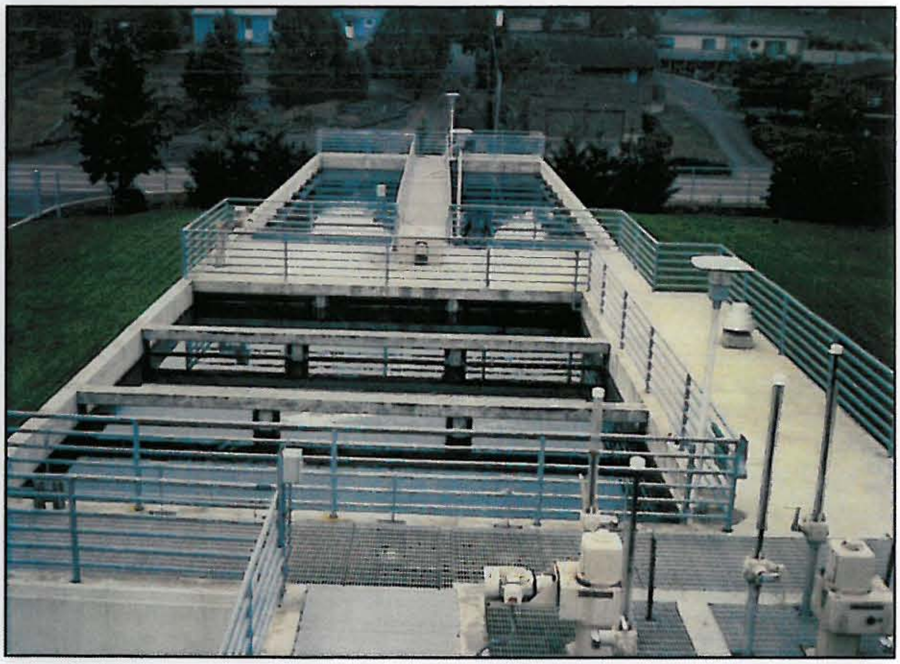
CITY OF ROSEBURG
WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

PROJECT NO. 06WA23

July 2009



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WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

FOR

CITY OF ROSEBURG, OREGON

JULY 2009

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JULY 13, 1973
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EXPIRES 6/30/10

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09-1015.401

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REFERENCES

A. "Long-Range Water Supply Plan, City of Roseburg, Oregon", July, 2009, Murray, Smith & Associates, Inc.

ABBREVIATIONS

AC	Asbestos-Cement
ACH	Aluminum Chlorhydrate
ADD	Average Daily Demand
CT	(Concentration) X (Time) in units of mg/l*min.
DBP	Disinfection By-Product
D/DBPR	Stage 2 Disinfectant/Disinfection By-Product Rule
DHS	Department of Human Services
ESA	Endangered Species Act
fps	Feet per Second
FTW	Filter-to-Waste
gpm	Gallons per Minute
gph	Gallons per Hour
gpm/sf	Gallons per Minute per Square Foot
gal/sf	Gallons per Square Foot
HAA_5	Haloacetic Acids (five compounds regulated as a group)
HDPE	High Density Polyethylene
HMI	Human-Machine Interface
hp	Horsepower
HSPS	High Service Pump Station
IDSE	Initial Distribution System Evaluation
IESWTR	Interim Enhanced Surface Water Treatment Rule
IOC	Inorganic Contaminant
LCR	Lead and Copper Rule
LRAA	Locational Running Annual Average
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
MCL	Maximum Contaminant Level
MDD	Maximum Day Demand
mgd	Million Gallons per Day
mg/L	Milligrams per Liter
mg/l*min	Milligrams per Liter per Minute
MIB	Methylisoborneal
MRDL	Maximum Residual Disinfectant Level
MSA	Murray, Smith & Associates, Inc.
NOM	Natural Organic Matter
NPDES	National Pollutant Discharge Elimination System
NPDWR	National Primary Drinking Water Regulations
NTU	Nephelometric Turbidity Unit
PAC	Powdered Activated Carbon

pCi/L	Picocuries per Liter
PDR	Preliminary Design Report
ppd	Pounds per Day
ppb	Parts per Billion
SCADA	System Control and Data Acquisition
scfm	Standard Cubic Feet per Minute
scfm/sf	Standard Cubic Feet per Minute per Square Foot
SCM	Streaming Current Monitor
SDWA	Safe Drinking Water Act
SLR	Surface Loading Rate
SOC	Synthetic Organic Compound
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TDH	Total Dynamic Head
T&O	Taste and Odor
TOC	Total Organic Carbon
THM	Trihalomethane
TTHM	Total Trihalomethanes
ug/L	Micrograms per Liter
μm	Micrometers (10^{-6} meters)
UBWV	Unit Backwash Volume
UFRV	Unit Filter Run Volume
UV	Ultraviolet
VOC	Volatile Organic Compound
VFD	Variable Frequency Drive
WTP	Water Treatment Plant

EXECUTIVE SUMMARY

Preface

A final report was submitted to the City of Roseburg (City) in June, 2008. Subsequently, the City updated its population forecasts which resulted in revised water demand forecasts. Murray, Smith & Associates, Inc. (MSA) was authorized to update the prior report to reflect any changes due to the revised water demand forecasts as well as to update certain sections of the report at the same time. Section 3 – Regulatory Overview is updated to reflect some subsequent regulatory changes, which are relatively minor, and the City's subsequent raw water testing results. Section 7 – Recommendations and Implementation Plan is updated to reflect current project costs and the revised proposed scheduling for plant improvements. An updated funding analysis and implementation plan for the Phase I plant expansion is included in the report. The Executive Summary is updated to reflect all of the report changes. The remaining sections of the report are unchanged. The revised water demand forecasts and other updating in the report have had no substantive impacts upon the conclusions and recommendations of the prior report.

Authorization

In June 2006, the firm of Murray, Smith & Associates, Inc. (MSA) was authorized by the City to undertake and complete this Water Treatment Facilities Pre-Design Report for the City's Winchester Water Treatment Plant.

Key Project Issues

A number of key issues to be addressed in conjunction with the planning for the plant expansion have been identified by City staff and the consultant team. These issues are outlined below.

- The appropriateness of the existing treatment processes and equipment for plant expansion
- Impact of drinking water regulations promulgated since the plant was constructed on selection of technology for plant expansion
- The potential to up-rate the capacity of existing processes
- Alternative treatment technologies
- Coordination of pre-design for plant expansion with long-range water supply plan
- Potential land acquisition requirements
- Determine the upgrading requirements for finished water pumping and surge control
- Evaluate the river intake
- Meet chlorine CT requirements at a higher plant capacity
- Develop a cost-effective plan to meet the long-term needs of the City

The expansion plan for the treatment plant needs to meet the long-term water demand requirements of the City's system in the most cost-effective manner. The expansion plan should address the key issues and result in a capital improvements plan that addresses project scheduling, cost, constructability and contract efficiency issues.

Historical Plant Performance

General: Historic water quality and operating data for the City of Roseburg's Winchester WTP were reviewed and analyzed. Data reviewed included selected raw, finished and distribution system water quality parameters, chemical usage data, sedimentation basin performance, and overall filter performance indicators. Discussions with plant operators were used to supplement and verify this information. The purpose of this data review is to assist in determining the performance of the existing WTP processes for operational efficiency and regulatory compliance. This performance evaluation is used, in part, to determine the process selection for the plant expansion.

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, since plant startup in 1992; however, plant production efficiencies are typically less than 97 percent, the desirable filter production efficiency, throughout the year, and generally decrease to as low as 92 percent when plant production is lower. Plant efficiencies can be improved through longer filter run times. This would reduce the cost for each unit of water produced by reducing pumping, chemical costs and washwater production per unit volume.

A summary of historical plant performance and analyses is presented below along with observations of potential improvements and/or further study.

- The plant has performed well and reliably over a range of flows and water quality conditions.
- The plant has produced up to 11.5 mgd during peak demand periods.
- The plant has successfully treated the North Umpqua River supply even during extremely high turbidity events (up to 1,000 NTU) during 1995 to 1997.
- Coagulation chemistry seems to have improved with the use of aluminum chlorhydrate (ACH) since 1999. This chemical produces excellent settled water and filtered water turbidities while also reducing plant operating costs through lower chemical costs and less sludge production compared to alum.
- The single flocculation/sedimentation basin uses a high-rate design with tube settlers to minimize space requirements and provides excellent pretreatment per historical data.
- The City should consider installing on-line turbidimeters to continuously monitor settled water turbidity. This work can be accomplished as part of the proposed plant expansion.

