Floc Formed, test aborted				
No 3	7.77	2.07	7.5	39.5		2.5	0.09	12					No Visible Floc Formed, test aborted				
No 4	7.77	2.07	7.5	39.5		2.5	0.09	15					No Visible Floc Formed, test aborted				
No 5	7.77	2.07	7.5	39.5		2.5	0.09	18					No Visible Floc Formed, test aborted				
No 6	7.77	2.07	7.5	39.5		2.5	0.09	21					No Visible Floc Formed, test aborted				

Experiment # **12** Alum + 2490

Date 11/4/2003
Time 2:00
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Alum at T0, 2490 at T30

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	2490 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	2.03	7.6	39.8		2.5	0.09	10	0.8		2.2	7.19	Pinpoint				
No 2	7.77	2.03	7.6	39.8		2.5	0.09	12	0.8		2.4	7.17	Small; settling ok	2.9	2.7	2.9	0.63
No 3	7.77	2.03	7.6	39.8		2.5	0.09	14	0.8		2	7.15	Small; settling ok	2.8	2.9	2.3	0.12
No 4	7.77	2.03	7.6	39.8		2.5	0.09	16	0.8		2.2	7.1	Small; settling ok				
No 5	7.77	2.03	7.6	39.8		2.5	0.09	18	0.8		2.3	7	Small; settling ok				
No 6	7.77	2.03	7.6	39.8		2.5	0.09	20	0.8		2.2	6.9	Small; settling ok				

Experiment # **13** Ferric + 2490

Date 11/4/2003
Time 3:00
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Ferric at T0, 2490 at T30

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Ferric (mg/L)	2490 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.91	7.6	39.8		2.5	0.09	8	0.6		3.15	7	No visible floc formation				
No 2	7.77	1.91	7.6	39.8		2.5	0.09	9	0.6		1.72	7	Medium/small (1.0 - 1.5 mm)				
No 3	7.77	1.91	7.6	39.8		2.5	0.09	10	0.6		0.81	7	Medium (1.5 - 2.25 mm)				0.09
No 4	7.77	1.91	7.6	39.8		2.5	0.09	8	1		1.85	7	Medium/small (1.0 - 1.5 mm)				
No 5	7.77	1.91	7.6	39.8		2.5	0.09	9	1		0.88	7	Medium (1.5 - 2.25 mm)				0.13
No 6	7.77	1.91	7.6	39.8		2.5	0.09	10	1		0.62	7	Medium (1.5 - 2.25 mm)				

Task IIi Results

Experiment # **14** PACl Alone

Date	11/4/2003
Time	5:00
Operator	J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8157 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.79	7.6	39.8		2.5	0.09	15			2.22	7.45	No floc formation				
No 2	7.77	1.79	7.6	39.8		2.5	0.09	20			1.43	7.44	Pinpoint/Small				
No 3	7.77	1.79	7.6	39.8		2.5	0.09	25			0.83	7.44	Small				
No 4	7.77	1.79	7.6	39.8		2.5	0.09	30			0.61	7.41	Medium/Small	1.8	1.9	0.97	0.14
No 5	7.77	1.79	7.6	39.8		2.5	0.09	35			0.55	7.37	Medium	1.8	1.5	1.2	0.25
No 6	7.77	1.79	7.6	39.8		2.5	0.09	40			0.28	7.34	Medium	2.2	1.45	0.85	

Experiment # **15** ACH Alone

Date	11/4/2003
Time	6:30
Operator	J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8187 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.77	7.6	39.8		2.5	0.09	10					Pinpoint; no settling, test aborted				
No 2	7.77	1.77	7.6	39.8		2.5	0.09	15					Pinpoint; no settling, test aborted				
No 3	7.77	1.77	7.6	39.8		2.5	0.09	20					Pinpoint; no settling, test aborted				
No 4	7.77	1.77	7.6	39.8		2.5	0.09	25					No visible floc formation; test aborted				
No 5	7.77	1.77	7.6	39.8		2.5	0.09	30					No visible floc formation; test aborted				
No 6	7.77	1.77	7.6	39.8		2.5	0.09	35					No visible floc formation; test aborted				

Experiment # **16** PACl Alone

Date	11/4/2003
Time	7:30
Operator	J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	8158 (mg/L)	Other (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.71	7.6	39.8		2.5	0.09	15			2.3	7.51	No apparent floc formation				
No 2	7.77	1.71	7.6	39.8		2.5	0.09	20			1.9	7.41	Pinpoint				
No 3	7.77	1.71	7.6	39.8		2.5	0.09	25			0.53	7.34	Medium/Small	1.5	0.98	0.67	0.16
No 4	7.77	1.71	7.6	39.8		2.5	0.09	30			0.29	7.31	Medium	0.67	0.54	0.5	0.08
No 5	7.77	1.71	7.6	39.8		2.5	0.09	35			0.31	7.32	Medium	1.5		0.65	
No 6	7.77	1.71	7.6	39.8		2.5	0.09	40			0.25	7.22	Large				

Experiment # **17** Alum + PACl

Date	11/5/2003
Time	8:30
Operator	J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000

Alum at T0, PACl at T30

Task IIi Results

	Speed (rpm)	Time (min)	G x t
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	8158 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.71	7.6	41.5		2.5	0.09	6	15		0.54	6.18	Large; superior settling				
No 2	7.77	1.71	7.6	41.5		2.5	0.09	8	15		0.54	7.09	Large; acceptable settling				
No 3	7.77	1.71	7.6	41.5		2.5	0.09	12	15		0.44	6.98	Large; good settling				
No 4	7.77	1.71	7.6	41.5		2.5	0.09	10	10		0.65	6.88	Large; acceptable settling				0.16
No 5	7.77	1.71	7.6	41.5		2.5	0.09	10	15		0.52	6.86	Large; good settling				
No 6	7.77	1.71	7.6	41.5		2.5	0.09	10	20		0.34	6.89	Large; superior settling				

Experiment # **18** Alum + PACl

Date 11/5/2003
Time 9:30
Operator J/J

Alum at T0, PACl at T30

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water					Chemicals					Settled Water			Settling Velocity			Filtered
	Temperatu (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Alum (mg/L)	8158 (mg/L)	Other (mg/L)	Turbidity (NTU)	pH	Remarks	2-min (NTU)	5-min (NTU)	10-min (NTU)	Turbidity (NTU)
No 1	7.77	1.67	7.6	41.5		2.5	0.09	6	6		1.9	7.2	No visible floc formation				
No 2	7.77	1.67	7.6	41.5		2.5	0.09	10	6		1.6	7.02	Pinpoint/Small	2.2	1.8	1.6	0.2
No 3	7.77	1.67	7.6	41.5		2.5	0.09	14	6		0.96	6.75	Medium/Small				
No 4	7.77	1.67	7.6	41.5		2.5	0.09	10	4		2	7.11	Pinpoint/Small				
No 5	7.77	1.67	7.6	41.5		2.5	0.09	10	8		0.7	7.11	Medium/Small	2	1.2	0.8	0.22
No 6	7.77	1.67	7.6	41.5		2.5	0.09	10	10		0.6	7.12	Medium	1.7	1.1	0.78	

Task IV Results

Experiment # **19** Alum + PACl (pH varied)

Date 11/5/2003
Time 10:30
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Alum at T0, PACl at T30

Jar Results	Raw Water: Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Chemicals: Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Lime (mL)	H2SO4 (mL)	Alum (mg/L)	8158 (mg/L)	Other (mg/L)	Settled Water: Turbidity (NTU)	pH	Remarks	Settling Velocity: 2-min (NTU)	5-min (NTU)	10-min (NTU)	Filtered Turbidity (NTU)
No 1	7.77	1.67	6.9	41.5		2.5	0.09		1	10	7		0.76	6.67	Medium				
No 2	7.77	1.67	7.14	41.5		2.5	0.09		0.75	10	7		0.7	6.92	Medium				
No 3	7.77	1.67	7.38	41.5		2.5	0.09		0.25	10	7		0.9	7	Medium				
No 4	7.77	1.67	7.62	41.5		2.5	0.09			10	7		0.72	7.06	Medium				
No 5	7.77	1.67	7.86	41.5		2.5	0.09	1.25		10	7		0.89	7.08	Medium				
No 6	7.77	1.67	8.1	41.5		2.5	0.09	2		10	7		0.9	7.1	Medium/Small				

Experiment # **20** Alum Alone

Date 11/5/2003
Time 11:30
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water: Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Chemicals: Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Lime (mL)	H2SO4 (mL)	Alum (mg/L)	Other (mg/L)	Other (mg/L)	Settled Water: Turbidity (NTU)	pH	Remarks	Settling Velocity: 2-min (NTU)	5-min (NTU)	10-min (NTU)	Filtered Turbidity (NTU)
No 1	7.77	1.67	6.9	41.5		2.5	0.09		1	18			2	6.8	Small; unsettleable				
No 2	7.77	1.67	7.14	41.5		2.5	0.09		0.75	18			2.5	6.9	Small; unsettleable				
No 3	7.77	1.67	7.38	41.5		2.5	0.09		0.25	18			2.1	6.9	Small; unsettleable				
No 4	7.77	1.67	7.62	41.5		2.5	0.09			18			2	6.9	Small; unsettleable				
No 5	7.77	1.67	7.86	41.5		2.5	0.09	1.25		18			2	6.95	Small; unsettleable				
No 6	7.77	1.67	8.1	41.5		2.5	0.09	2		18			2.2	7.04	Small; unsettleable				

Experiment # **21** PACl Alone

Date 11/5/2003
Time 12:45
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Jar Results	Raw Water: Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Chemicals: Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Lime (mL)	H2SO4 (mL)	8158 (mg/L)	Other (mg/L)	Other (mg/L)	Settled Water: Turbidity (NTU)	pH	Remarks	Settling Velocity: 2-min (NTU)	5-min (NTU)	10-min (NTU)	Filtered Turbidity (NTU)
No 1	7.77	1.60	6.9	41.5		2.5	0.09		1	25			0.5	7	Medium; very settleable				
No 2	7.77	1.60	7.14	41.5		2.5	0.09		0.75	25			0.5	7.13	Medium; very settleable				
No 3	7.77	1.60	7.38	41.5		2.5	0.09		0.25	25			0.5	7.28	Medium; very settleable				
No 4	7.77	1.60	7.62	41.5		2.5	0.09			25			0.5	7.3	Medium; very settleable				
No 5	7.77	1.60	8.1	41.5		2.5	0.09	2		25			0.5	7.36	Medium; very settleable				
No 6	7.77	1.60	8.3	41.5		2.5	0.09	3		25			0.5	7.43	Medium; very settleable				

Experiment # **22** Ferric + 2490

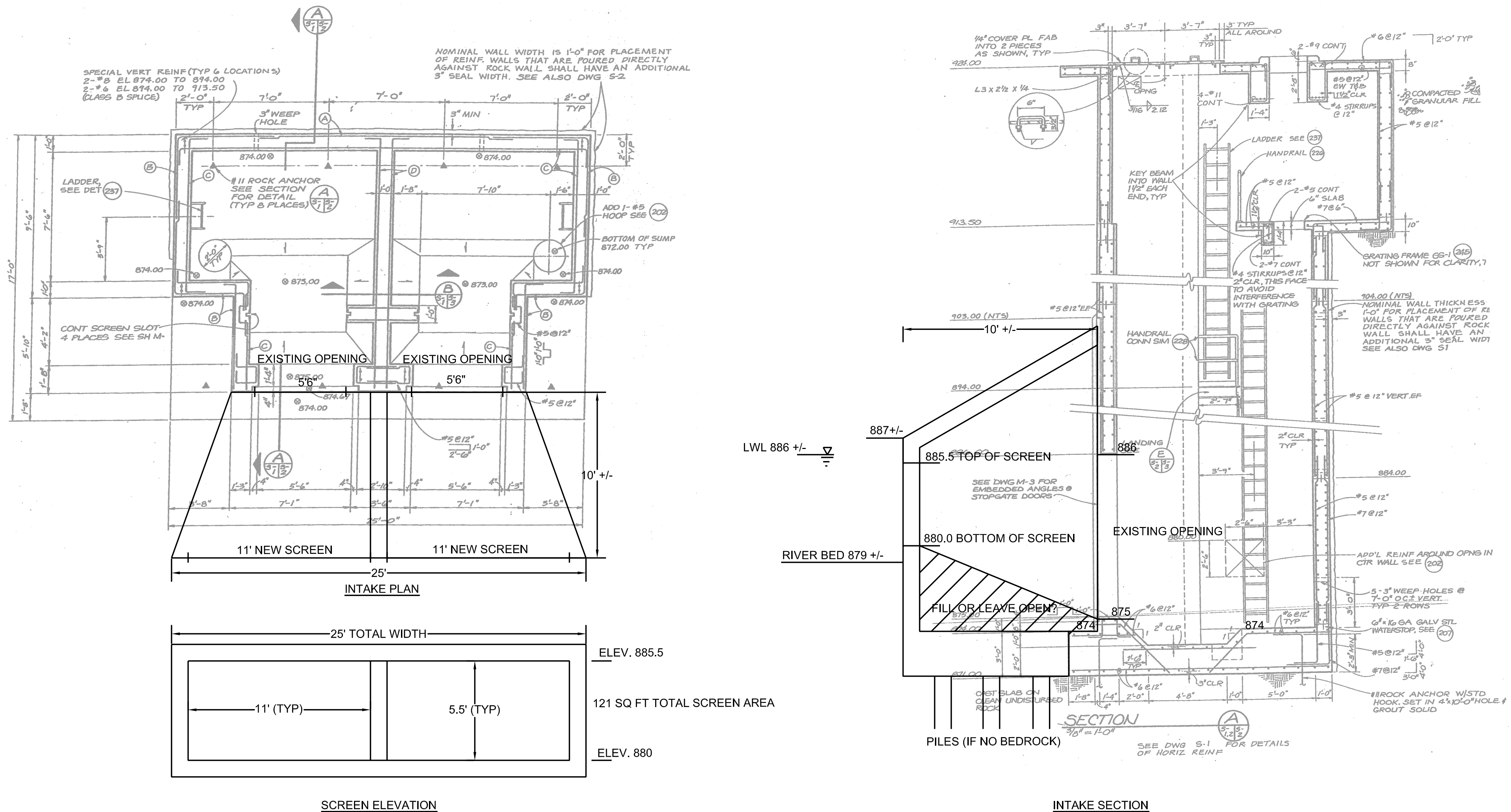
Date 11/5/2003
Time 1:20
Operator J/J

Mixing Conditions:	Speed (rpm)	Time (min)	G x t
Rapid Mix	200	1	6000
Stage 1	70	5	18000
Stage 2	40	5	900
Stage 3	25	5	4500
Sedimentation	0	20	0
Total			29400

Ferric at T0, Poly at T30

Jar Results	Raw Water: Temperature (deg C)	Turbidty (NTU)	pH	Alkalinity (mg/L)	TOC (mg/L)	Chemicals: Pre-Cl2 (mg/L)	KMnO4 (mg/L)	Lime (mL)	H2SO4 (mL)	Ferric (mg/L)	2490 (mg/L)	Other (mg/L)	Settled Water: Turbidity (NTU)	pH	Remarks	Settling Velocity: 2-min (NTU)	5-min (NTU)	10-min (NTU)	Filtered Turbidity (NTU)
No 1	7.6	1.58	6.9	41.5		2.5	0.09		1	10	0.5				Medium/Small				
No 2	7.6	1.58	7.14	41.5		2.5	0.09		0.75	10	0.5				Medium/Small				
No 3	7.6	1.58	7.38	41.5		2.5	0.09		0.25	10	0.5				Medium				
No 4	7.6	1.58	7.62	41.5		2.5	0.09			10	0.5				Medium				
No 5	7.6	1.58	8.1	41.5		2.5	0.09	2		10	0.5				Pinpoint/almost no visible floc (missed chemical?)				
No 6	7.6	1.58	8.3	41.5		2.5	0.09	3		10	0.5				Medium				

NOTE: All turbidities, less jar 5, "appear" the same, settling not impacted by pH… pH has little impact on ferric settling.



Not to Scale

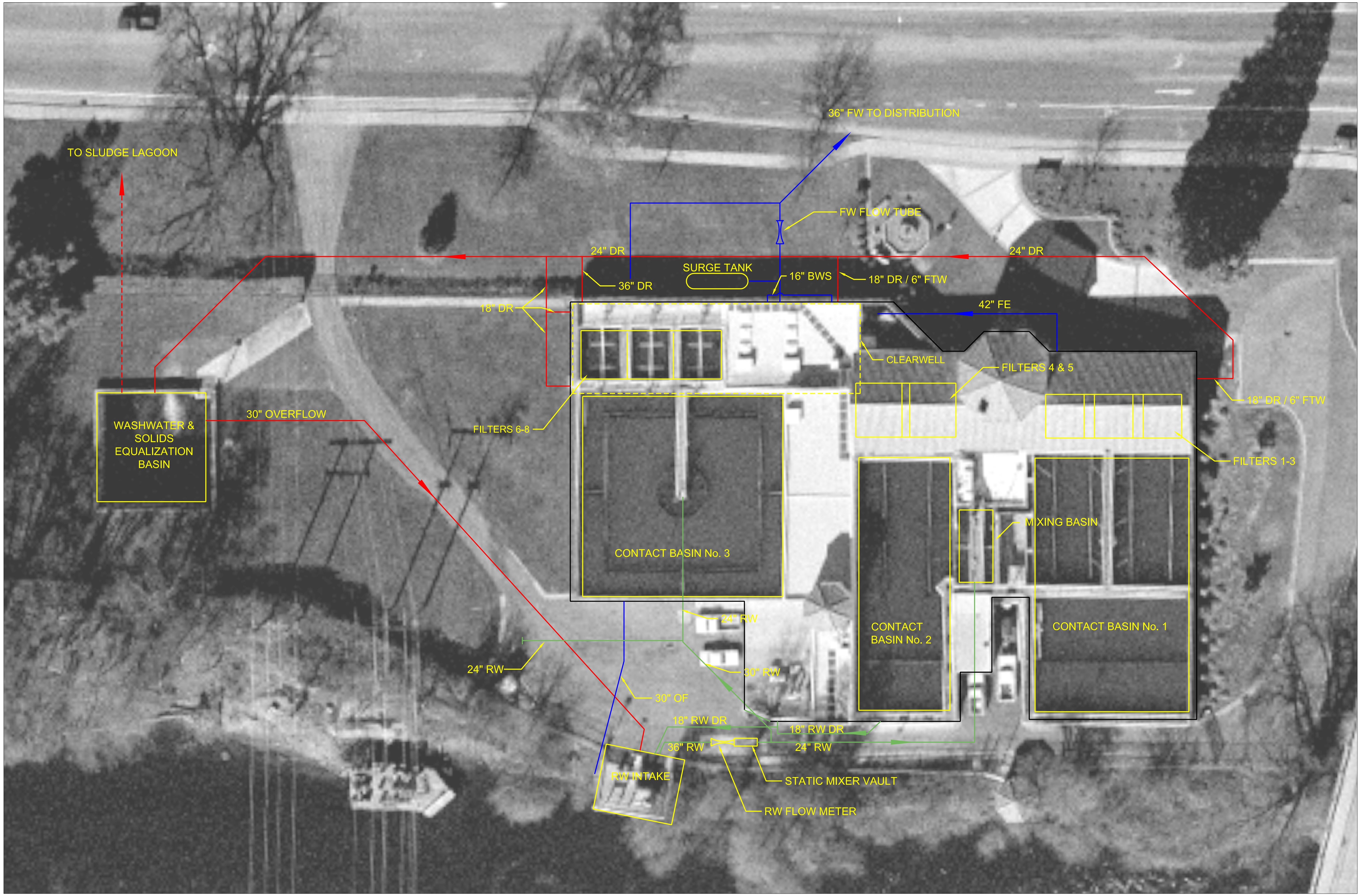
MWH

City of Grants Pass
Water Treatment Plant Facility Plan
Suggested Intake Improvements