- The plant has four mixed-media gravity filters with the support gravel and media being almost 10 years old since the 1997 re-build. The filters produce excellent filtered water and do not have any apparent problems at this time.
- The filter backwash procedures appear to be acceptable in maintaining clean filter beds.
- The City should perform detailed filter investigations, including corings, sieve analyses and backwash efficiency analyses, in the next one to two years to assist in the determination of the remaining useful life of the filter media and support media. This work can be accomplished as part of the proposed plant expansion.
- The City should install on-line particle counters for each of the filters to operate in parallel with the on-line turbidimeters. This work can be accomplished as part of the proposed plant expansion.
- The City should consider extending the maximum filter run length to as long as 48 hours to achieve better production efficiencies.
- The City should purchase a bench-top ultraviolet (UV) spectrophotometer to monitoring total organic carbons. This should be accomplished independent of the proposed plant expansion.
- The finished water pH occasionally drops below 7.5 during late fall and winter conditions. The City may wish to consider seasonal use of the existing chemical feed system with a pH adjustment chemical such as lime or soda ash to maintain a minimum finished water pH of 7.5 to 8.0. This would provide a more-consistent and less corrosive finished water quality throughout the year.

Regulatory Overview

General: 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 is provided. 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 the City's 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.

Summary and Recommendations: In general, the Winchester WTP has consistently met all existing water quality regulations since it started operations in 1992. The two biggest drinking water regulatory issues of concern at this time are:

- Ability to consistently meet 0.5-log *Giardia* inactivation following filtration under all current and future plant flows and under a wide range of plant operating conditions (clearwell level, finished water temperature, finished water pH and finished water chlorine residual), and

- Bin classification per the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) depending on raw water *Cryptosporidium* concentrations.
- Compliance with maximum approach velocity to the raw water intake for protection of salmonid fish species at flows above 16.5 mgd.

The City should evaluate post-filtration CT compliance as part of the preliminary design effort for the upcoming plant expansion, as well as discuss the CT calculation methodology further with Oregon Department of Human Services (DHS). The evaluation should include a tracer study to determine the actual baffling efficiency in the existing clearwell. A tracer study will be completed in the fall of 2009. These data can then be used for updating the City's calculations of CT and for designing improvements to the clearwell baffling that will be needed to increase contact time for the plant expansion. The City should continue and complete its 2-year monitoring program for *Cryptosporidium*. That program will be completed in the summer of 2009.

Based on historical low disinfection by-product (DBP) concentrations within the Roseburg system, it is anticipated that the City will have no major compliance issues for the Stage 2 Disinfection By-Products Rule (D/DBPR).

The City should continue to monitor its raw and finished water total organic carbon (TOC) on a monthly basis to ensure continued TOC removal compliance through the plant. 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.

The City should consider adding on-line particle counters to each filter effluent, in parallel with filtered water turbidimeters, to better understand filter performance and to anticipate turbidity breakthrough and other performance problems earlier.

The finished water pH occasionally drops below 7.5 during late fall and winter conditions, although the pH has not dropped below 7.0 since the City changed coagulation and disinfection chemicals. Even though the City has remained in compliance with the Lead and Copper Rule, the City may wish to maintain a minimum finished water pH of 7.5 to 8.0 to provide a more-consistent and less corrosive finished water quality throughout the year. This would require seasonal use of the existing chemical feed system to add a pH adjustment chemical such as lime or soda ash.

Capacity Review

General: A review of the hydraulic and treatment capacities of the Winchester WTP was performed to determine the current capacity and possible future capacity given the limitations of each process and the interconnected system as a whole. The hydraulic capacity is related to the piping, pumping, volume and flow control systems, which limit the ability of the water to flow through individual unit operations and through the interconnected system as a whole. The treatment capacity is related to the ability of each treatment unit process to meet regulatory requirements or generally accepted industry standards, whichever is applicable.

The Winchester WTP has a current rated plant capacity of 12 mgd. This is a treatment capacity limitation determined by the four existing filters. Most of the existing equipment and treatment processes already have capacities exceeding that value. The plant was designed to be expanded to a nominal capacity of 18 mgd. This is also a treatment capacity limitation based on an ultimate build out of six filters using the same media as the existing four filters. Several unit operations will require modification to achieve this capacity.

The City currently has water rights on the North Umpqua River totaling 20 mgd and has investigated the possibility of obtaining additional rights from that source as part of the City's Long-Range Water Supply Plan. This topic is discussed in greater detail in that plan. Since the existing water rights exceed 18 mgd, and because there is the possibility of obtaining additional rights, the ability of individual unit operations to accept higher flow rates, between 18 and 24 mgd, is analyzed. For each unit operation, the likely ultimate hydraulic and treatment capacity is analyzed, modifications required to achieve the higher rate are identified, and suggested improvements are noted if they appear to be feasible. Figure ES-1 shows the general site plan for the existing Winchester WTP. Figure ES-2 shows the plant hydraulic profile.

<u>Summary of Hydraulic Capacity Evaluation:</u> The plant has the hydraulic capacity to handle 18 mgd and may be capable, with appropriate modifications, of achieving a hydraulic capacity of up to 24 mgd. Unit processes that need modifications to achieve a hydraulic capacity of 18 mgd are:

- Fish Screening at Intake
- Raw Water Pumping
- Flocculation/Sedimentation
- Settled Water Transmission
- Filtration
- Clearwell
- Finished Water Pumping

The unit processes that will need to be modified or replaced subsequently to achieve an ultimate plant capacity greater then 18 mgd are:

- Fish Screening at Intake
- Raw Water Pumping
- Flocculation/Sedimentation
- Settled Water Transmission
- Filtration
- Clearwell
- Finished Water Pumping.
- Washwater and Solids Handling

Summary of Treatment Process Capacity Evaluation: The following is a summary of the treatment process capacity evaluation. The conclusions of this evaluation are:

1. Chemical Feed Systems: With the exception of the sodium hypochlorite system, the existing chemical systems are adequate without modification for the expanded plant capacity to 18 mgd and up to 24 mgd.

2. Flocculation/Sedimentation: Constructing a second flocculation and sedimentation basin in parallel to the existing basin should achieve up to 24 mgd of pretreatment capacity. Pretreatment performance will improve at 18 mgd capacity compared to the existing performance with the single basin. At 24 mgd, flocculation/sedimentation should approximate existing performance.

3. Filtration: Adding two new filters with the same mixed media configuration as the existing four filters will provide for treatment of up to 18 mgd. For treatment of flows in excess of 18 mgd, the existing mixed media configuration is not recommended. Use of a deep media design in the six filter basins that comprise the plant's ultimate build out would provide for a maximum treatment capacity for filtration up to 22 mgd.

4. Clearwell: Clearwell storage is insufficient for flows above 18 mgd. For flows up to 18 mgd, the storage volume may also be insufficient since the minimum water level needed for disinfection contact time requires that the majority of the treated water in the clearwell not be available as storage.

5. Disinfection By-Products: DBP formation is low and is expected to remain low in the future. Alternative disinfection strategies are not required to meet future regulations.

6. Washwater and Solids Handling System: The new system is likely to be adequate for treating the solids generated by flows up to 18 mgd. The effectiveness of this system for treating the solids generated by higher flows can only be evaluated after the system has operated for some time, but it is considered unlikely that the system will be able to treat the solids generated by flows greater than 18 mgd.

Conclusions and Recommendations: The ultimate plant capacity for the Winchester WTP will depend upon whichever is limiting among the following factors:

1. The extent to which the existing configuration and treatment unit operations can be economically upgraded to increase hydraulic and treatment capacities within the existing property.

2. The hydraulic or treatment capacity of alternative treatment unit operations that could be economically located within the existing property.

3. The economic feasibility of obtaining additional, adjoining property to ensure sufficient space for treatment of higher flows using either an expansion of existing technology or replacement of existing technology with alternative treatment technology.

4. The maximum withdrawal from the North Umpqua River that can be achieved combining the City's current water rights with any additional rights that the City may be able to secure on the North Umpqua River.

Typically, it is more economical to expand the capacity of an existing treatment facility than to construct a new facility on a site with no existing treatment infrastructure; therefore, it is desirable in the long term to expand the plant capacity to treat all the water from the North Umpqua for which the City has or can obtain rights. This is preferable to forfeiting any opportunity to obtain additional water rights because of capacity limitations at the Winchester facility. Using the existing four filters and the two filters to be added during the upcoming expansion, the treatment capacity limitation using the current treatment technology is 22 mgd. This capacity limitation is based on filtration and assumes that the following improvements can be made to other treatment units:

1. The clearwell volume available for disinfection contact time can be increased to provide sufficient CT, or alternative disinfection technology is added when the plant is expanded from 18 to 22 mgd.

2. The recently constructed washwater and solids handling system is determined to be capable of handling the solids generated at 22 mgd or additional solids handling capacity is constructed. This can be done using additional space to expand the existing system or by replacing the existing system with alternative technology having a smaller footprint.

It is recommended that the City proceed to expand the plant to 18 mgd in the near term and to plan for an ultimate capacity of 22 mgd in the long term. Based on the Long Range Water Supply Plan, the Winchester WTP will need to be expanded beyond 18 mgd by approximately the year 2025.

To ensure that there is maximum flexibility and economy in selecting the best technology to be employed for expansion of the plant beyond 18 mgd and to ensure that the City can address any new regulations that may be promulgated in the intervening years, it is recommended that the City purchase additional land adjoining the WTP property to the west for potential future plant expansion to 22 mgd and potentially beyond this capacity. The additional property could be utilized for some or all of the following processes:

- Additional clearwell volume for disinfectant contact time.
- Additional clearwell volume for treated water storage.
- Additional washwater and solids handling system facilities.
- Additional or modified process technologies driven by new regulations.
- Additional potential future unidentified treatment facilities for potential expansion beyond 22 mgd if additional water rights and/or raw water supplies can be acquired.

Assuming that an additional sludge drying bed and a 1.25 mg clearwell will be constructed on the property to be acquired, it is recommended that the easterly 200 feet of Tax Lot 800 be acquired. This portion of the tax lot is approximately 1.54 acres and it has an estimated total assessed value of approximately $350,000. Figure ES-3 shows the property that is proposed to be acquired.

The ability of the City to protect its existing water rights and to acquire additional water rights and potentially other raw water supplies for treatment at the Winchester WTP site are not known at this time. The Long-Range Water Supply Plan recommends further actions and studies to address these questions. The City may desire to defer acquiring the additional property pending development of further information on water rights and other water supplies that will more fully support the need to acquire the additional property.

Facilities Condition Review

Each of the Winchester Water Treatment Plant's major equipment, systems and structures were observed to determine their existing condition and to determine if replacement, upgrading or other improvements are required presently. This facilities condition review was conducted to help the City make decisions regarding maintenance and equipment replacement and upgrading requirements for all process and support facilities at the plant. Recommendations are then made for repairs, upgrading or improvements to be accomplished prior to and independent of the plant expansion program and those recommended to be accomplished as part of the plant expansion.

An estimate of the useful life of each item of major equipment, system or structure is made and the approximate estimated remaining useful life calculated based upon the year of installation or construction. Equipment, systems or structures which have a remaining estimated useful life of 20 years or less are noted. Recommendations are then made as to upgrading or replacement of equipment, system and structures which have less than 20 years of remaining useful life. Budget costs are provided for those items. This work constitutes a capital maintenance plan for the Winchester Water Treatment Plant. The total estimated cost in present dollars of the 20-year capital maintenance plan is $687,000.

Alternatives Analysis for Critical Process Issues

General: Several critical process issues were analyzed at the Winchester WTP. These issues included alternatives for fish screening at the river intake, alternative clarification approaches in the sedimentation basins, membrane technology as an alternative to conventional granular media filtration, and alternative approaches to meeting CT requirements including use of alternative disinfectants.

Fish Screening at Intake: The intake and traveling fish screens complied with the regulations that existed at the time they were installed. At flows above 16.5 mgd, the requirements of the current regulations will be violated. Fixed fish screens on the exterior of the structure will need to be installed at that time to meet the current fish screening requirements for intake structures.

Sedimentation Basin Settlers: Clarification technologies for conventional treatment have advanced in recent years. As an alternative to installing tube settlers, the installation of Lamella plate settlers was considered. Although tube settlers have a shorter life expectancy than Lamella plates, tube settlers have a lower life-cycle cost. The tube settlers have performed well in the basin at the Winchester WTP and it is recommended that plastic tube settlers be installed in the new basin.

Filtration: Two options for expanding the treatment capacity of the filtration process were considered - maintain the existing, conventional treatment process adding one new flocculation/sedimentation basin and two new granular media filters or replace the conventional media filters with a membrane filtration system. Retrofitting the existing plant with submerged membrane technology to achieve an expanded capacity of approximately 20 mgd is estimated to cost approximately $15 million. Further consideration of membrane filtration for expansion of the Winchester WTP is not warranted.

The expansion of the Winchester WTP to 18 mgd should be accomplished by adding two new filters using the same media configuration as the existing four filters. If a subsequent plant expansion is undertaken to increase capacity beyond 18 mgd, replacing the existing granular media design with a deep bed design would expand capacity to about 22 mgd. Based on this treatment capacity limitation, it is recommended that 22 mgd be considered the ultimate Winchester plant capacity using the plant’s current treatment technologies.

Disinfection: The existing clearwell configuration coupled with current plant operating practices limit the plant’s ability to meet the regulatory requirement for 0.5-log *Giardia* inactivation after filtration, even under current flow and water quality conditions.

The City should immediately conduct a tracer study to determine the clearwell contact time under existing conditions. The results of the tracer study can be used to change the manner in which CT is presently calculated and reported to the State. The results can also be used to determine how much additional contact time will be needed to meet CT at the expanded capacity of 18 mgd. Improvements to the baffling in the clearwell that will increase the contact time should be included in the design of the plant expansion. It is likely that the existing clearwell can be modified to provide adequate CT when the new filters are constructed to treat up to 18 mgd. For disinfection at plant capacities greater than 18 mgd, additional contact volume will be required or alternative disinfection processes must be introduced. The tracer study will be completed by the City during the fall of 2009.

Rather than increasing the clearwell contact time to ensure proper disinfection with free chlorine, the City could transition to an alternative disinfection process. Hypochlorite would continue to be used, but solely to maintain a residual in the distribution system. Alternative disinfection processes to consider would include ozone, ultraviolet light (UV) and chlorine dioxide. While all of these alternative disinfection processes could be used, they are all substantially more expensive and complex than the sodium hypochlorite that is currently used.

A portion of the finished water transmission system directly downstream of the plant could be used to provide additional contact volume to meet CT requirements beyond that provided by the clearwell. It is preferable to retain all treatment unit operations at the water treatment plant site and deliver only completely treated water that meets all regulatory requirements from the site. This approach allows for better control of the treatment process, provides greater ease of operation and provides the greatest assurance of full regulatory compliance. It is recommended that the City provide for both disinfectant contact volume and treated water storage volume on the existing or expanded plant site.

<u>Recommendations:</u> The City should immediately conduct a tracer study, using a methodology approved by DHS, and begin using the results of the tracer study to change the manner in which CT is presently calculated and reported. The design for expansion to 18 mgd should include improvements to the baffling in the clearwell to increase the contact time to the greatest extent possible. A second tracer study should be conducted after the expansion to 18 mgd to determine the contact volume with the modifications. Hypochlorite generated on-site should continue to serve as the disinfectant until the plant is expanded from 18 mgd to its ultimate capacity of 22 mgd. Determination of the preferred disinfection alternative for the expansion to 22 mgd can be made after tracer tests are done on the modified clearwell and after testing for *Cryptosporidium* in the source water, in line with the requirements of the LT2ESWTR, has been completed.

Recommendations and Implementation Plan

General: The plant has performed well as it approaches its current design capacity of 12 mgd. Expansion of the plant using the current treatment technology can increase the plant capacity to 18 mgd. Figure ES-4 which follows shows the existing plant site with the major recommended improvements identified for the expansion to 18 mgd. With appropriate modification of the filter media, the treatment capacity of the plant can be expanded to 22 mgd with six filters in operation. Based on this filter treatment capacity, it is recommended that 22 mgd be considered the ultimate capacity for the Winchester WTP using rapid sand filtration. Figure ES-5 which follows is the water demand and water supply schedule from the City's companion long-range water supply plan and illustrates the recommended and plant expansion increments and schedules.

Significant Regulatory Compliance Issues and Recommended Actions: The three most significant regulatory issues of concern regarding the existing plant and the plant expansion are:

- Ability to consistently meet 0.5-log *Giardia* inactivation following filtration under all current and future plant flows and under a wide range of plant operating conditions.
- Bin classification per the LT2ESWTR depending on raw water *Cryptosporidium* concentrations.
- Compliance with maximum approach velocity to the raw water intake for protection of salmonid fish species at flows above 16.5 mgd.