SCALE: 1" = 125' IF
LINE MEASURES 1"

MWH

Figure 1-1
City of Grants Pass
Water Treatment Plant Facility Plan
System Overview



LEGEND

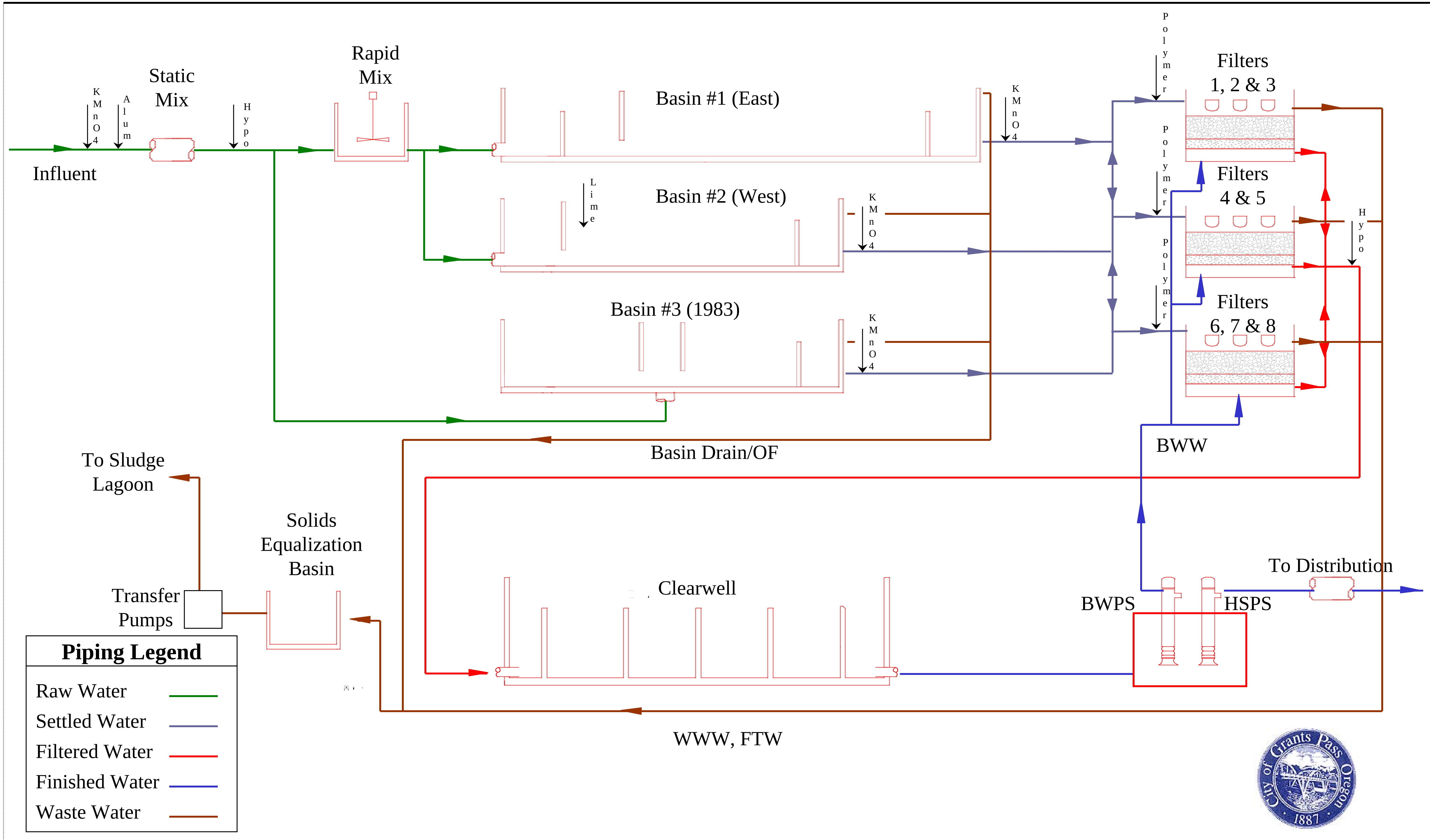
- BUILDING OUTLINE
- MAJOR PROCESS COMPONENTS
- FW PIPING
- RW PIPING
- WASTE/OF PIPING

SCALE: 1" = 40' IF LINE MEASURES 1"

City of Grants Pass Oregon 1887

MWH

Figure 1-2
City of Grants Pass
Water Treatment Plant Facility Plan
Plan-view Layout



MWH

Figure 1-3
City of Grants Pass
Water Treatment Plant Facility Plan
Process Flow Schematic

Figure 2-1
Grants Pass WTP:
1999-2003-- Daily Raw Water and Finished Water Flows

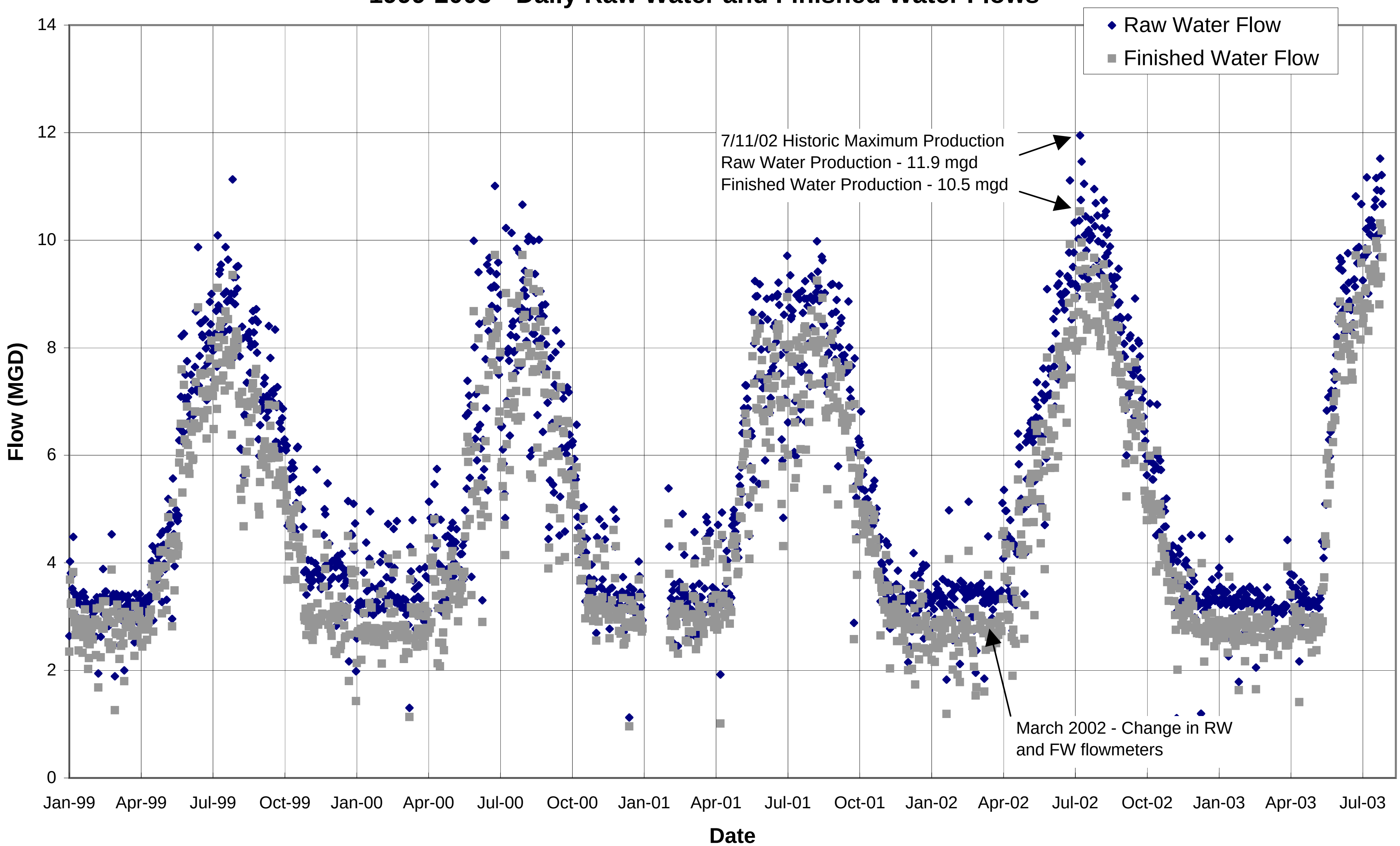


Figure 2-10
Grants Pass WTP:
April 2002 - June 2003-- Average Daily Contact Basin Effluent Turbidity
9am-3pm Continuous Monitoring

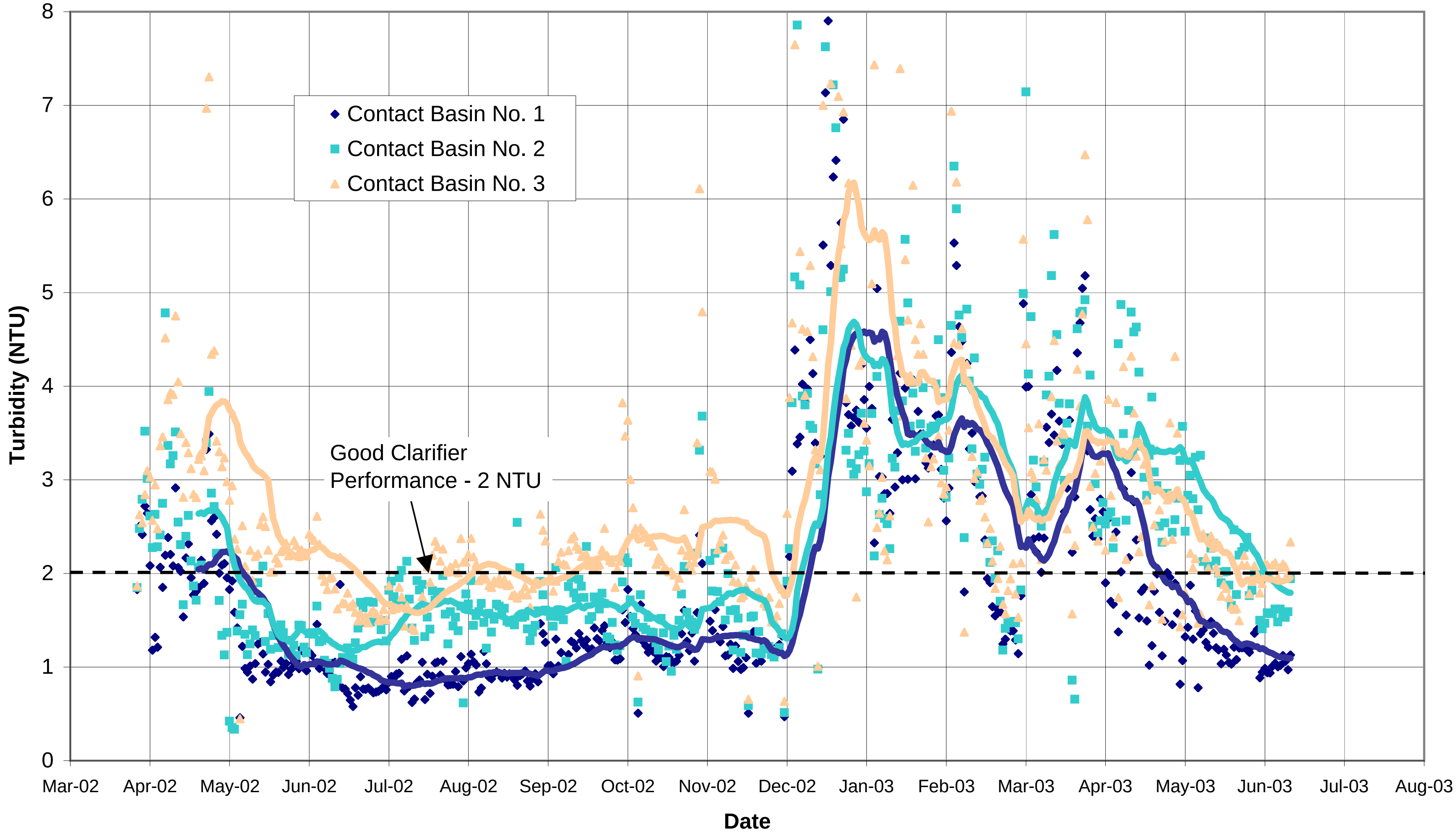
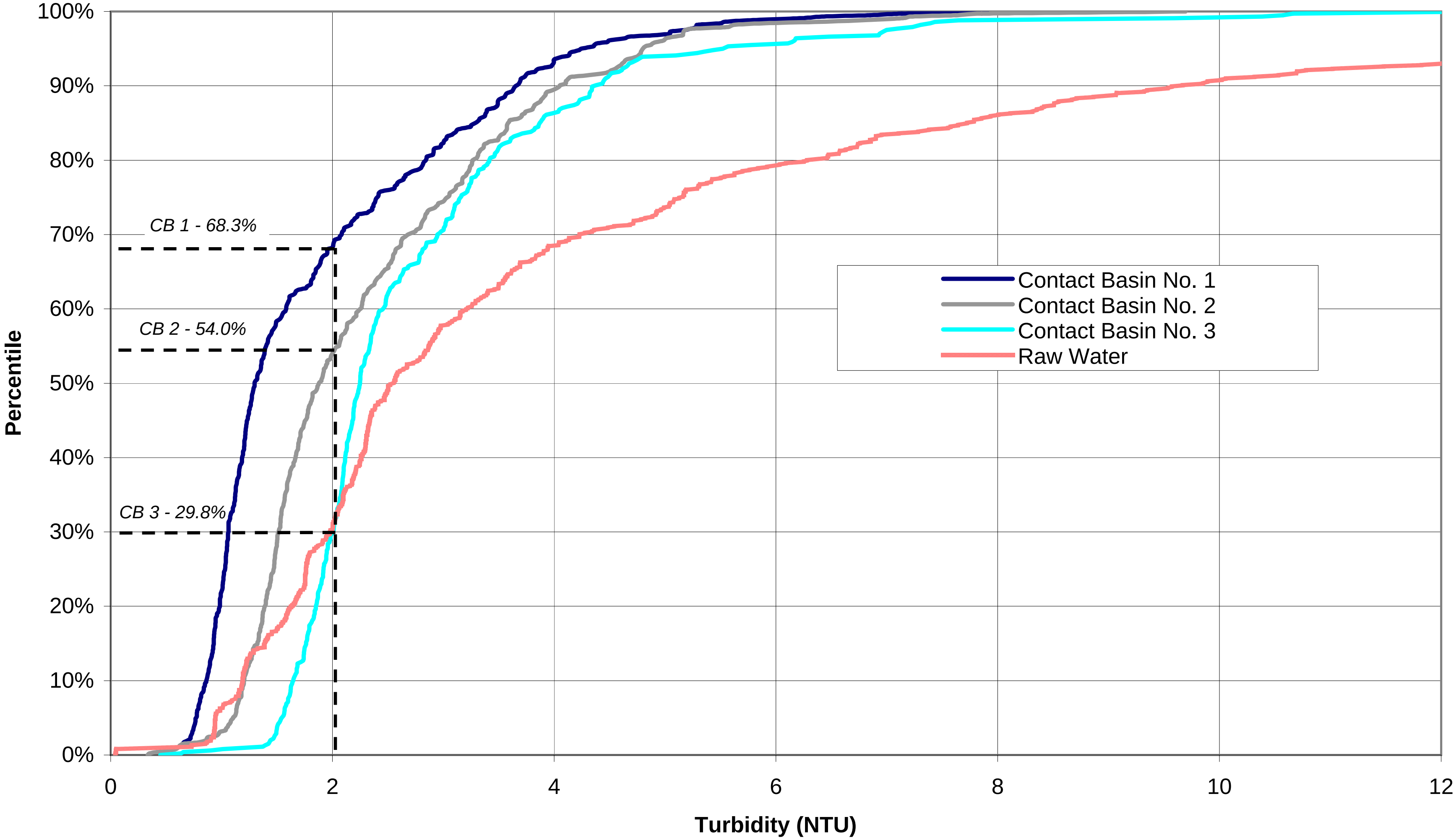
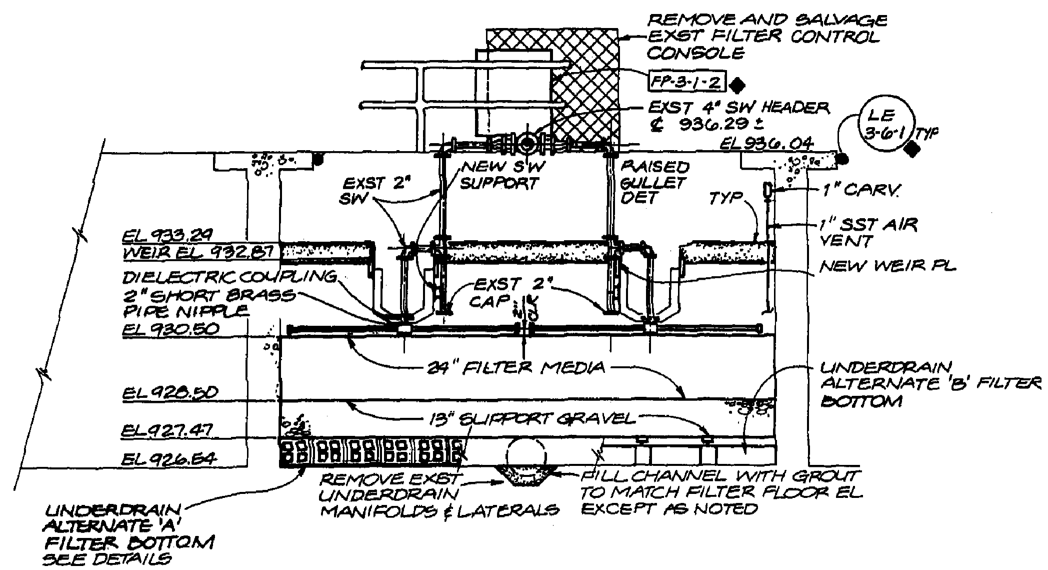
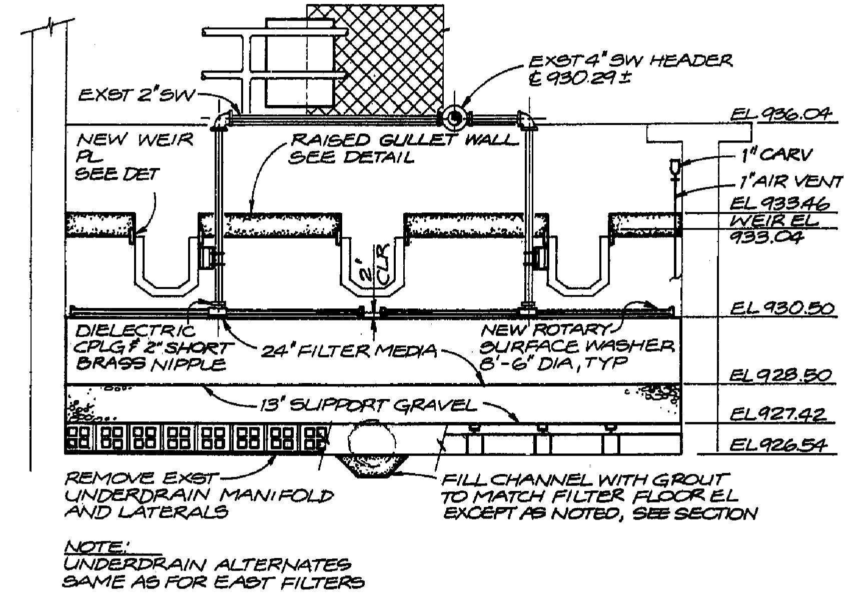