The following actions are recommended to address these compliance issues:

- The City should immediately conduct a tracer study to evaluate post-filtration CT compliance. That study will be completed in the fall of 2009.
- The City should continue and complete its 2-year monitoring program for *Cryptosporidium*. That program will be completed in the summer of 2009.
- Incorporate into the plant expansion improvements the installation of fixed screens in the river intake to replace the traveling screens or defer these improvements until the required plant production capacity approaches 16.5 mgd which is estimated to be in the year 2022.

Other Recommended Immediate Actions: Additional actions are recommended for immediate accomplishment. These are described as follows:

- Undertake and complete the work resulting from the evaluation of historical plant performance that is recommended to be accomplished independent of the proposed plant expansion.

- Undertake and complete the work resulting from the facilities condition review that is recommended to be accomplished independent of the proposed plant expansion.
- Complete the recommended administrative actions with the Oregon Water Resources Department to secure the City's existing water rights on the North Umpqua River at Winchester as described in the Long-Range Water Supply Plan.
- Undertake the recommended actions to seek to acquire additional water rights in the North Umpqua River Basin for use at the Winchester WTP to provide at least up to 22 mgd capacity as described in the Long-Range Water Supply Plan.
- Based upon the City's success in securing its existing water rights at Winchester, acquiring additional water rights at the plant site, and developing additional water supply at Winchester, consider proceeding with acquisition of property adjacent to the plant (portion or all of Tax Lot 800) to provide for expansion of the plant to beyond 18 mgd capacity and potentially to 22 mgd or more.
- Adopt the capital maintenance plan and budget for the plant.

Plant Improvements to Achieve 18 mgd Capacity: It is recommended that the Winchester Water Treatment Plant be expanded soon to 18 mgd capacity. The unit processes that need modification to expand the plant to 18 mgd are as follows:

- Fish screening at river intake
- Raw water pumping
- Flocculation/sedimentation basin
- Settled water transmission pipeline
- Filtration
- Clearwell baffling
- Finished water pumping
- Hydropneumatic surge control.
- On-site sodium hypochlorite generation system
- Improvements recommended to be accomplished as part of the plant expansion project resulting from the review of historical plant performance
- Improvements, repairs, replacements and upgrading recommended to be accomplished as part of the plant expansion project resulting from the facilities condition review

Plant Improvements to Achieve up to 22 mgd Capacity: Based on the Long-Range Water Supply Plan, the Winchester WTP will need to be expanded beyond 18 mgd by the year 2025. The general scope of work required and the unit processes that need modification to expand the plant up to 22 mgd are as follows:

- Fish screening at river intake (unless new fixed screens are designed to accommodate flows up to 22 mgd)
- Raw water pumping

- Filtration
- Clearwell contact time for disinfection:
- Clearwell storage volume
- On-site hypochlorite generation system
- Finished water pumping
- Hydropneumatic surge control
- Backwash waste and solids handling system
- Electrical system upgrades

Recommended Implementation Schedule

General: Water demands are approaching the current 12 mgd capacity of the Winchester Water Treatment Plant. The City's Long-Range Water Supply Plan has projected near-term and long-term water demands and recommends proceeding immediately with expansion of the Winchester plant to 18 mgd. This capacity is forecast to meet the City's water demands until the year 2025.

The Long-Range plan further recommends expansion of the plant by that time to its maximum capacity of up to 22 mgd assuming the continued use of the present conventional treatment technologies at the plant and the availability of water rights and water supply in that amount at the Winchester site. The following are descriptions of the recommended implementation schedule for the recommended work described above.

Phase 1 - Plant Improvements and Expansion to 18 MGD – 2009 through 2012

- Evaluate and achieve CT compliance
- Complete monitoring program for *Cryptosporidium*
- Acquire bench-top UV spectrophotometer
- Study shoaling condition in river at intake
- Replace high service pump station roof
- Complete river intake pump testing and rebuild, if necessary
- Replace all turbidimeters
- Complete miscellaneous work as identified in facilities condition review
- Acquire additional property
- Undertake and complete plant expansion program to 18 mgd

Phase 2 - Plant Improvements and Expansion up to 22 MGD – 2022 to 2025

- Undertake and complete plant expansion program from 18 mgd up to 22 mgd

Cost Estimates

Estimates of cost have been developed for the recommended work. Construction cost estimates represent opinions of cost only, acknowledging that final costs of projects will vary depending on actual labor and material costs, market conditions for construction, regulatory factors, final project scope, project schedules, and other factors.

Table ES-1 presents the estimated costs for the recommended regulatory compliance actions and other recommended immediate actions that are not included in the overall proposed Winchester plant improvement and expansion project.

TABLE ES-1

BUDGET ESTIMATES - REGULATORY COMPLIANCE AND OTHER IMMEDIATE RECOMMENDED ACTIONS (YEAR 2009)

Item	Estimated Budget, Current $
1. CT compliance review including tracer study & operator & DHS consultations	$8,000
2. Purchase UV spectrophotometer	$7,000
3. Property acquisition (portion of Tax Lot 800)	$350,000
4. Evaluation study of shoaling at river intake	$12,000
5. Replace roof of high service pump station	$25,000
6. Test and rebuild river intake pumps	$55,000
7. Replace turbidimeters	$20,000
8. Miscellaneous improvements per Section 5	$10,000
Total Estimated Budget	**$487,000**

Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

Table ES-2 presents the estimated project costs for the initial Winchester plant improvement and expansion phase work to be accomplished from 2009 through 2012. This phase will expand the plant to 18 mgd capacity. The estimate in this table includes an inflation allowance.

TABLE ES-2
PROJECT COST ESTIMATE
PHASE 1 – PLANT IMPROVEMENTS AND EXPANSION TO 18 MGD (YEARS 2009-2012)

Item	Estimated Cost, Current $
Estimated Construction Costs	
Fish screening at river intake	$690,000
Raw water pumping improvements	$102,000
Flocculation & sedimentation basin no. 2	$1,407,000
Filters 5 & 6	$1,833,000
Additional clearwell baffling	$300,000
Finished water pumping improvements	$429,000
Hydropneumatic surge system upgrading	$186,000
Total Estimated Direct Construction Cost	$4,947,000
Construction Contingency (15%)	$742,000
Total Estimated Construction Cost	$5,689,000
Allowance for Inflation (2 years - 3%/yr. – 6% total)	$341,000
Total Estimated Construction Cost With Inflation Allowance	$6,030,000
Estimated Indirect Costs	
Design Engineering (15%)	$904,000
Construction Engineering (10%)	$603,000
Administration, Legal, Permits & Approvals (1%)	$60,000
Total Estimated Indirect Costs	$1,567,000
Total Estimated Project Cost	**$7,597,000**

Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

Table ES-3 presents the estimated project costs for the second plant improvement and expansion phase work to be accomplished from 2022 through 2025. This phase expands the plant from 18 mgd to as much as 22 mgd. Since the scope of the work required for this expansion is only generally defined, the costs presented are conceptual level cost estimates. The costs presented do not include an inflation allowance.

TABLE ES-3
PROJECT COST ESTIMATE
PHASE 2 – PLANT IMPROVEMENTS AND EXPANSION UP TO 22 MGD (YEARS 2022 - 2025)

Item	Estimated Cost, Current $
Estimated Construction Costs	
Raw water pumping improvements	$425,000
Remove & replace underdrains on 4 filters & remove & replace media in 6 filters	$540,000
Construct 1.25 million gallon clearwell addition	$2,500,000
Expand on-site hypochlorite generation system	$125,000
Finished water pumping improvements	$290,000
Backwash waste and solids handling system	$675,000
Electrical power supply & distribution system upgrade	$350,000
Total Estimated Direct Construction Cost	$4,905,000
Construction Contingency (25%)	$1,226,000
Total Estimated Construction Cost	$6,131,000
Estimated Indirect Costs	
Design Engineering (15%)	$920,000
Construction Engineering (10%)	$613,000
Administration, Legal, Permits & Approvals (1%)	$61,000
Total Estimated Indirect Costs	$1,594,000
Total Estimated Project Cost	**$7,725,000**

Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

Conclusions

This preliminary design report evaluated the historical performance of the City's existing Winchester Water Treatment Plant. The report also reviewed the current and anticipated regulations governing water treatment, performed a hydraulic and treatment capacity review, evaluated the condition of the existing plant and developed a capital maintenance plan, and analyzed alternatives for critical processes. Recommendations for plant upgrading, improvements and expansion are then made and a plan to implement the recommendations is proposed. It is recommended that immediate actions consisting of plant repairs, improvements, further evaluations, and property acquisition be taken in the years 2009 and

2010. It is further recommended that the plant be expanded to 18 mgd by the end of the year 2012 and that, by approximately the year 2025, the plant be further expanded up to 22 mgd which is its approximate maximum ultimate capacity using rapid sand filtration technology.

Plan Adoption

It is recommended that the City adopt this preliminary design report for the City's Winchester Water Treatment Plant to guide improvements to and expansion of the plant.

G:\09\1015\401\CAD\09-1015-401-OR-FIG-ES-X.dwg FIGURE ES-1 6/24/09 08:30 (DAK)

CITY OF ROSEBURG, OR

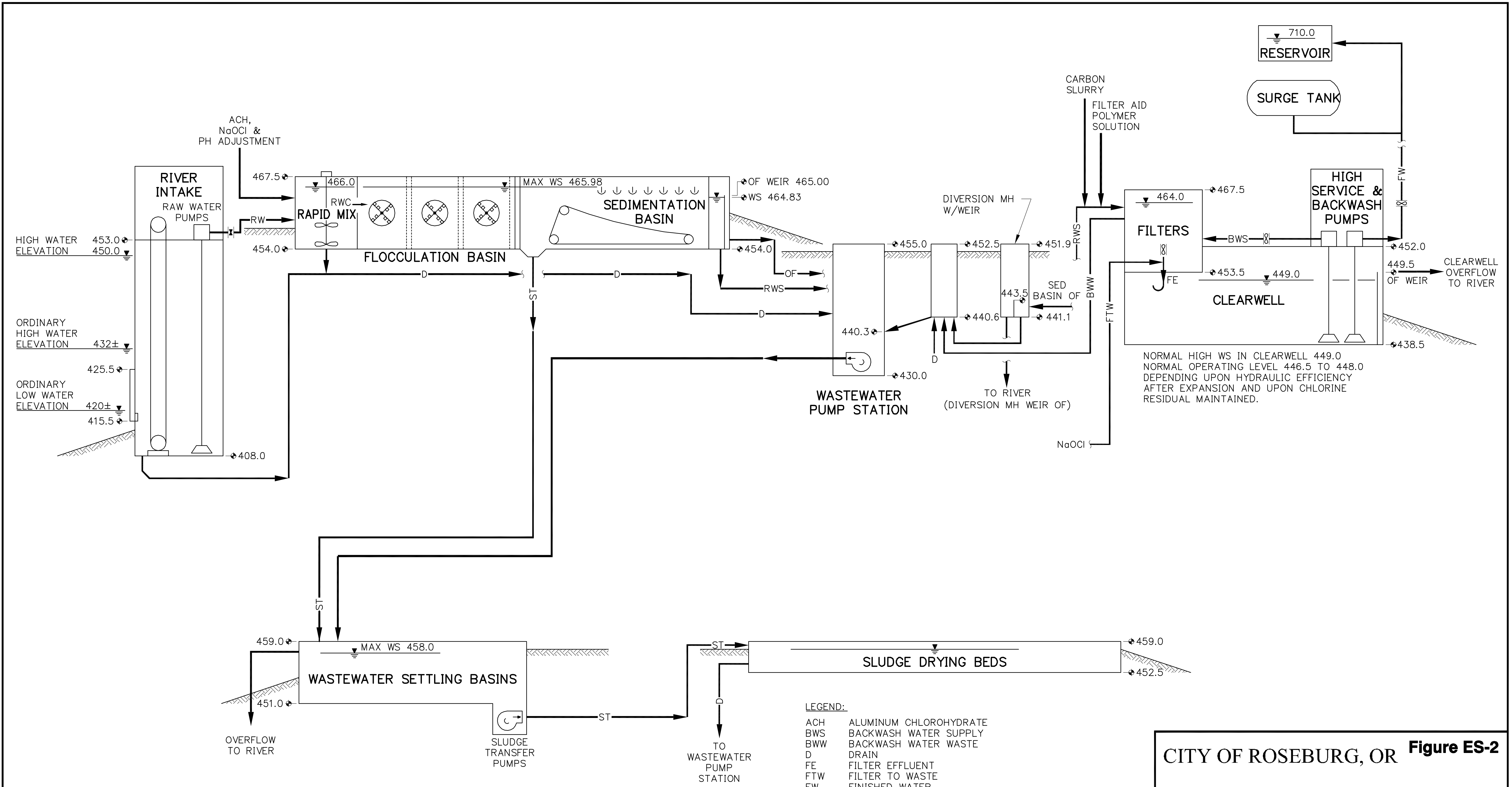
Figure ES-1

WATER TREATMENT FACILITIES PRELIMINARY DESIGN REPORT

EXISTING WINCHESTER WTP SITE PLAN

JULY 2009

MSA
Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 PHONE 503-225-9010
Portland, Oregon 97204 FAX 503-225-9022



LEGEND:

ACH	ALUMINUM CHLOROHYDRATE
BWS	BACKWASH WATER SUPPLY
BWW	BACKWASH WATER WASTE
D	DRAIN
FE	FILTER EFFLUENT
FTW	FILTER TO WASTE
FW	FINISHED WATER
NaOCl	SODIUM HYPOCHLORITE
RW	RAW WATER
RWC	RAW WATER COAGULATED
RWS	RAW WATER SETTLED
ST	SLUDGE TRANSFER
WS	WATER SURFACE
⧗	MAGNETIC FLOW METER
8	PROPELLER METER

NOTE:

1. ▨ SYMBOL REPRESENTS APPROXIMATE FINISH GRADE RELATIONSHIP.

2. BASED ON AN ULTIMATE PLANT FLOW OF 22 MGD.

CITY OF ROSEBURG, OR

Figure ES-2

WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

WINCHESTER WTP
HYDRAULIC PROFILE

JULY 2009

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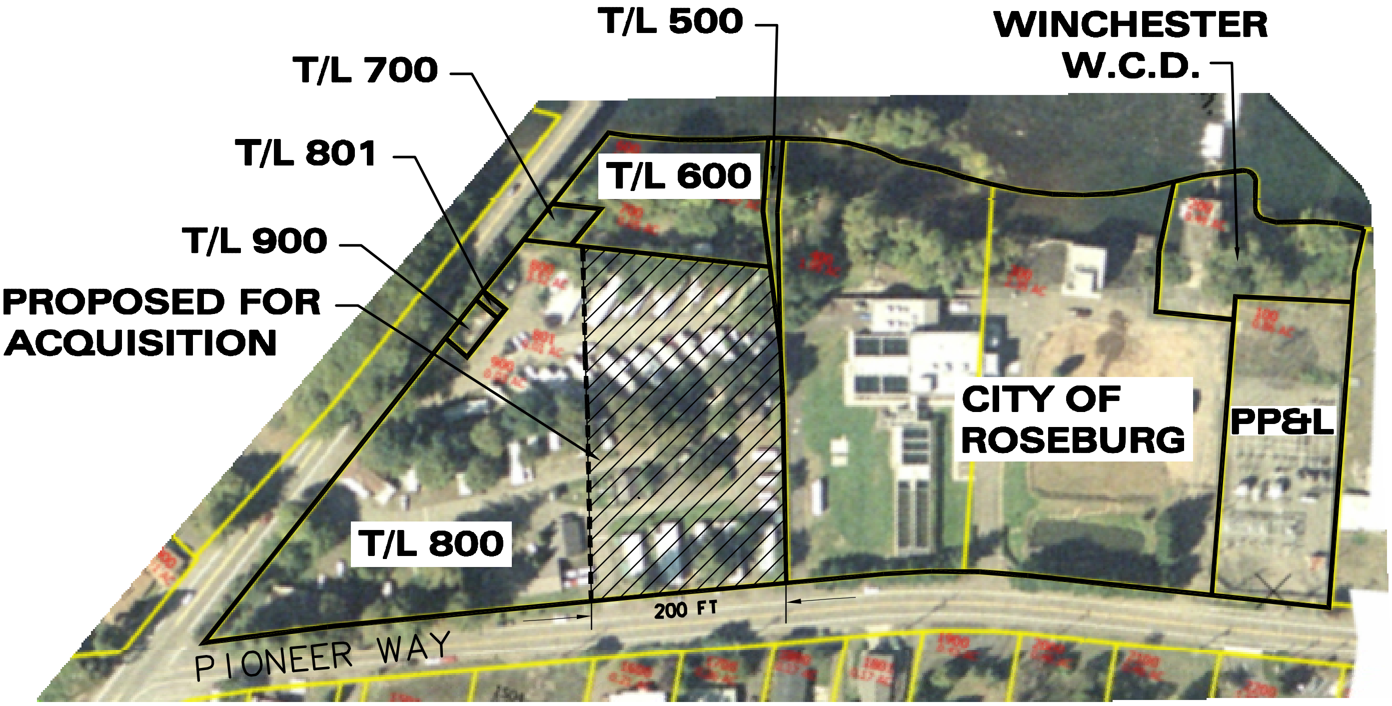
G:\09\1015\401\CAD\09-1015-401-OR-FIG-ES-X.dwg FIGURE ES-2 6/24/09 08:30 (DAK)