Figure 2-11
Grants Pass WTP:
April 2002 - June 2003-- Contact Basin Effluent Turbidity Probability Distibutions
9am-3pm Continuous Monitoring

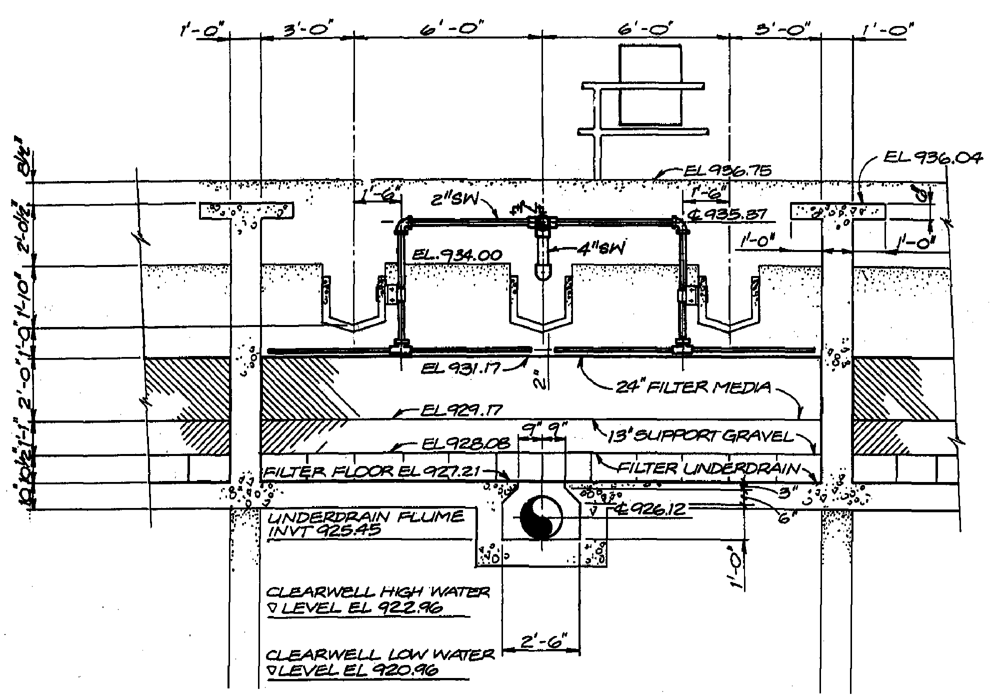




EAST FILTERS A
N.T.S.



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

f:\GrantsP536\DSN\WFP\filter x-section.dgn

MWH
MONTGOMERY WATSON HARZA

Figure 2-12
City Of Grants Pass
WTP Facility Plan
Existing Filter Cross-Sections

Figure 2-13
Grants Pass WTP:
1999-2003-- Daily Maximum Finished Water Turbidity

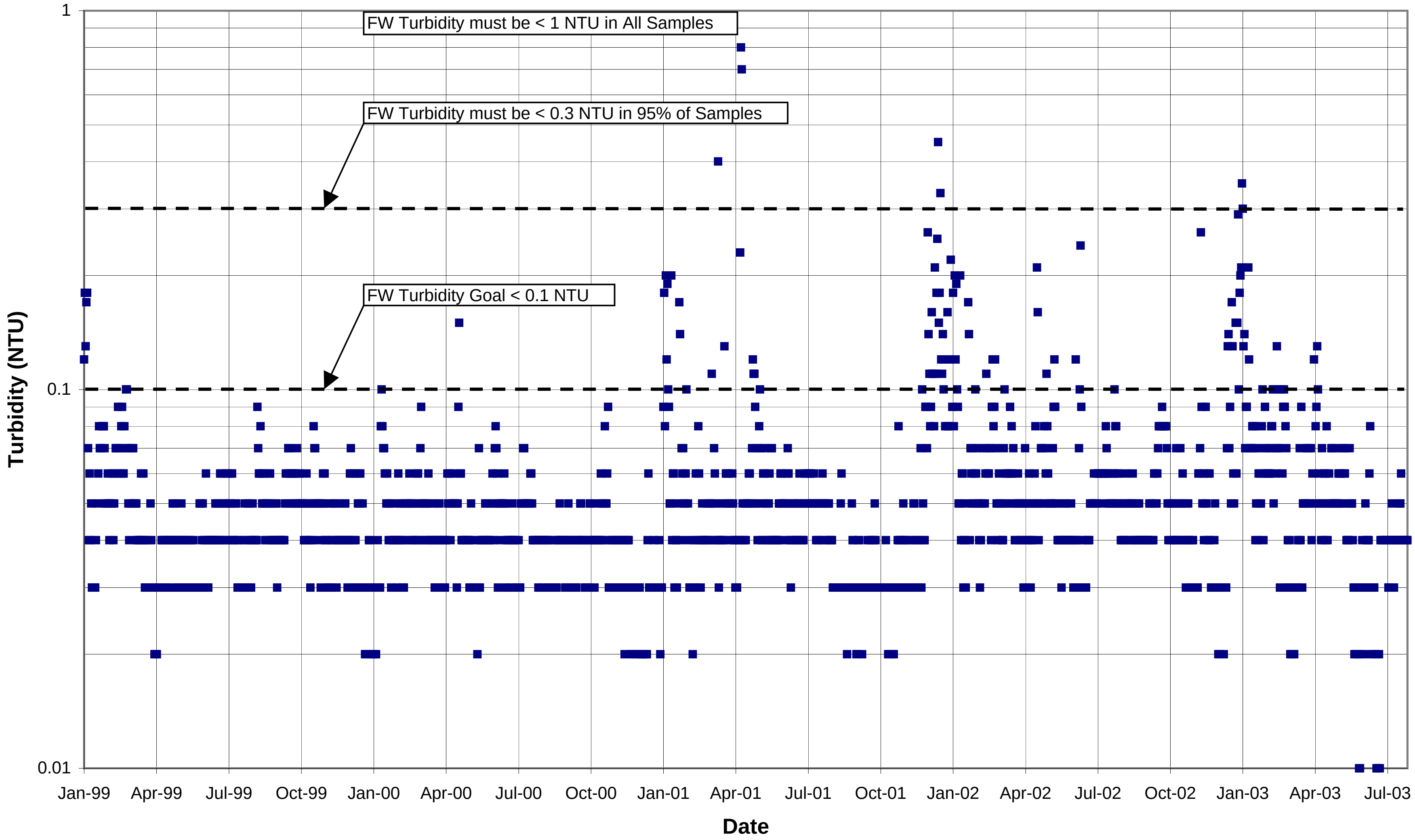


Figure 2-14
Grants Pass WTP:
1999-2003-- Average Daily Plant Finished Water Turbidity Probability Distribution

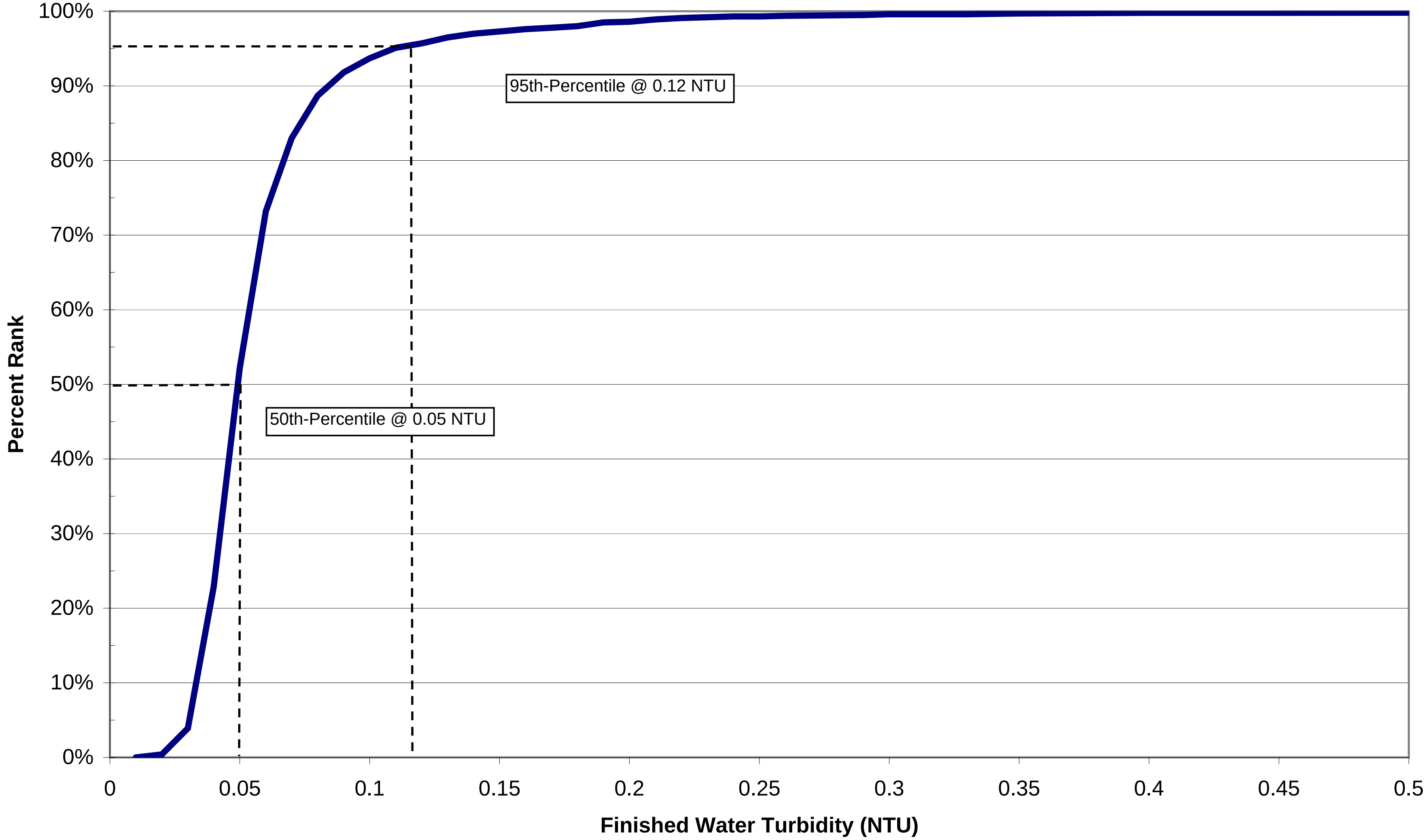
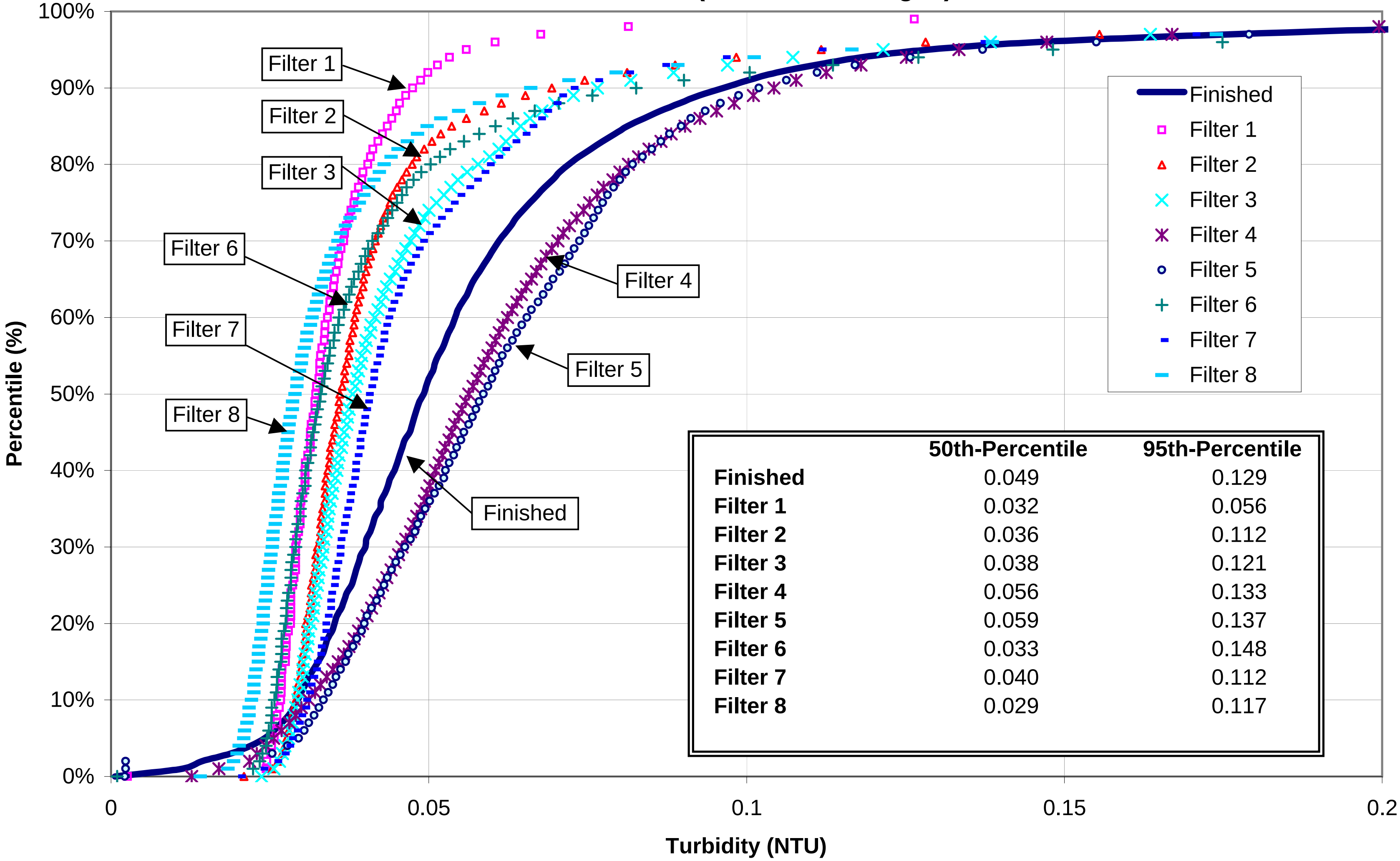


Figure 2-15
Grants Pass WTP:
April 2002 - June 2003 -- Individual Filter Effluent and Plant Effluent Turbidity Probability Distributions (5-minute Averages)



	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

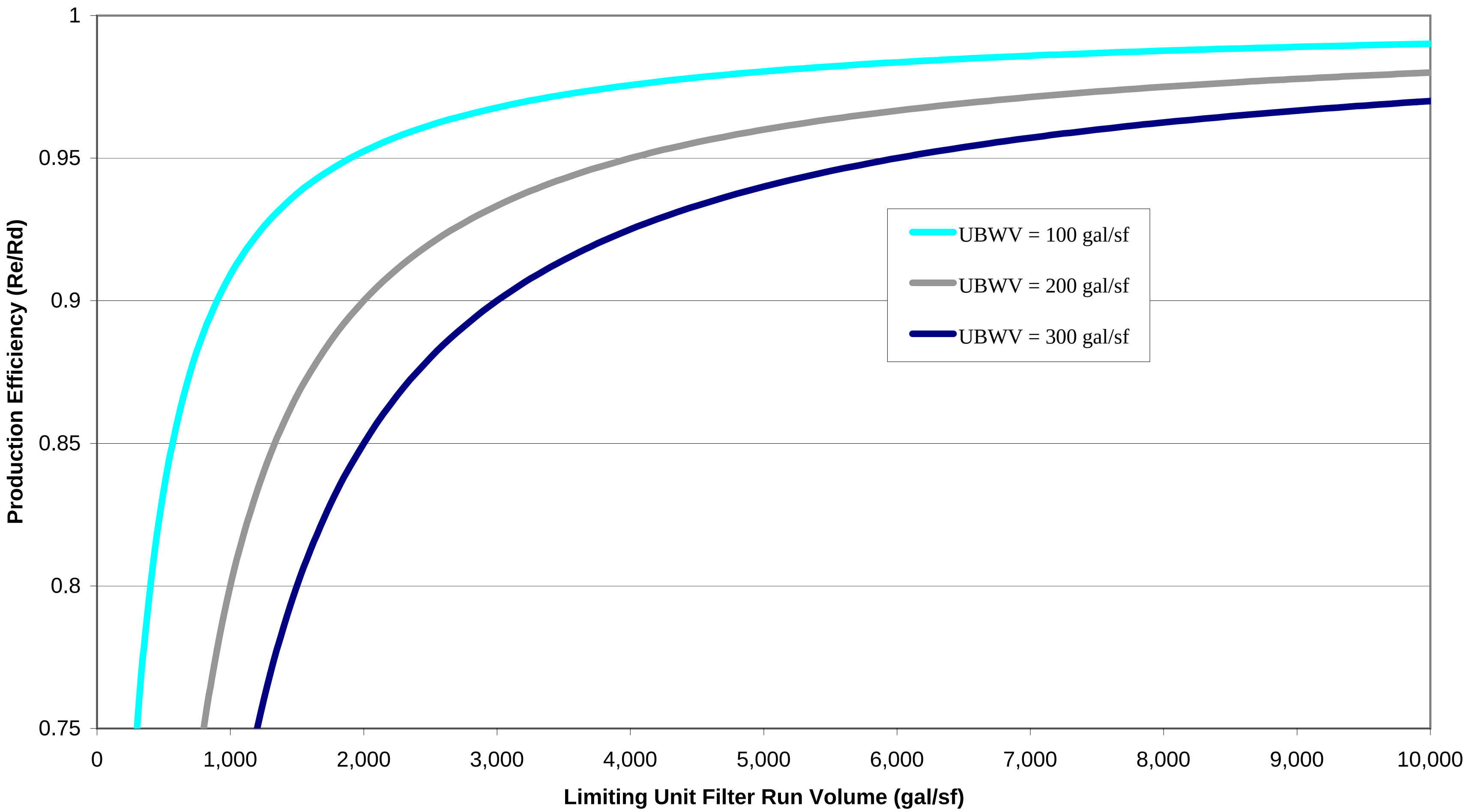


Figure 2-17
Grants Pass WTP:
1999-2003-- Plant Weekly Average Filter Production Efficiency and Weekly Backwash Volume