1 Inch = 200 Feet

CITY OF ROSEBURG, OR

Figure ES-3

WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

WINCHESTER WTP AND ADJOINING
PROPERTIES

JULY 2009

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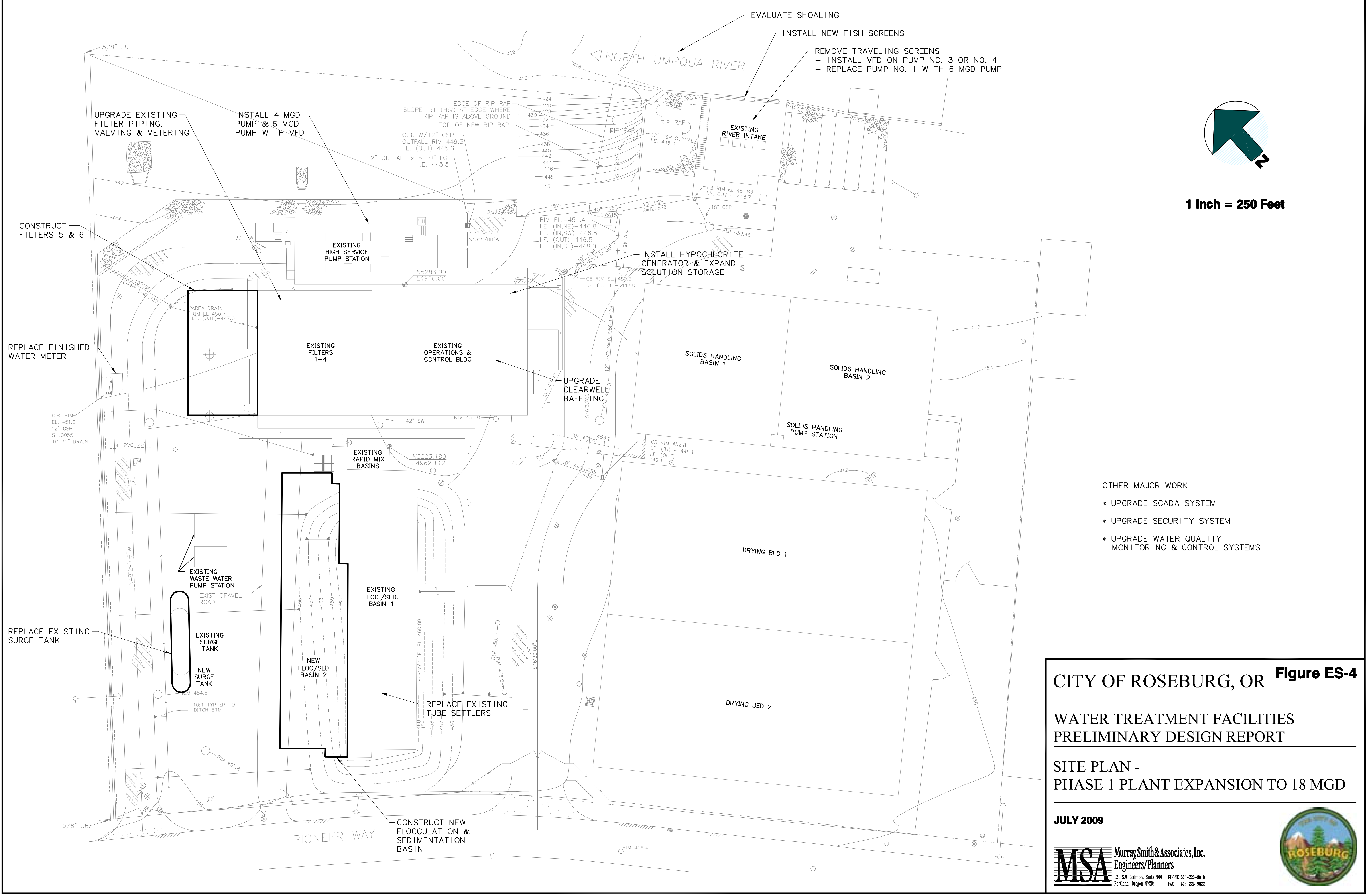
G:\09\1015\401\CAD\09-1015-401-OR-FIG ES-3.dwg ES-3 6/24/09 08:28 (DAK)



OTHER MAJOR WORK

- UPGRADE SCADA SYSTEM
- UPGRADE SECURITY SYSTEM
- UPGRADE WATER QUALITY MONITORING & CONTROL SYSTEMS

CITY OF ROSEBURG, OR

Figure ES-4

WATER TREATMENT FACILITIES PRELIMINARY DESIGN REPORT

SITE PLAN - PHASE 1 PLANT EXPANSION TO 18 MGD

JULY 2009

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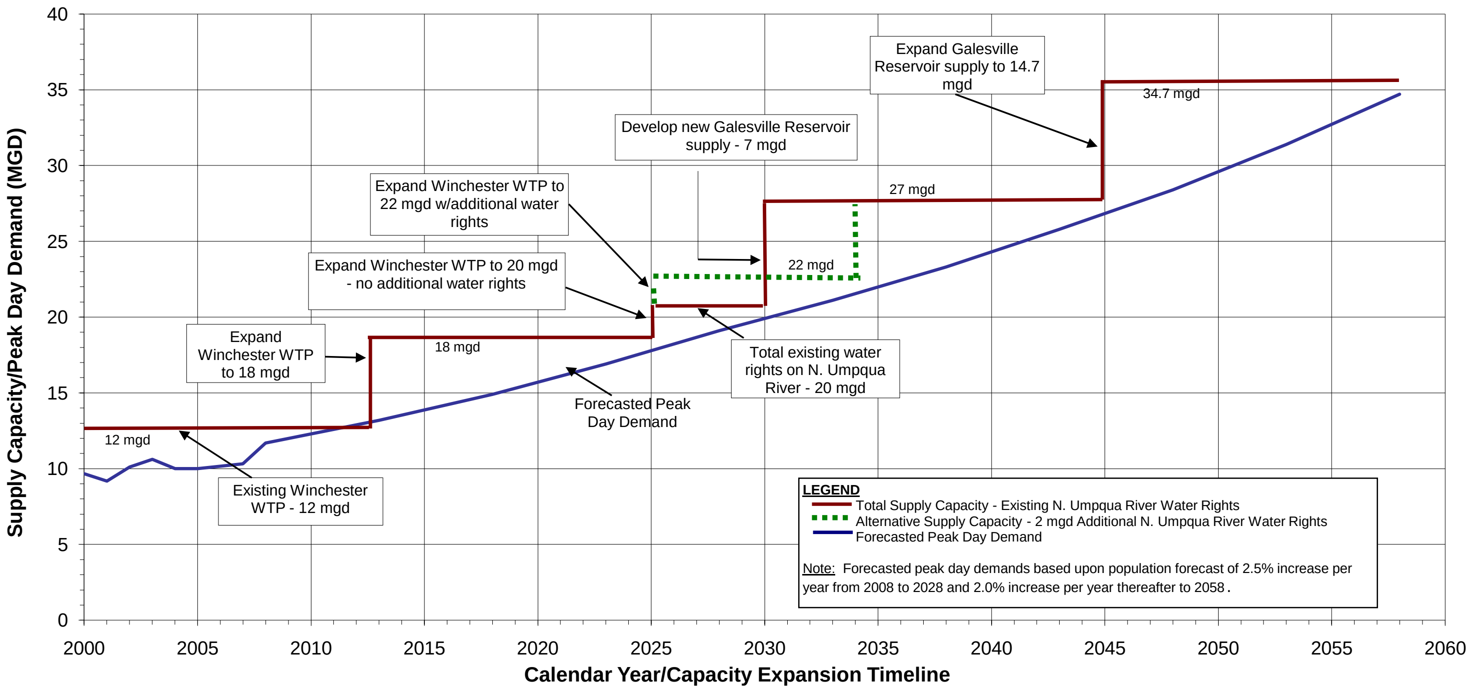
G:\09\1015\401\CAD\09-1015-401-OR-FIG-ES-X.dwg FIGURE ES-4 6/24/09 08:30 (DAK)

Figure ES-5
City of Roseburg
Water Demand and Water Supply Schedule



Note: Forecasted peak day demands based upon population forecast of 2.5% increase per year from 2008 to 2028 and 2.0% increase per year thereafter to 2058.

SECTION 1

Preface

A draft final report was submitted to the City in June, 2008. Prior to adoption of the report, the City corrected the population forecasts to match percentages used in other planning documents to ensure consistency among plans. This resulted in revised water demand forecasts. Murray, Smith & Associates, Inc. (MSA) was authorized to update the prior report to reflect any changes due to the revised water demand forecasts as well as to update certain sections of the report at the same time. Section 3 – Regulatory Overview is updated to reflect some subsequent regulatory changes, which are relatively minor, and the City's subsequent raw water testing results. Section 7 – Recommendations and Implementation Plan is updated to reflect current project costs and the revised proposed scheduling for plant improvements. An updated funding analysis and implementation plan for the Phase I plant expansion is included in the report. The Executive Summary is updated to reflect all of the report changes. The remaining sections of the report are unchanged. The revised water demand forecasts and other updating in the report have had no substantive impacts upon the conclusions and recommendations of the prior report.

Authorization

In June 2006, the firm of Murray, Smith & Associates, Inc. (MSA) was authorized by the City of Roseburg to undertake and complete this Water Treatment Facilities Pre-Design Report for the City's Winchester Water Treatment Plant.

Background of Plant and North Umpqua River Supply

The initial supply of water from the North Umpqua River is understood to have commenced in 1900. The Roseburg Water and Light Company constructed a pumping plant on the river at Winchester along with a wood-stave transmission pipeline to serve the City. In 1904, the Umpqua Water, Light and Power Company was formed as a consolidation of Roseburg Water and Light Company and the company succeeded the Roseburg Water Company. Umpqua Water and Light was purchased by A. Welch in 1906 and subsequently the system was sold to the Kendall family in 1907. During this time period, pressure filters were installed at the pumping station site. The Douglas County Light and Water Company purchased the water system in 1912.

In 1923, the California-Oregon Power Company (COPCO) purchased the water system. In 1934, the original wood-stave transmission main system was replaced with a 20-inch diameter steel pipe, portions of which are in use today. The Oregon Water Corporation (OWC) acquired the system and others in the region in June, 1950, and owned and operated the system until 1977 when the City of Roseburg purchased the system. Over the decades, the water system facilities including the original treatment plant were upgraded and expanded to meet the increasing water demands.

In the mid-1980s, the need for a new water treatment plant became apparent to the City. The existing river intake, pressure filtration plant, and related facilities at the Winchester site were at capacity, antiquated and not readily expandable, and were well beyond their useful lives. Interest rates at that time were very high, city revenues were limited, and the economic conditions in the community were depressed. Conventional financing methods for a large project to upgrade or replace the plant would have created a large debt burden on the community. City leaders, including the members of the Utility Commission and the City Council, conceived of an approach to the plant replacement project that would avoid debt financing and limit the financial burden to its citizens. The concept was to construct a new intake and treatment plant in stages with "pay-as-you-go" financing to avoid, if possible, the issuance of bonded debt.

In 1984, the City retained Tucson Myers & Associates to evaluate the concept and determine its feasibility and prepare a report. MSA supported the work as the treatment plant engineering consultant to the Myers firm. The concept of a staged replacement program was developed and conceptual level cost estimates were prepared. It was determined that the approach was feasible from an engineering and financial perspective. The report recommended funding on a "pay-as-you-go" basis using available water funds plus implementation of a surcharge on customer water bills to finance the construction over a 7-year period. The Utility Commission and the Council approved the report and proceeded with the program.

Engineering and construction extended from 1985 to 1992 on a schedule to match available funds. New facilities systematically replaced the old during four project phases: Phase 1 - River Intake; Phase 2 - High Service Pump Station; Phase 3 - Rapid Mix, Flocculation/Sedimentation; and Phase 4 - Filters and Operations Building. Water was produced during the plant replacement period through a combination of new and old facilities. The project was accomplished without duplicating any facilities and without retrofitting or modifying any completed work during the subsequent phases.

The complex design and construction process required sound, detailed forward planning. Comprehensive preliminary engineering culminated in a Preliminary Design Report (PDR), completed in 1986, which described how each new system phase would coordinate with previous work and the existing plant. Coordination between phases was critical to the project's success. Final design engineering for each project phase orchestrated the technical details required for proper construction sequencing following the guidance of the PDR. The treatment facilities operated without interruption during the construction, providing the community with a modern, highly reliable and safe supply of high-quality water. The PDR planned for the logical and economical expansion of the plant beyond the initial four project phases, which initially provided 12 million gallons per day (mgd) of capacity. The plan provided for expansion of the plant on the existing 3.5 acre site to a nominal 18 mgd treatment capacity.

The initial $8.4 million replacement program was successfully completed on time and within the original budget. Its unique approach enabled the City to afford state-of-the-art water treatment and supply technology. At project completion, the customer surcharge was removed. The “pay-as-you-go” approach saved the City an estimated $4 to $5 million in interest payments.

In 2001, the City implemented some additional improvements to the plant. An on-site sodium hypochlorite generation system was installed to replace the existing gaseous chlorine system. A fourth raw water pump was installed in the river intake and a new magnetic flow meter was installed on the raw water pipeline between the river intake and the rapid mix basins.

In 2005, the City proceeded with implementation of the upgrading of the plant’s washwater and solids handling system as provided for in the PDR. That project was completed and placed in service in the fall of 2006. Additional improvements to add capacity to the hypochlorite system were recently completed along with the installation of two new variable frequency pump drives, one on a raw water intake pump and one on a high service pump.

The plant has been producing water near its 12 mgd capacity during peak demand periods over the past several years. Significant growth has been occurring within the City’s water service area over the past several years. Planned annexations to the City are anticipated to soon increase water demands beyond the plant’s current capacity.

Key Project Issues

A number of key issues to be addressed in conjunction with the planning for the plant expansion have been identified by City staff and the consultant team. These issues are outlined below.

The Appropriateness of the Existing Treatment Processes and Equipment for Plant Expansion

The existing treatment processes and equipment have been evaluated to determine whether expansion of the plant to 18 mgd using the same technology as existing is likely to continue to produce excellent, low turbidity water with high efficiency under a variety of raw water quality conditions. This analysis is based on discussions with plant operators and a review of the historical data presented in Section 2.

Impact of Drinking Water Regulations Promulgated Since the Plant was Constructed on Selection of Technology for Plant Expansion

Drinking water quality regulations have changed since the plant was originally designed. These changes, briefly summarized here and discussed in detail in Section 3, may have a significant impact on planning for the plant expansion. Two new regulations have been recently promulgated. These are:

- Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)
- Stage 2 Disinfectant/Disinfection By-Product Rule (D/DBPR)

The LT2ESWTR requires compliance by 2010 and requires utilities to sample their source water for *Cryptosporidium* and *Giardia* for 24 consecutive months to allow determination of how "at risk" the source is from pathogen contamination. High-risk waters with higher concentrations of *Cryptosporidium* will be required to install specific removal and/or inactivation processes, such as ultraviolet (UV) light disinfection or membrane filtration.

Low-risk waters with very low concentrations of *Cryptosporidium* will be able to continue using conventional filtration and disinfection processes. The City has undertaken a formal *Cryptosporidium* sampling and testing program. The results to date indicate that the North Umpqua River has very low concentrations of *Cryptosporidium* similar to other western Cascade Range rivers and therefore significant process changes will not be required due to this rule.

The Stage 2 D/DBPR requires compliance by 2010 and will set a total trihalomethane (TTHM) limit of 80 parts per billion (ppb) and a haloacetic acid (HAA_5) limit of 60 ppb as measured at the longest residence time locations in the distribution system. The current rules allow averaging of all DBP samples taken from within the system on a running annual average basis. This rule may require changes to the disinfection process for some utilities, such as use of an alternative disinfectant or deferred addition of chlorine. Preliminary data review indicates that existing DBP concentrations are relatively low and the City appears to have little or no risk of non-compliance with the future D/DBPR.