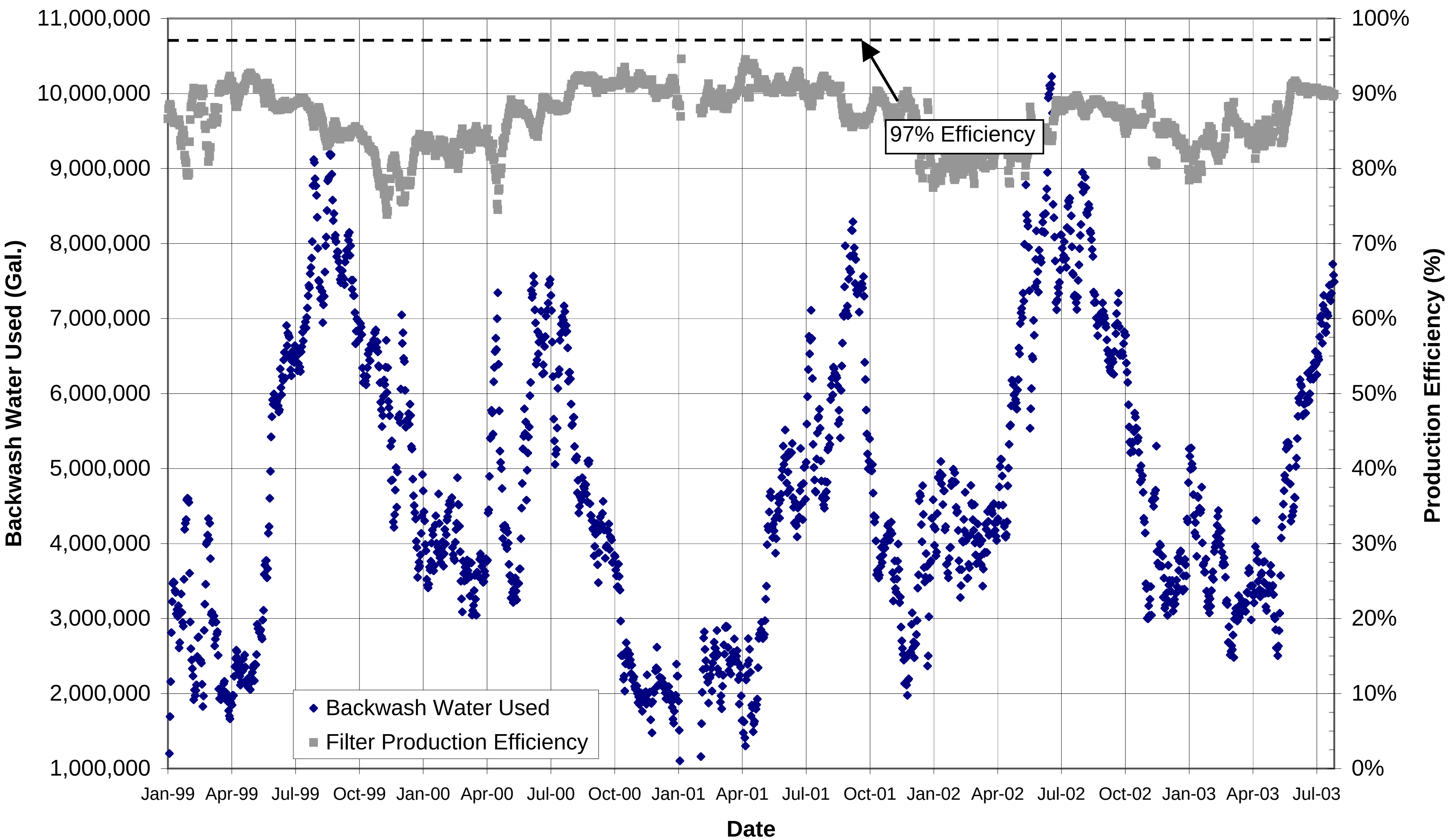
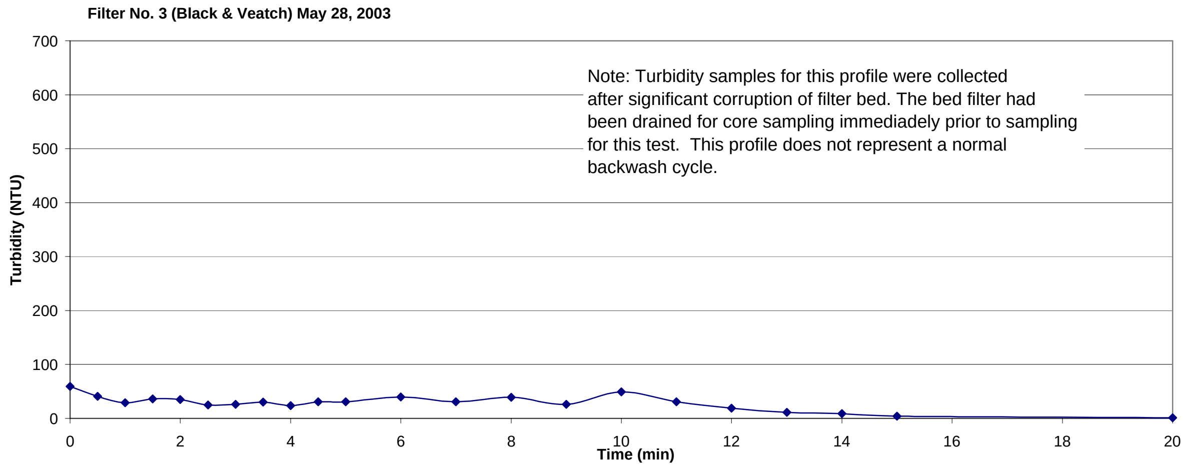
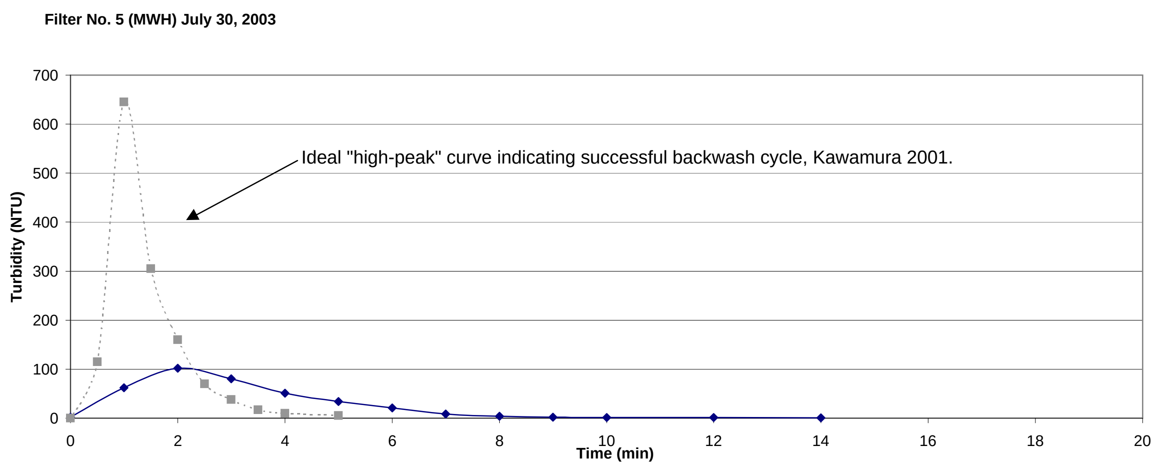


Figure 2-18
Grants Pass WTP: Filter Performance Evaluation
Backwash Turbidity Profiles





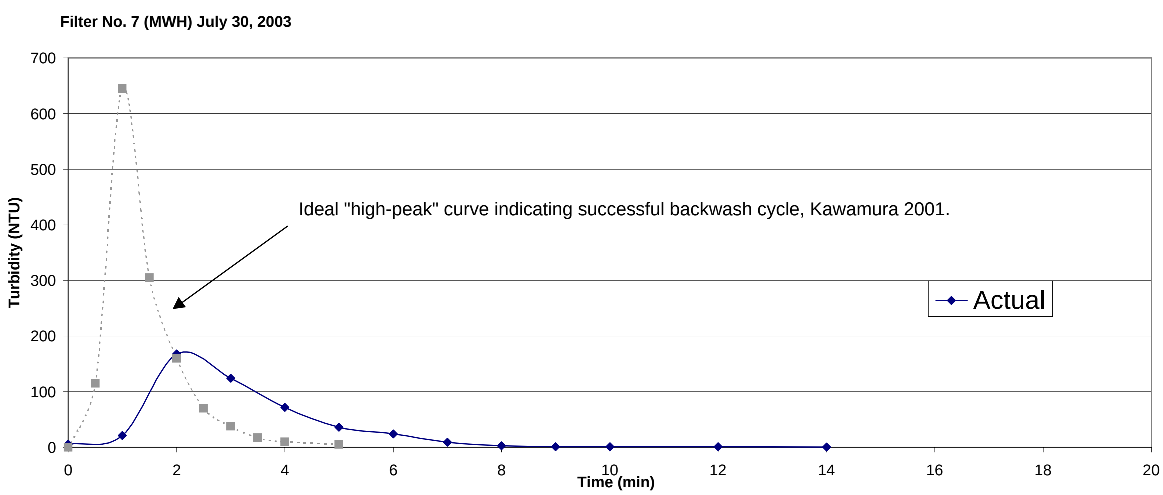
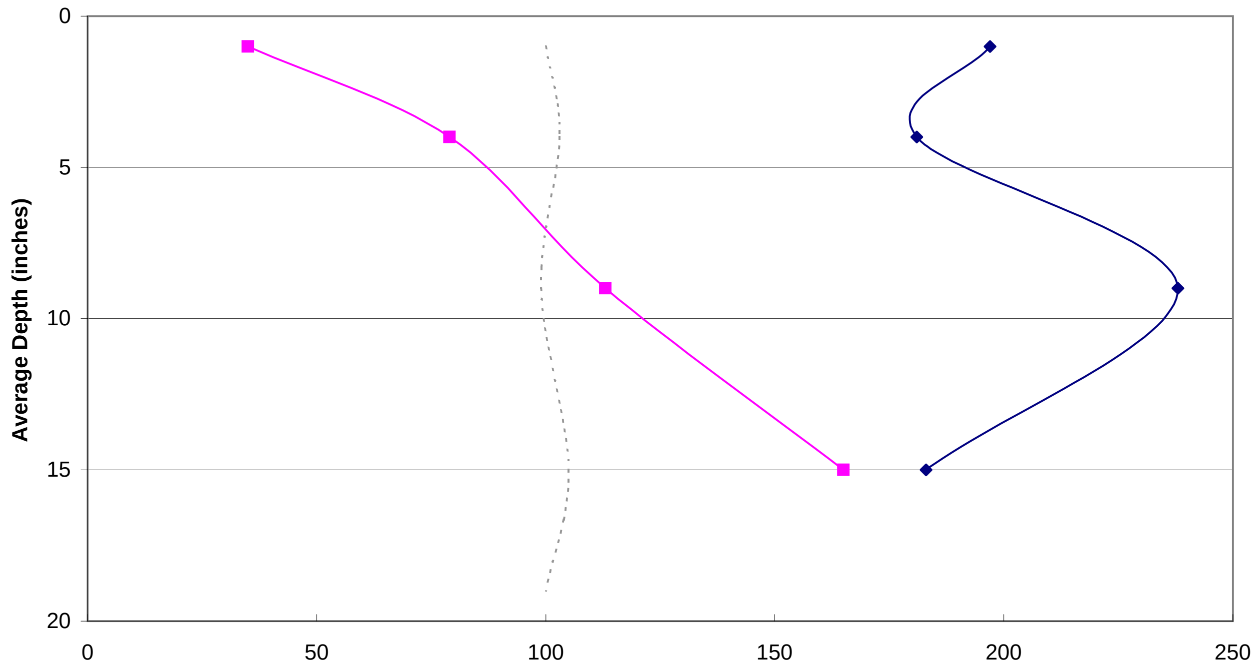


Figure 2-19:
Grants Pass WTP: Filter Performance Evaluation
Floc Retention Profiles

Filter No. 1 (MWH) July 29, 2003



Filter No. 3 (Black & Veatch) May 28, 2003

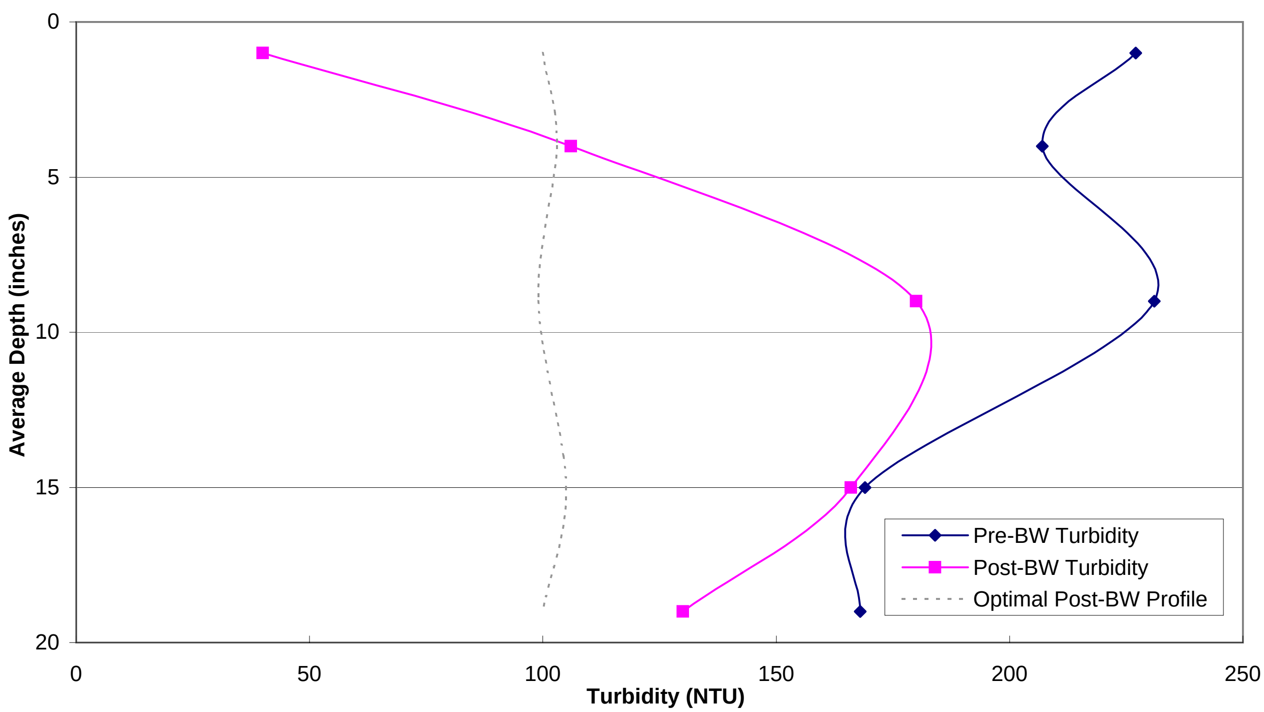
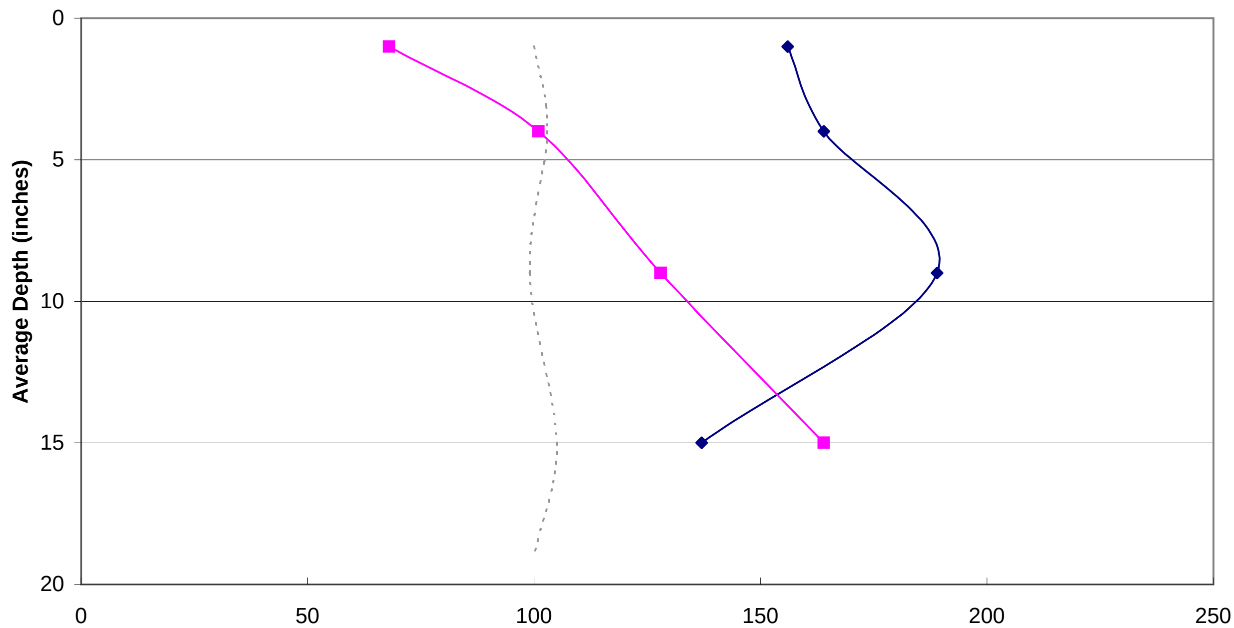


Figure 2-19 (Cont.)
Grants Pass WTP: Filter Performance Evaluation
Floc Retention Profiles

Filter No. 5 (MWH) July 29, 2003



Filter No. 7 (MWH) July 29, 2003

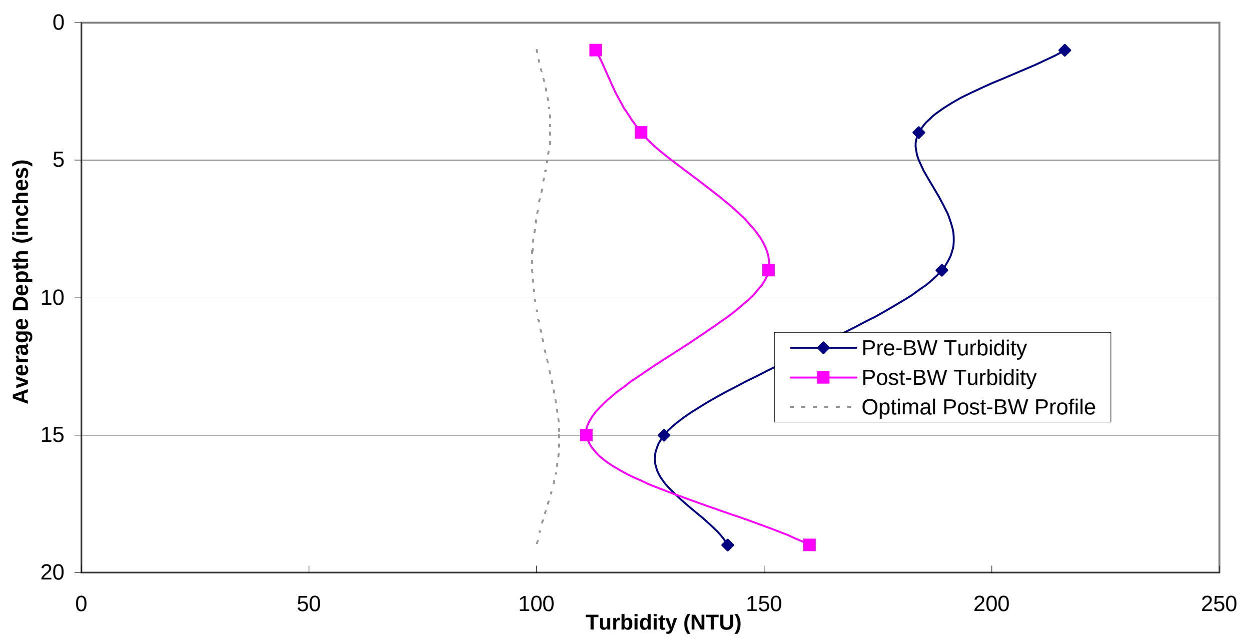


Figure 2-2
Grants Pass WTP:
1999-2003-- Average Daily Plant Raw Water Turbidity and Precipitation

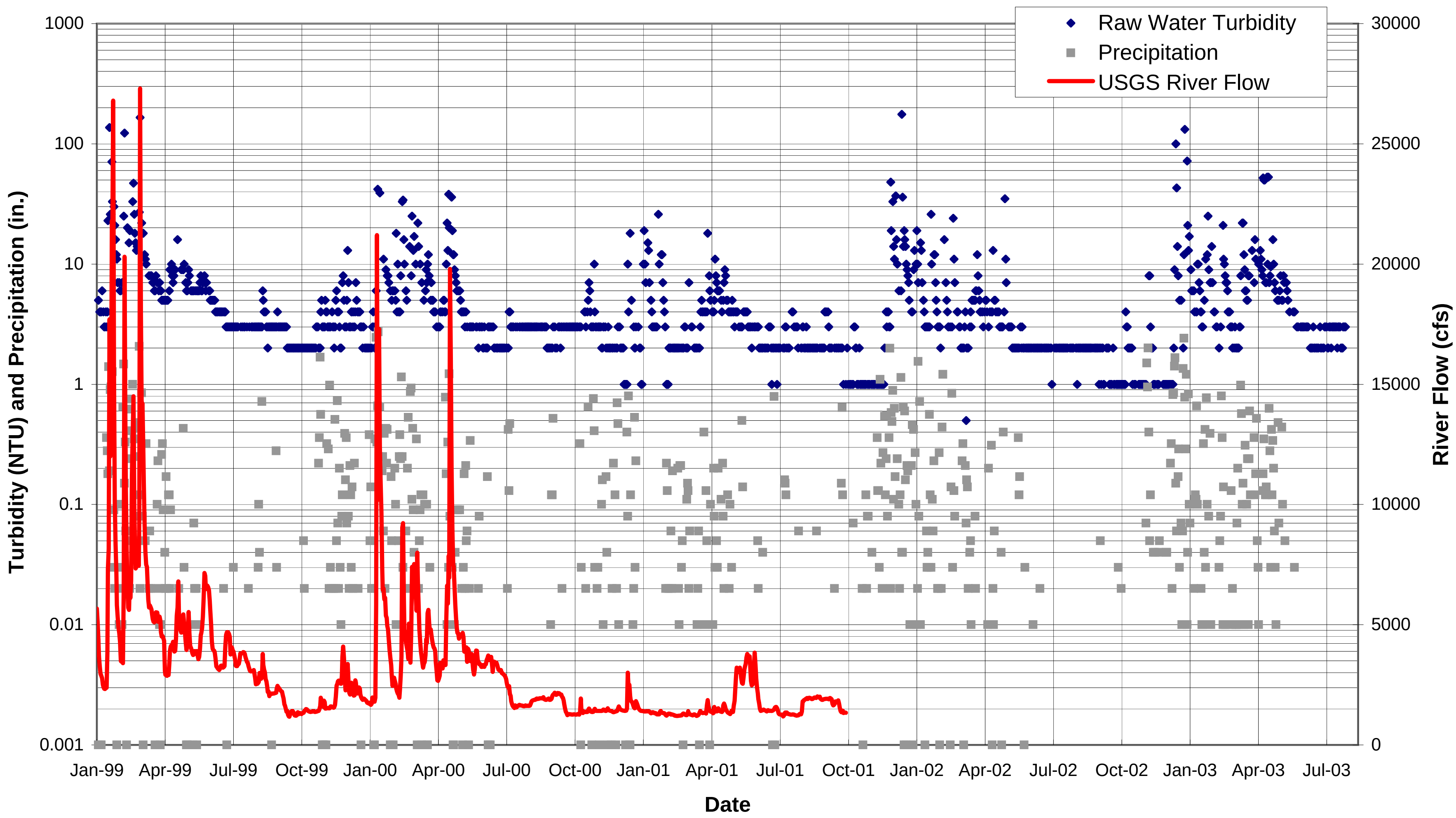


Figure 2-3
Grants Pass WTP:
1999-2003-- Average Daily Raw Water Temperature

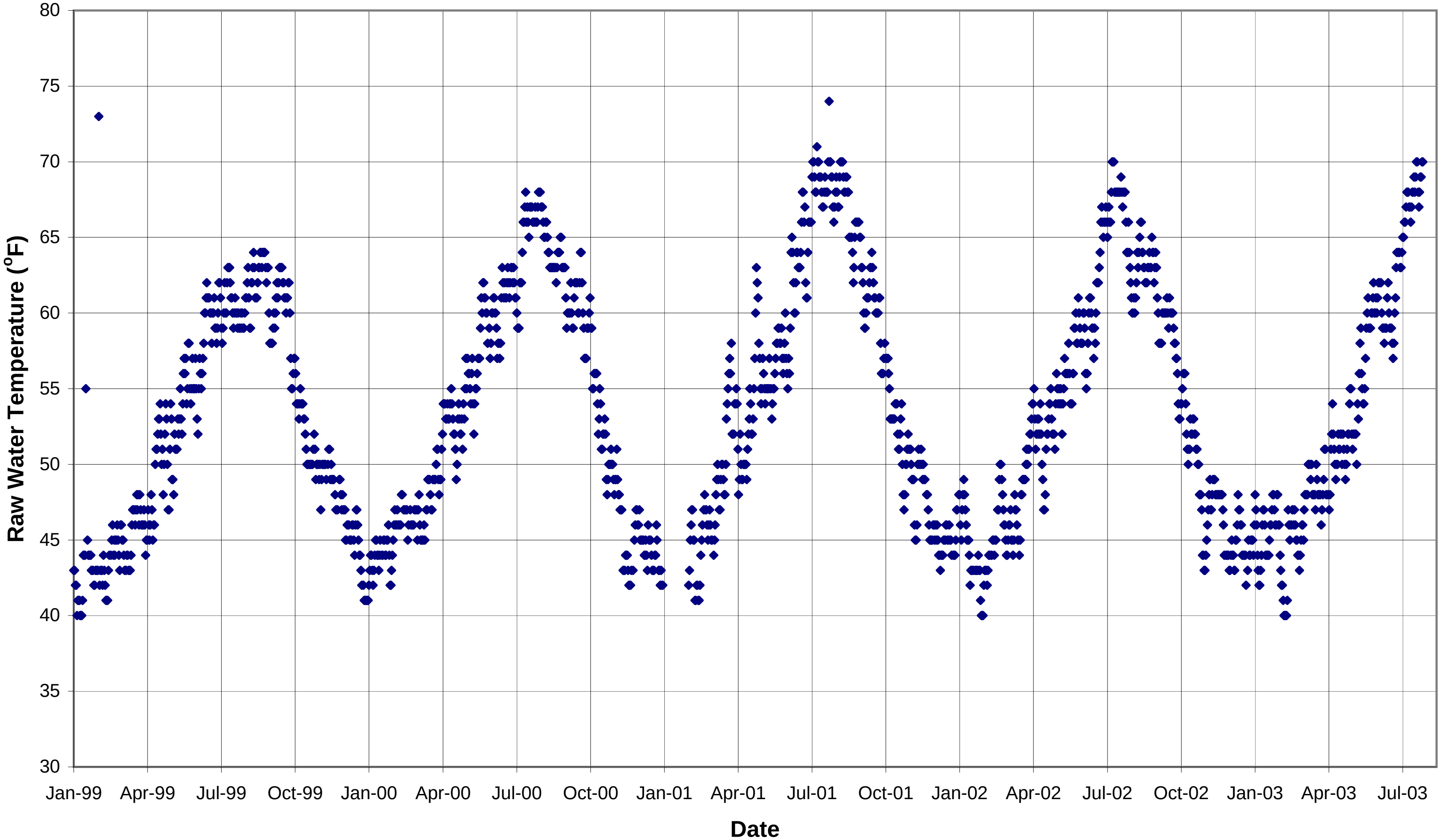


Figure 2-4
Grants Pass WTP:
1999-2003-- Average Daily Plant pH: Raw and Finished Water

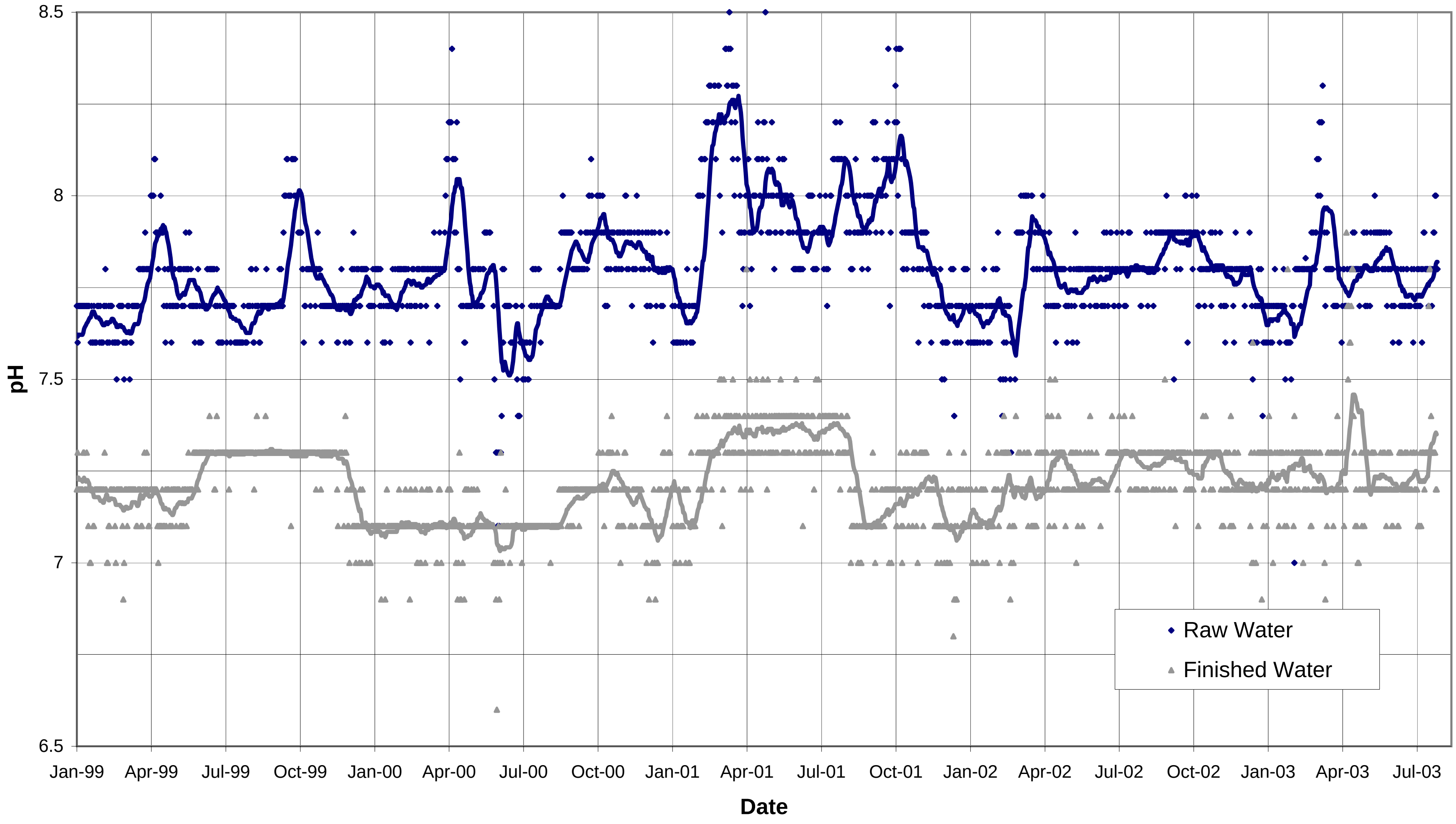


Figure 2-5
Grants Pass WTP:
2001-2002 -- Plant Raw Water and Finished Water TOC and Removal Efficiency

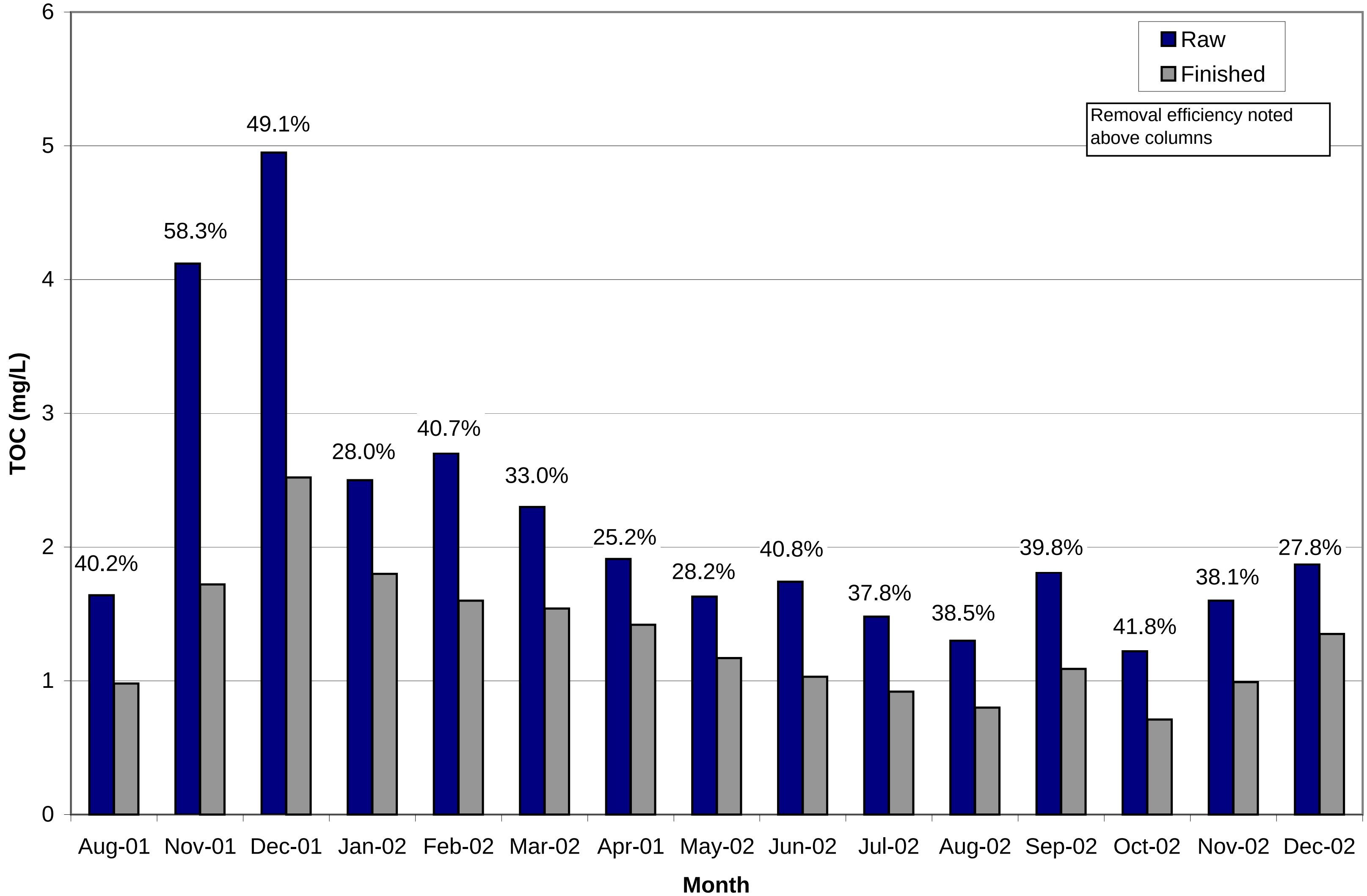


Figure 2-6
Rogue River Raw Water:
Geosmin Concentrations between Lost Creek Dam and City of Rogue River
Peak Taste and Odor Season Samples

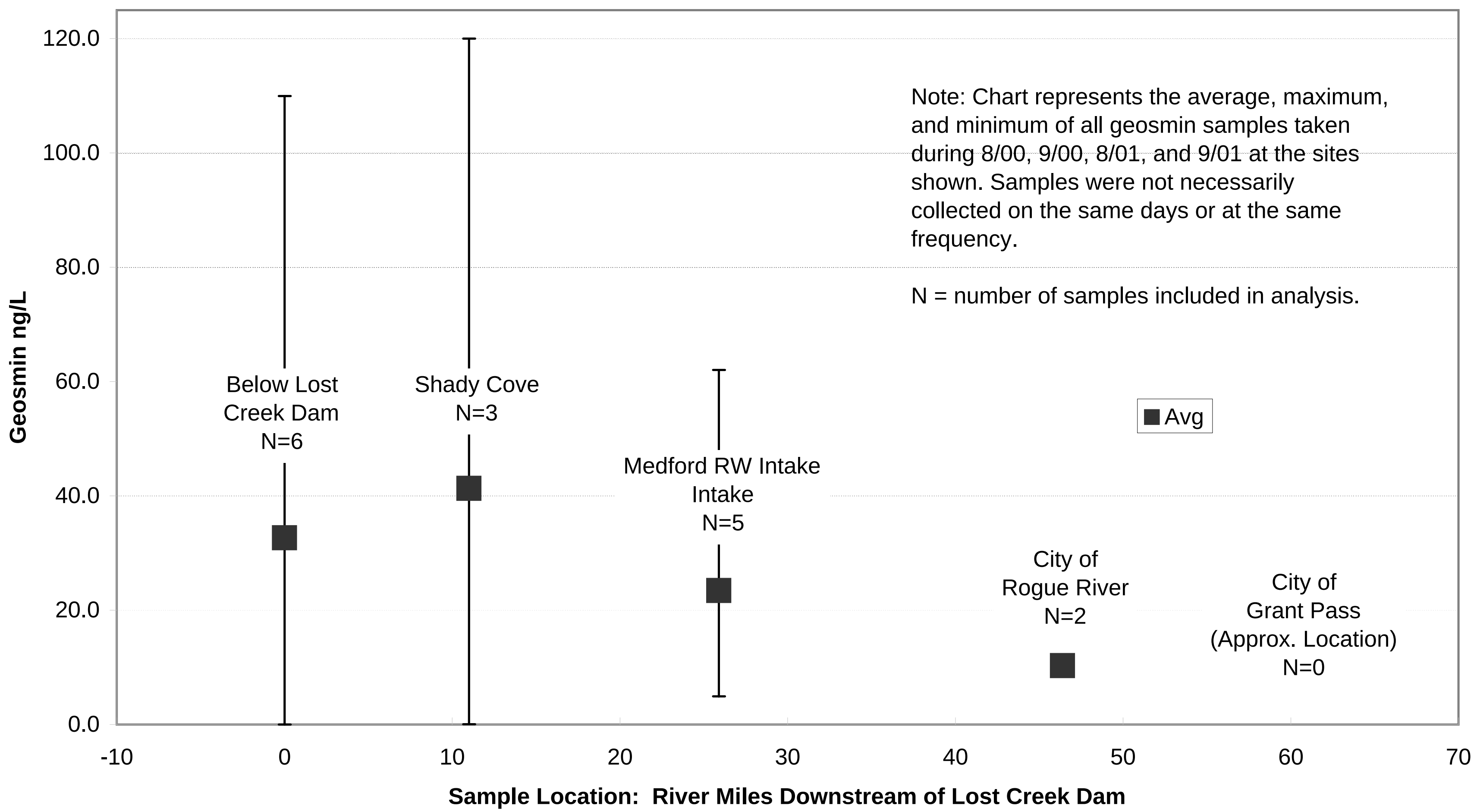


Figure 2-7
Grants Pass WTP:
1999-2003-- Plant Average Daily Alum and Filter Aid Polymer Dose

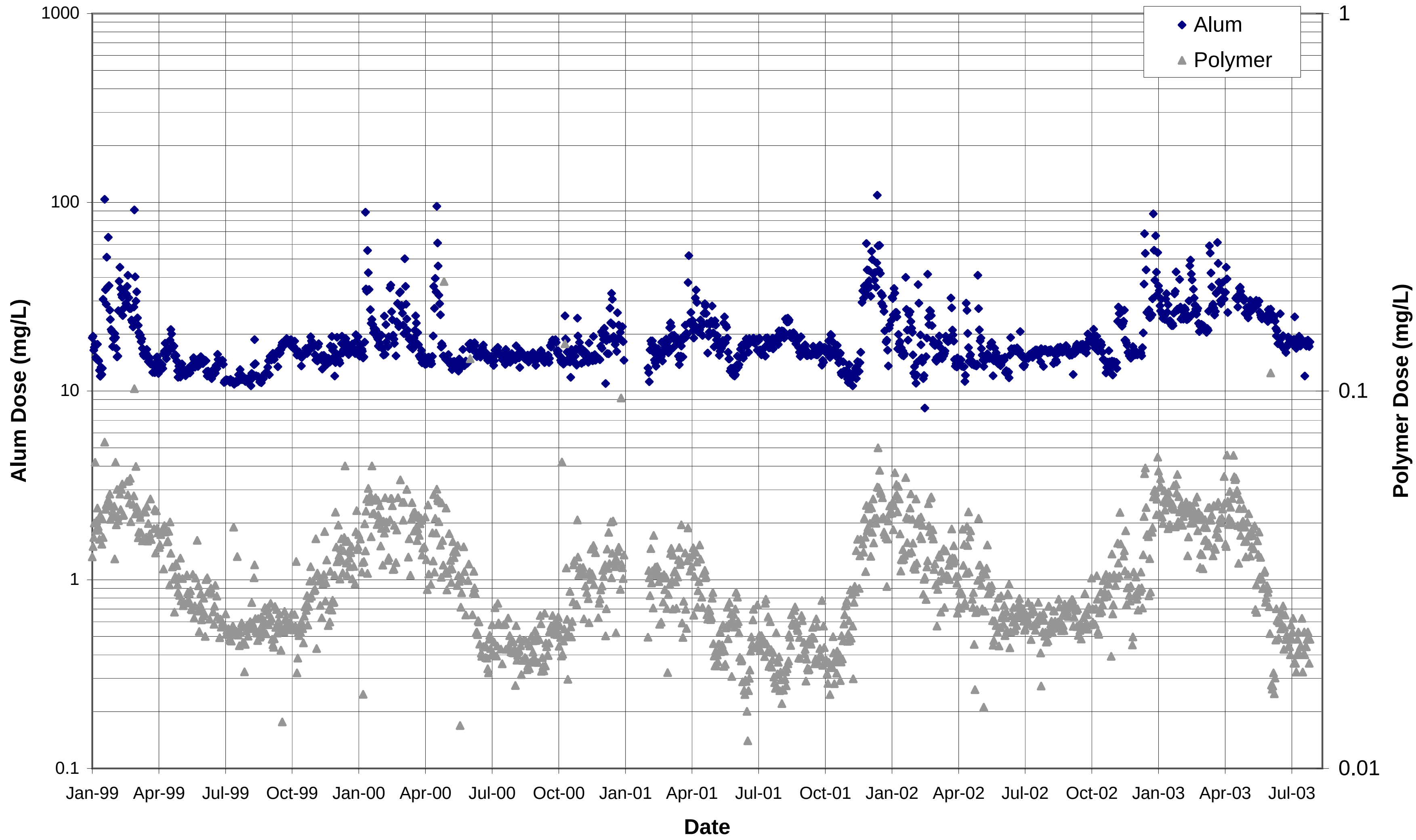


Figure 2-8
Grants Pass WTP:
1999-2003-- Plant Average Daily Lime and Potassium Permanganate Dose

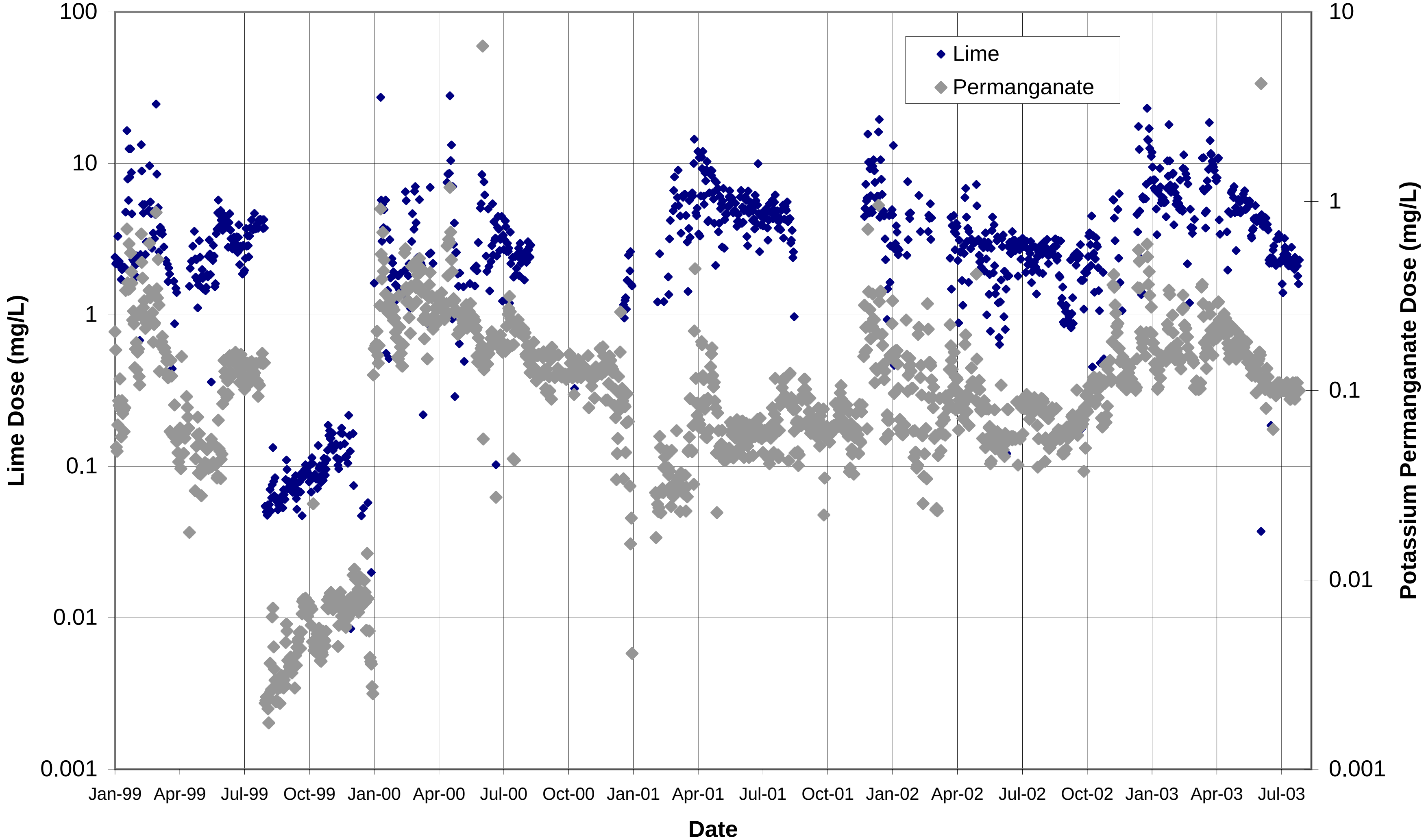


Figure 2-9
Grants Pass WTP:
1999-2003-- Plant Average Daily Chlorine Residuals

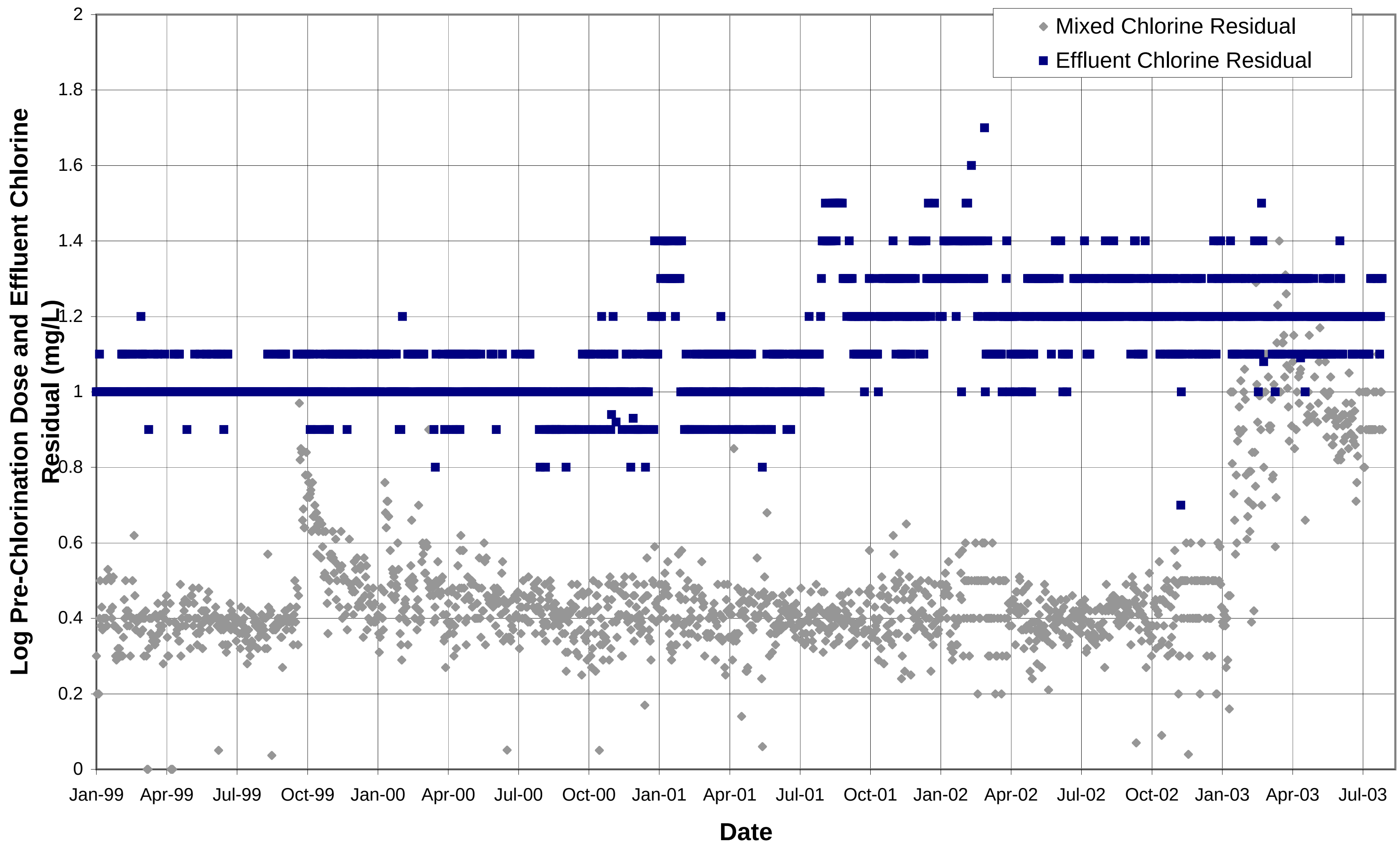


Figure 3-1
Grants Pass WTP:
1999-2003 -- Historical Log Inactivation of *Giardia*

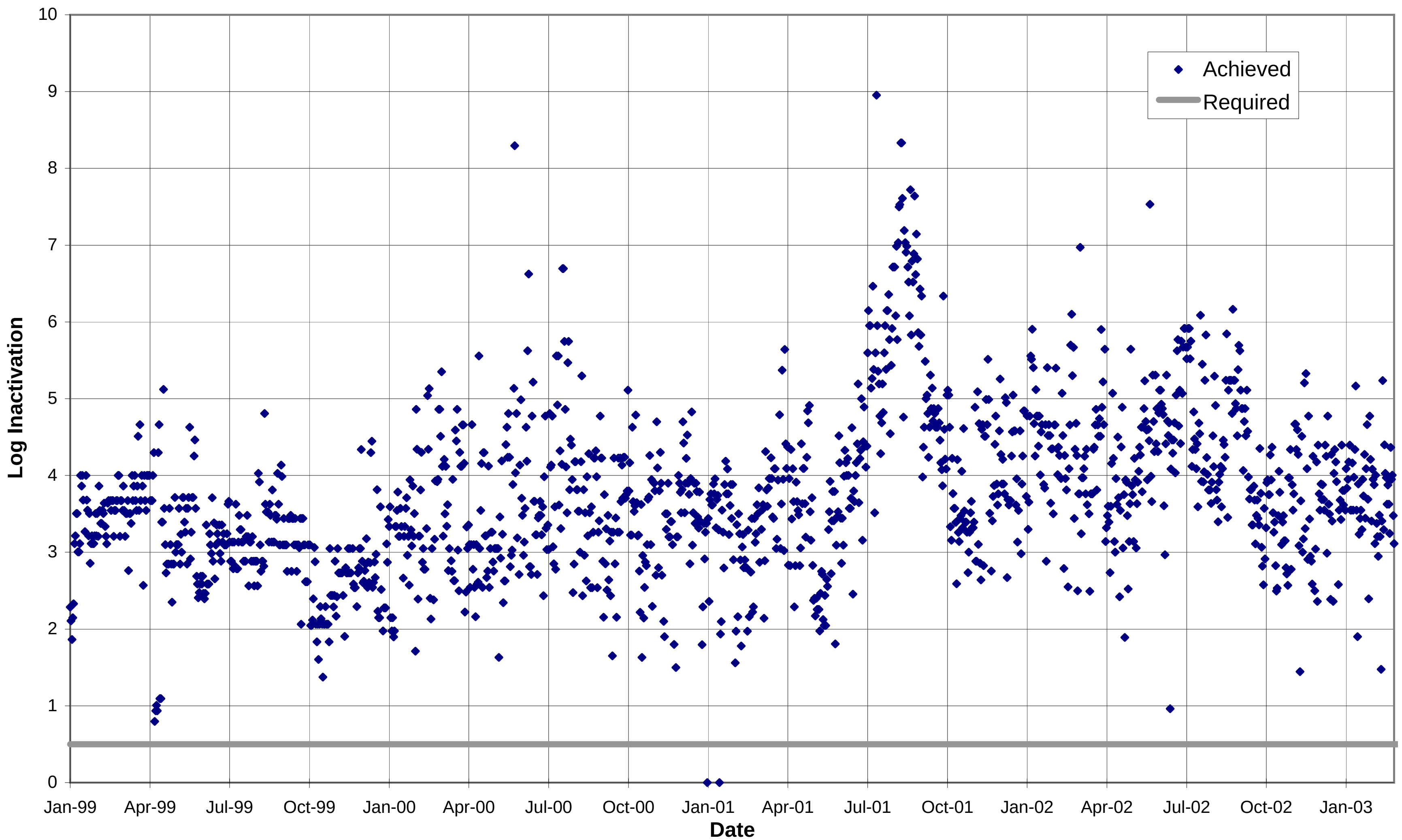


Figure 3-2
Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia
Worst-Case Winter Condition (Temp = 5^{o}C)

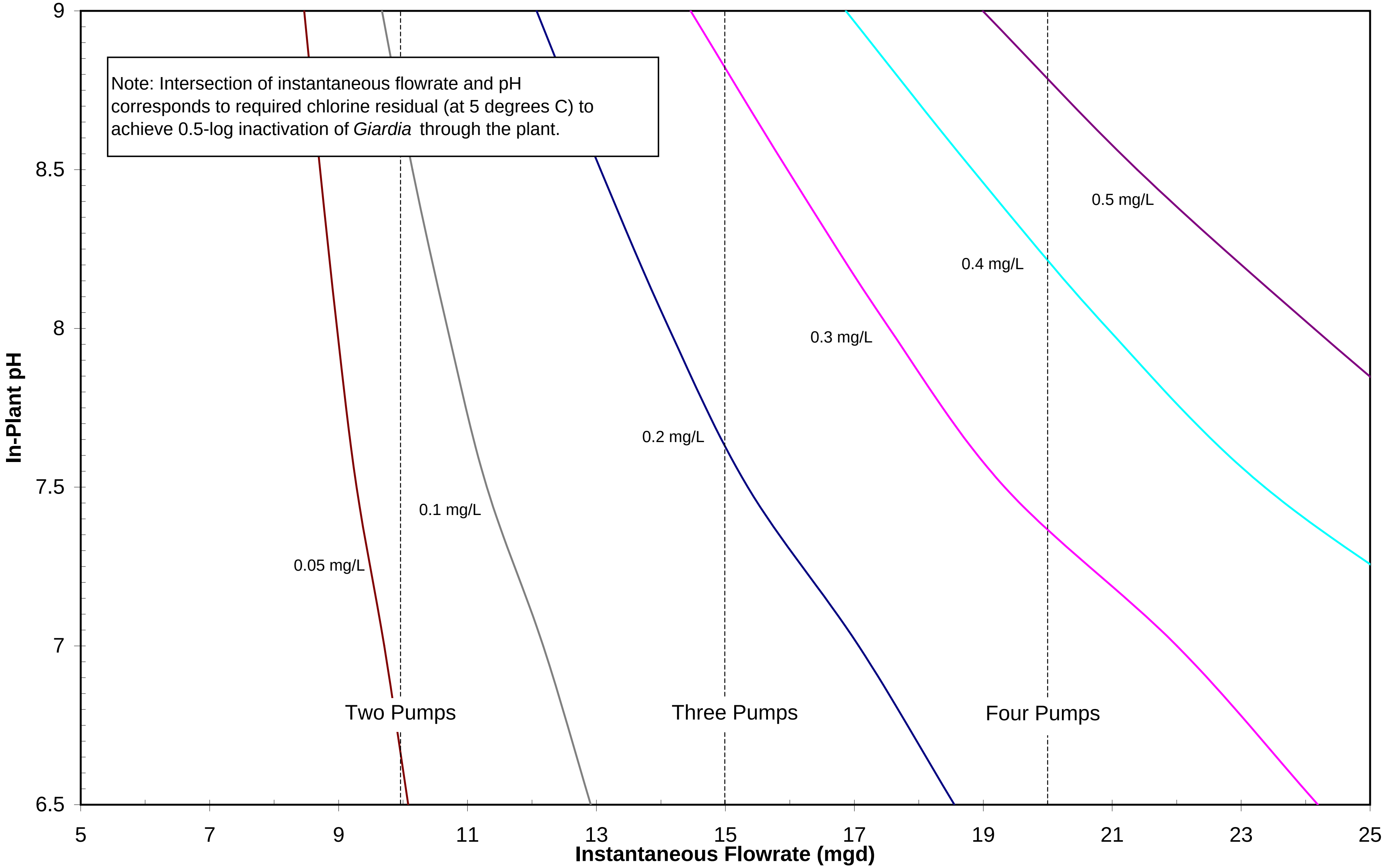


Figure 3-3

Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia

Worst-Case Spring/Fall Condition (Temp = 10°C)

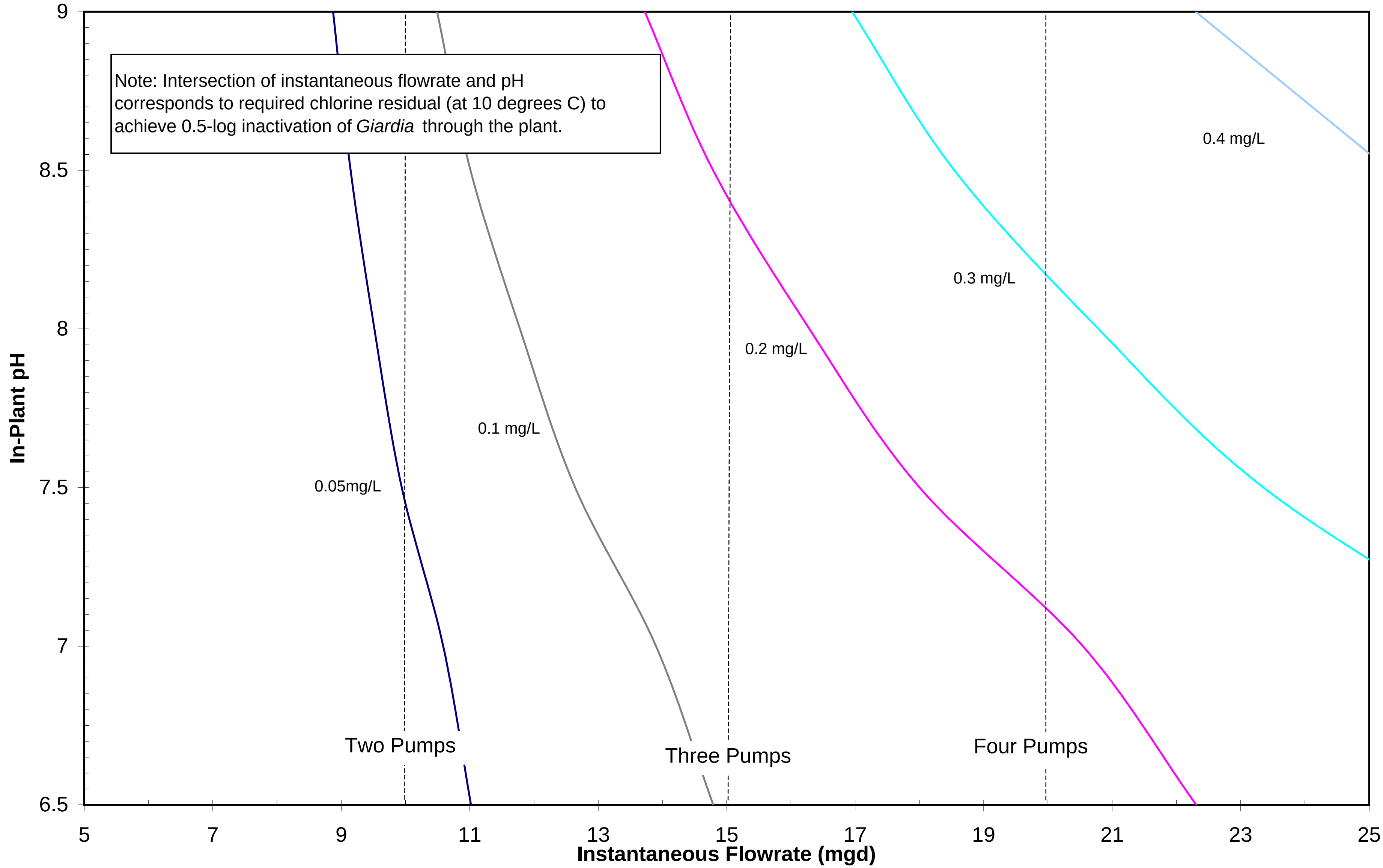
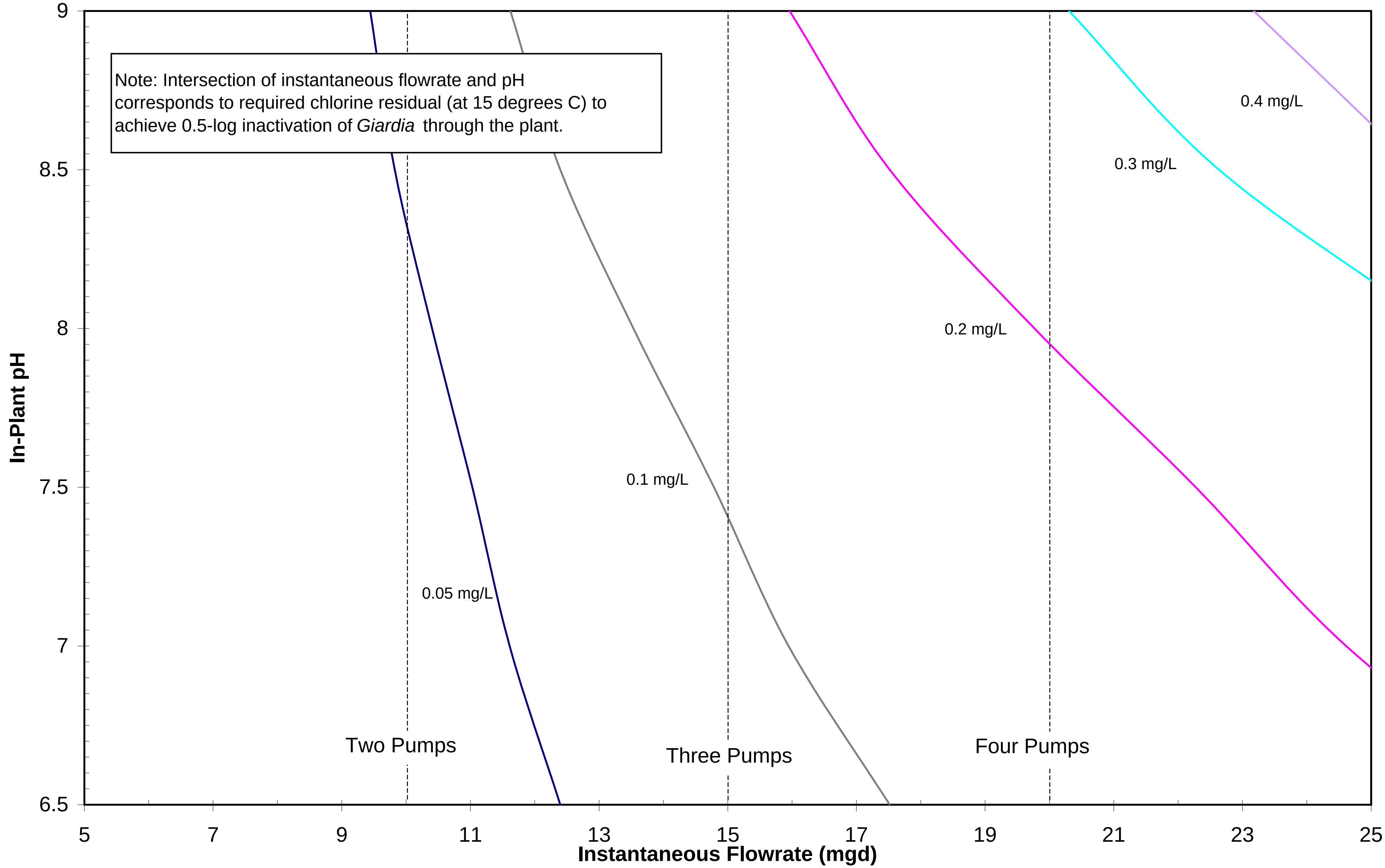
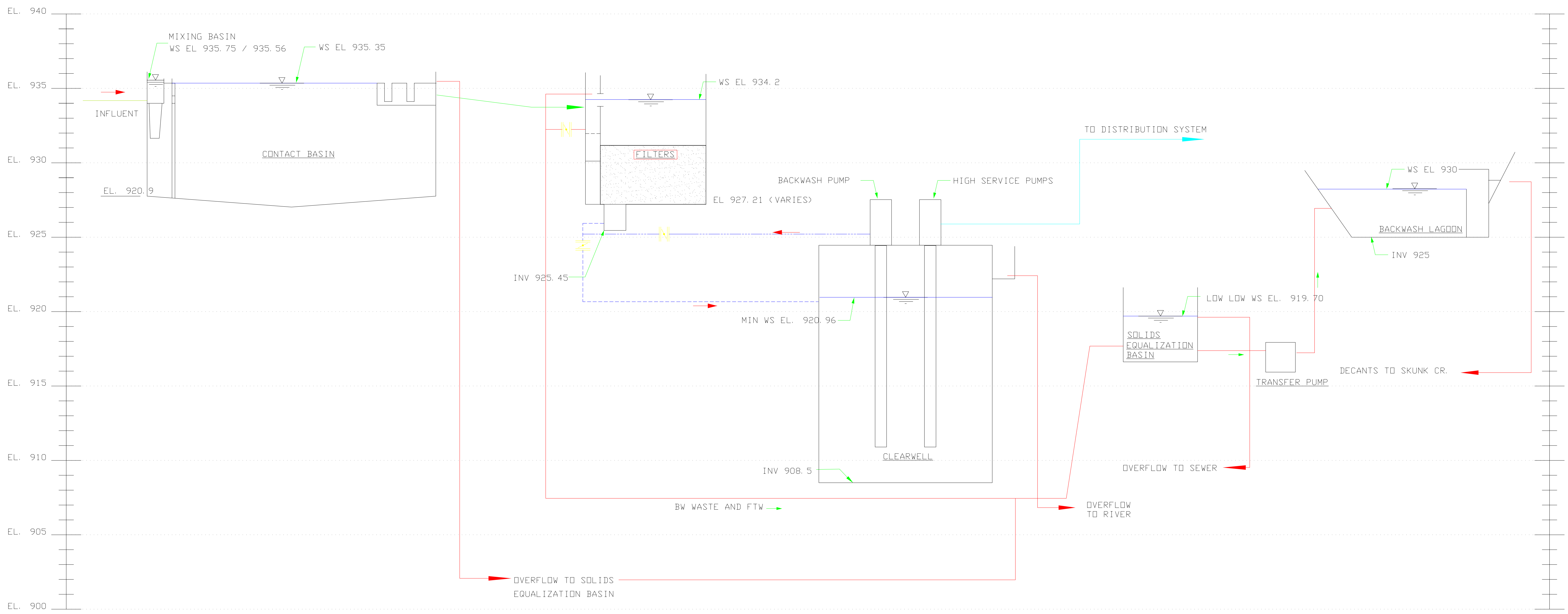


Figure 3-4
Chlorine residual (at filter influent) required to inactivate 0.5-log Giardia
Worst-Case Summer Condition (Temp = 15^{o}C)





LEGEND

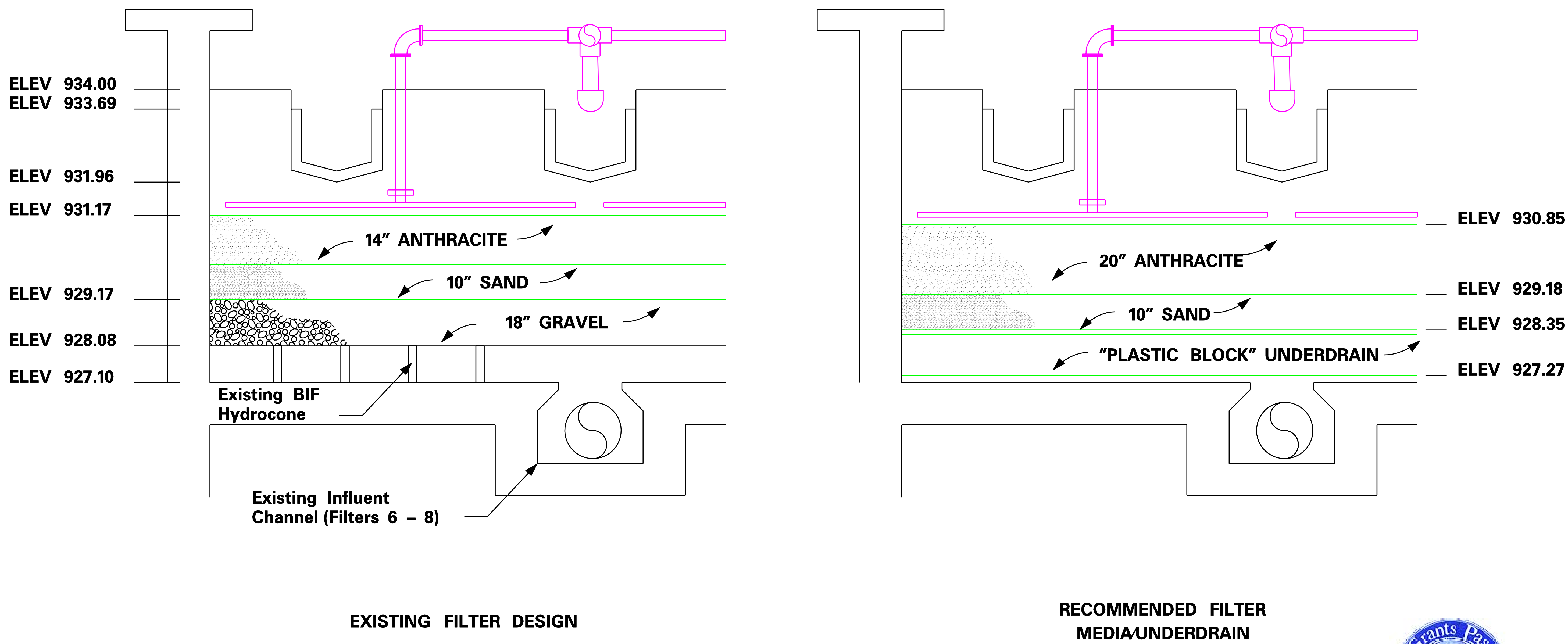
- SETTLED WATER
- FILTER EFFLUENT
- BACKWASH SUPPLY
- BACKWASH WASTE
- RAW WATER
- FINISHED WATER
- WATER SURFACE

NOTES:

1. ELEVATIONS FROM 1980 PLANT EXPANSION DRAWINGS.
2. BASED ON 20 mgd DESIGN FLOW.

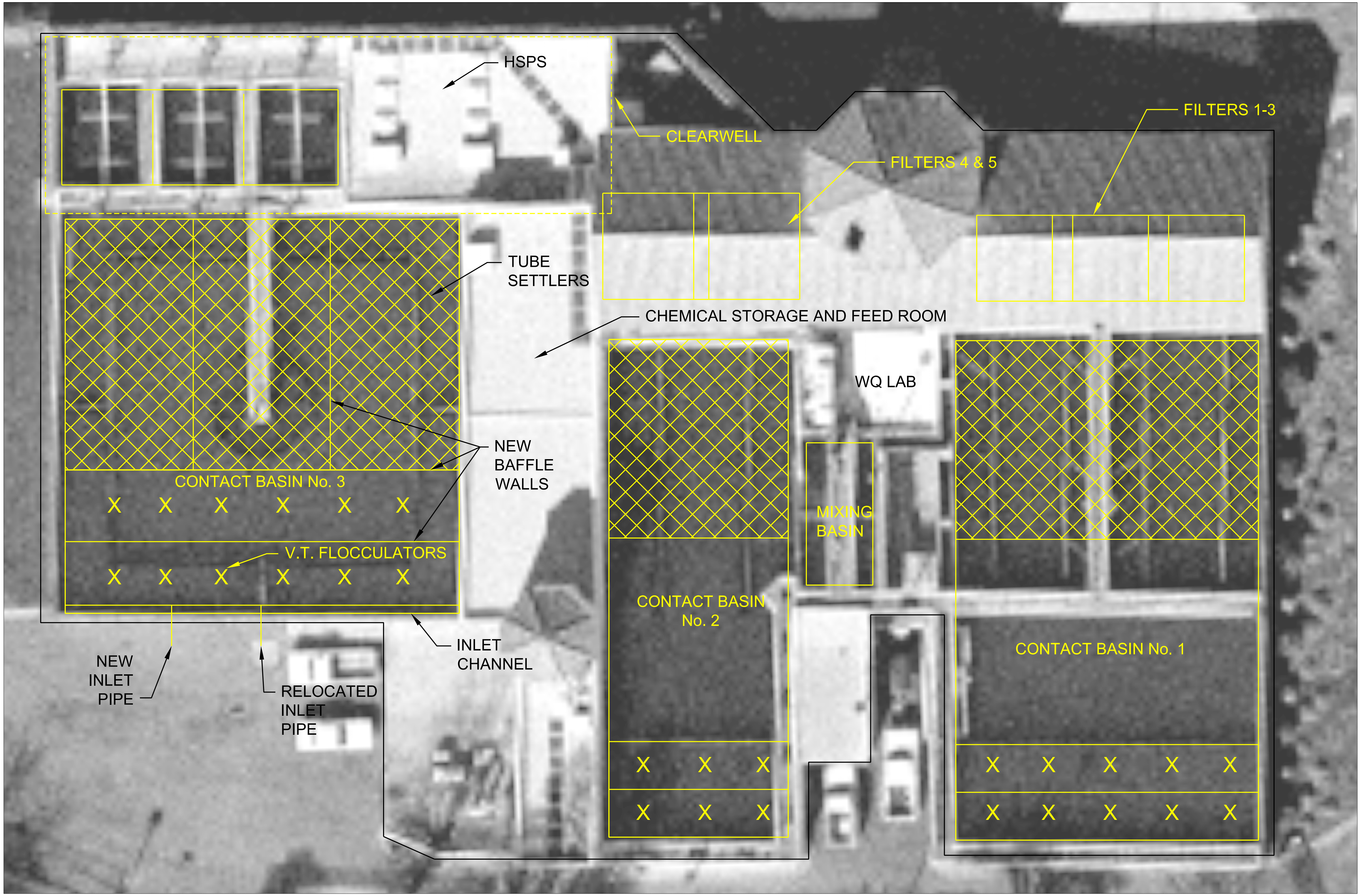
MWH

Figure 4-1
City of Grants Pass
Water Treatment Plant Facility Plan
Existing WTP Hydraulic Profile



MWH
MONTGOMERY WATSON HARZA

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



Basin 3 Modifications

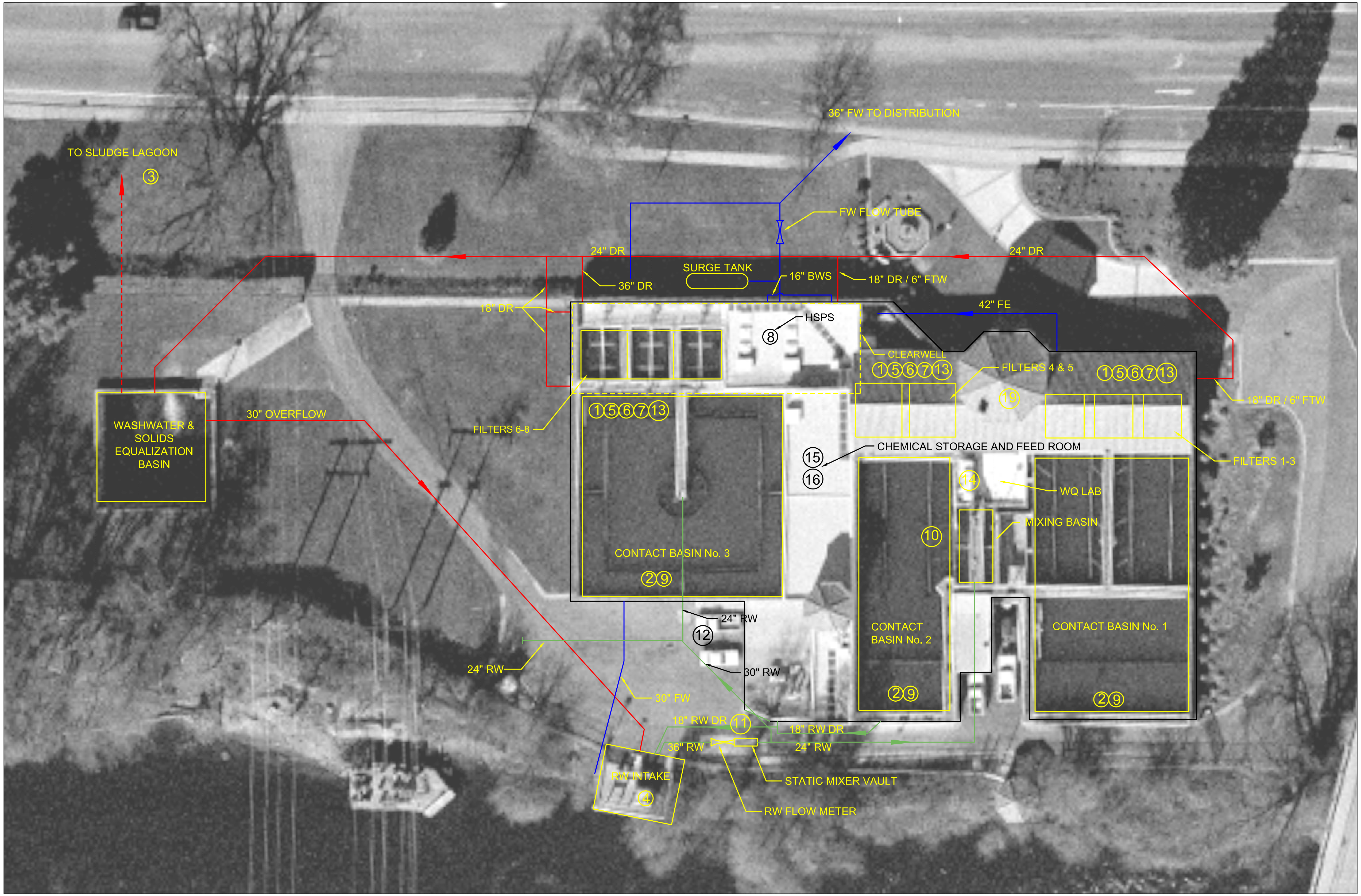
- Remove walkway and center feed
- Remove launders
- Relocate inlet pipe
- Re-shape floor for sludge removal
- Continuous sludge removal system (Not Shown)

MWH

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

MWH
MONTGOMERY WATSON HARZA

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



Tier One Improvements

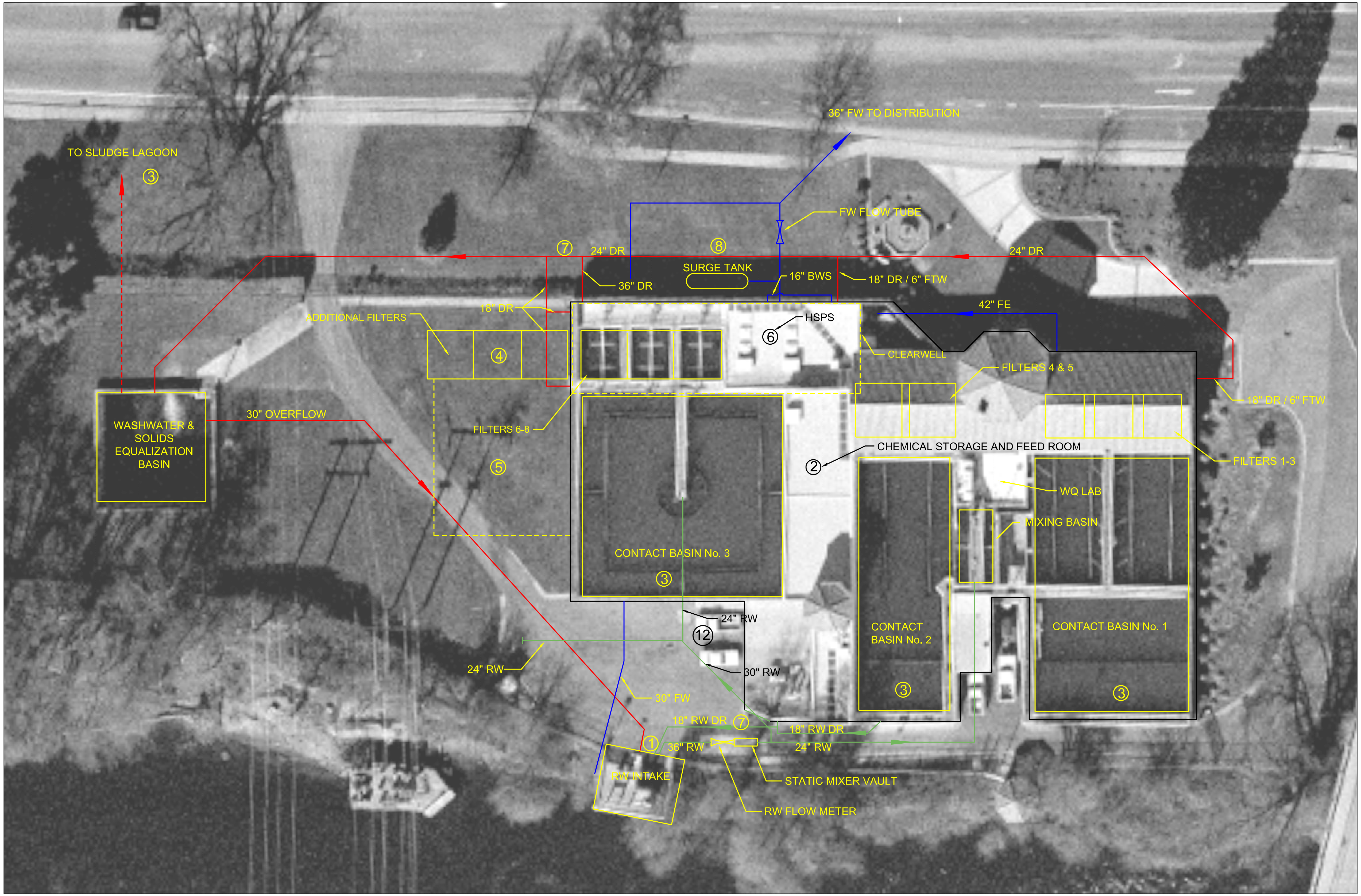
1. Re-build Filters
2. Add Floc and Baffling
3. Solids Handling and Disposal
4. Intake Modifications (30 MGD)

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)

MWH

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



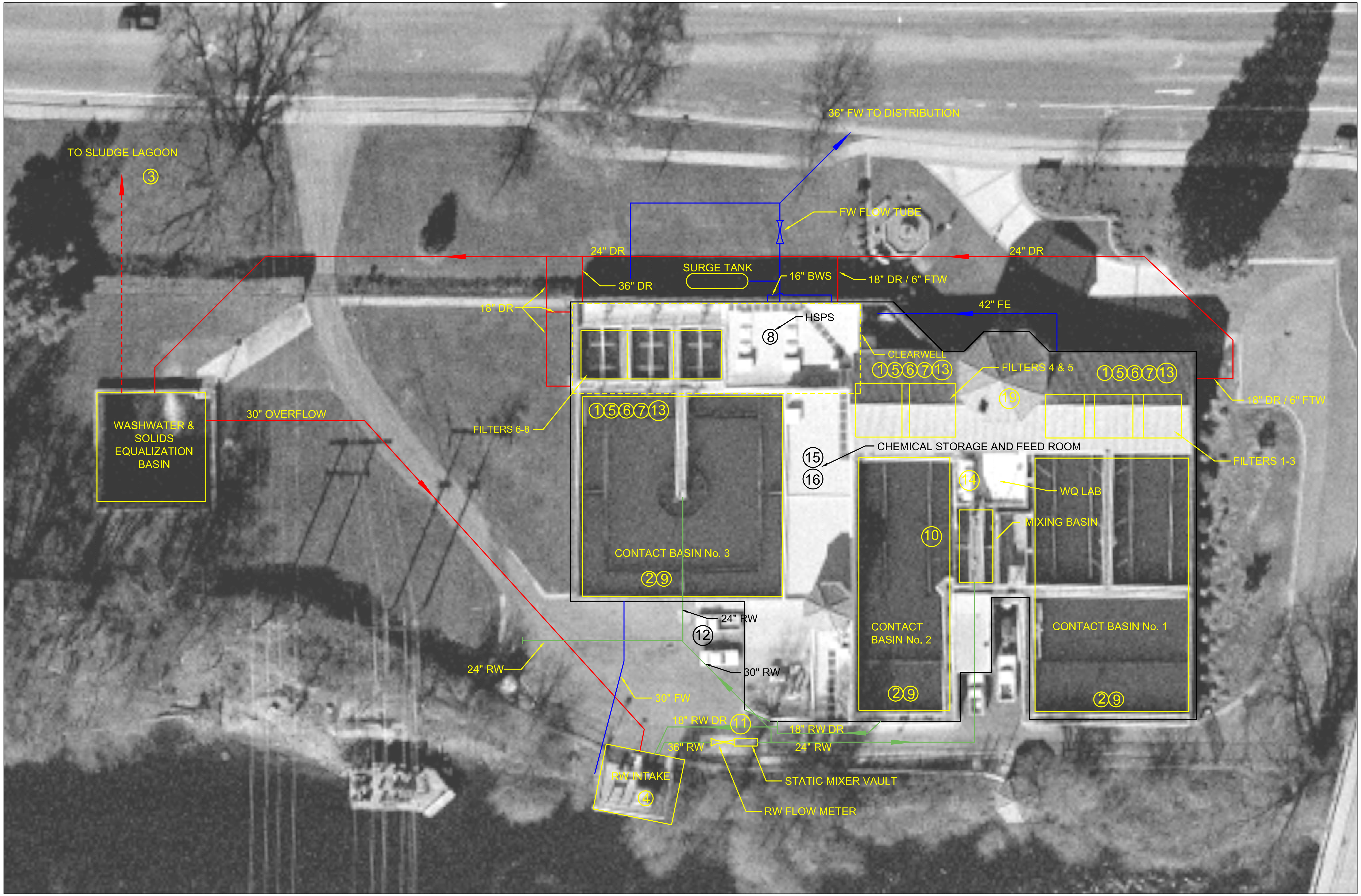
Plant Improvements (30 MGD)

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)

City of Grants Pass Oregon 1887

MWH

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



Tier One Improvements

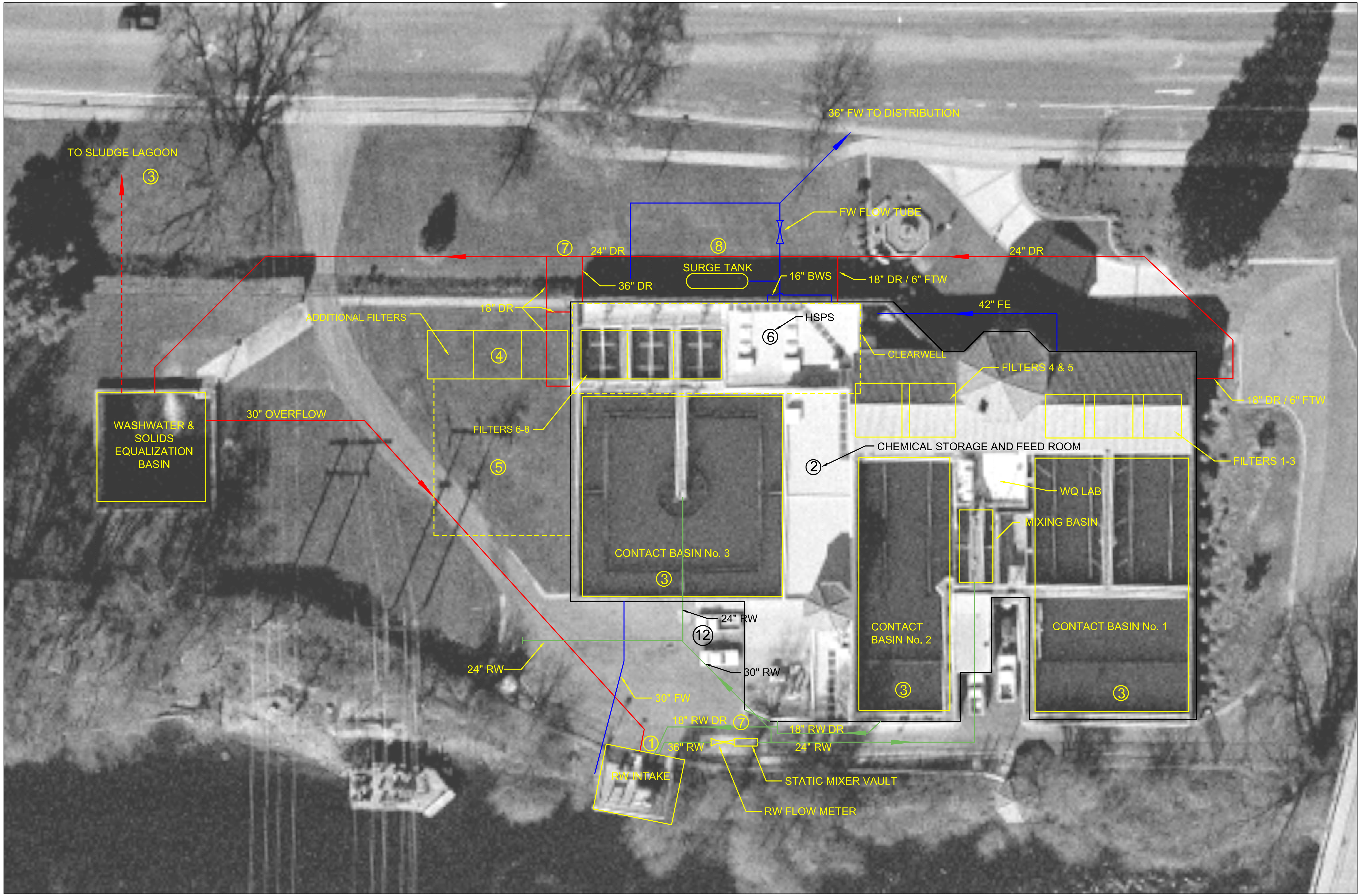
1. Re-build Filters
2. Add Floc and Baffling
3. Solids Handling and Disposal
4. Intake Modifications (30 MGD)

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)

MWH

Figure ES-1
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 ES-2
City of Grants Pass
Water Treatment Plant Facility Plan
Improvements to Expand to 30 MGD

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		Zurn/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 Teirnan/ Encore 700	0.75 hp/16.7gph
Post-chlor Metering	1		Wallace and Teirnan/ Encore 700	0.75 hp/16.7gph
Back-up Metering	1		Wallace and Teirnan/ 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 Cl-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