In addition, subsequent to the design of the plant, the State of Oregon promulgated regulations pursuant to the Surface Water Treatment Rule that changed the determination of disinfection credit. Prior to that time, inactivation of pathogens through disinfection was calculated based on disinfection residuals throughout the plant. State regulations now require that inactivation be calculated solely on the basis of disinfection achieved after filtration.

The Potential to Up-Rate the Capacity of Existing Processes

The City's existing water rights on the North Umpqua River total 20 mgd and there is the possibility of obtaining additional rights from that source. The nominal rating of the plant as described in the PDR will be 18 mgd after the planned expansion with one new flocculation/sedimentation basin and two new filters. The possibility of up-rating the capacity of the plant using the current flocculation, sedimentation and gravity filtration technologies to treat the existing 20 mgd water right, and possibly even greater flows if additional water rights are obtained is evaluated in Section 4.

Alternative Treatment Technologies

Alternative technologies that could potentially improve upon the current processes or provide additional treatment capacity are considered in Section 6. Alternatives for clarification and for filtration are considered.

Alternative Clarification Technologies: The existing tube settlers in the existing basin need to be replaced. Clarification technologies for conventional treatment have advanced in recent years. As an alternative to installing new tube settlers, the installation of Lamella plate settlers is considered.

Alternative Filtration Technologies: One alternative filtration approach is to replace the existing rapid sand filtration system with membrane filtration. In membrane filtration, hollow fiber membranes are used to separate particles from the water. The membranes have small pores, on the order of 0.1 micrometers (μm) or less. The pores allow water to pass through the membrane while retaining particles larger than about 1.0 μm. The membranes are formed into hollow fibers which are bundled together longitudinally and either encased into a pressure vessel or submerged in a basin.

Pressure membranes operate with the unfiltered water pumped through the inside of the hollow fiber. Particles are retained on the inside of the hollow fiber while filtered water passes through the pores to the outside of the fiber. Submerged, or "vacuum," membranes operate with the unfiltered water on the outside of the fiber. Particles are retained on the outside of the fiber while filtered water passes through the pores to the inside of the hollow fiber under the pressure differential provided by a vacuum applied to the inside of the hollow fiber.

The existing plant could be converted to membrane technology as part of the program to expand the plant to 18 mgd. Alternatively, the initial expansion to 18 mgd could be done with rapid sand filtration and expansion beyond 18 mgd could be accomplished with membranes. For retrofitting an existing rapid sand filtration plant for membrane filtration, it is common to use submerged membranes located inside the existing filter bays. This generally results in a lower capital cost than retrofitting with pressure membranes.

Another filtration alternative is to continue using granular media filtration but with a different media design. Changing the media could increase the plant capacity at much lower cost than retrofitting the plant to operate on membrane filtration; however, the ultimate plant capacity achievable through changes in the media may be less than what membrane filtration could provide.

Coordination of Pre-Design for Plant Expansion with Long-Range Water Supply Plan

The long-range water supply plan for the City of Roseburg is being prepared concurrently with this report. The long-range water supply plan is contained in a separate document that forecasts the population and water demands for a 50-year period and develops a plan to maximize the City's water supply system on the North Umpqua River. The conclusions of long-range water supply plan regarding both the necessity and the possibility of obtaining additional water rights on the North Umpqua will guide the decision regarding the ultimate design flow that the Winchester plant should be capable of treating.

Potential Land Acquisition Requirements

The Winchester Water Treatment Plant was planned with a nominal capacity of 18 mgd on the existing site. The water rights work and demand projections in the long-range water supply plan indicate that the plant should be expanded beyond this capacity. The goal is to accommodate this added capacity within the existing site. Alternative treatment processes are considered that may reduce space requirements and/or accommodate anticipated future regulatory compliance requirements on the available space; however, the site is spatially constrained at 18 mgd, thus acquisition of additional land must also be considered.

Determine the Upgrading Requirements for Finished Water Pumping and Surge Control

The finished water pumping capacity will be expanded with the addition of two new vertical turbine pumps in spaces provided when the high service pump station was originally constructed and the replacement of one existing pump with a higher capacity pump. The basic plant electrical system was planned and constructed for the electrical load of these additional pumps. Some upgrading of the electrical system may be required. The City recently completed installation of a variable frequency drive on one of the pumps to provide for improved flow control and balancing through the plant. It may be desirable to install variable frequency drives on one or more additional high service pumps.

The existing hydropneumatic surge tank will need to be evaluated to determine if it has adequate control capability for the higher flow rates. The surge tank was designed for the pumping conditions and the transmission system that existed at the time of initial construction. A Preliminary Engineering Study for Water Transmission System, prepared in 1986, recommended specific upgrades that would achieve a transmission main capacity of 18 mgd. Most of those upgrades have been completed by the City.

A new analysis of the existing surge tank operating in conjunction with the upgraded transmission main will be needed. The results of this analysis will be governed by the characteristics of the finished water transmission system. Hydraulic modeling should be conducted during the design phase of the plant expansion using data on the existing transmission system between the plant and the terminal reservoirs to confirm the adequacy of the existing surge tank and the potential need for additional surge control.

Evaluate the River Intake

The City has expanded the pumping capacity of the river intake by installing a pump in the space that was provided when the river intake was originally constructed. The City recently completed installation of a variable frequency drive on one of the pumps to provide for improved flow control and balancing through the plant. Installation of a second variable frequency drive on one more intake pump may be advisable to increase operational reliability. The river intake screens which provide for fish protection are a concern. The existing intake configuration allows for up to 16.5 mgd of capacity while meeting the current criteria of a maximum approach velocity of 0.4 feet per second (fps). Screening improvements are not anticipated to be required by the State or other regulatory agencies until the rate of withdrawal from the river approaches 16.5 mgd.

Meet Chlorine CT Requirements at a Higher Plant Capacity

The plant clearwell volume was planned and designed to meet disinfection contact time requirements at 18 mgd nominal capacity based on regulations in effect at the time. The clearwell is baffled so as to increase its contact time efficiency. Subsequent to design of the plant, disinfection requirements became more stringent. The existing plant needs to be evaluated regarding its capability to meet CT requirements under current and expanded flow rates.

Prospective plant improvements to meet CT requirements include:

- Increasing clearwell volume
- Improving the efficiency of the existing clearwell volume with additional baffling
- Increasing disinfectant (chlorine) concentrations
- Incorporating transmission system volume outside the plant into the CT compliance strategy
- Limiting plant production rates during critical periods of low raw water temperature and low raw water pH
- Changing the disinfectant process

Develop a Cost-Effective Plan to Meet the Long-Term Needs of the City

The expansion plan for the treatment plant needs to meet the long-term water demand requirements of the City's system in the most cost-effective manner. The expansion plan should address the key issues and result in a capital improvements plan that addresses project scheduling, cost, constructability and contract efficiency issues.

Water Demand Forecasts

Population and water demand forecasts for the City have been prepared as part of the companion document, the Long-Range Water Supply Plan, City of Roseburg, July 2009. Appendix A of this report is Figure 7-1, Water Demand and Water Supply Schedule, from that long-range plan. The figure illustrates the recommended water supply plan for the City to the year 2058 including the expansion of the Winchester WTP.

Other Report

At the request of the City, a report entitled “Treatment Plant Expansion, Funding Analysis and Implementation Plan, City of Roseburg, May 2009 Update” as prepared by Integrated Consulting Services, Inc., is included in Appendix E. This report develops an implementation plan for expanding the treatment facilities, identifying major milestones and a tentative schedule, and evaluates funding alternatives and prepares a preliminary rate analysis including financial projections. This report was prepared under a separate agreement between Integrated Consulting Services, Inc. and the City.

SECTION 2
HISTORICAL PLANT PERFORMANCE

General

Historic water quality and operating data for the City of Roseburg's Winchester 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. This performance evaluation is used, in part, to determine the process selection for the plant expansion. Section 3 addresses compliance with drinking water regulations based in part on data presented herein.

All available information relevant to the plant's current condition and performance was reviewed for this evaluation. Plant performance data dating back to plant startup in 1992 was available. However, this performance review focused on more recent data, with the exception of plant production which was analyzed since 1992. Three years of data and information, from January 2003 to December 2005, were reviewed including selected raw, finished and distribution system water quality parameters, chemical usage data, sedimentation basin performance, and overall filter performance indicators. Discussions with plant operators were used to supplement and verify this information.

Plant Flows

The Winchester WTP measures and records raw and finished water flow rates through the plant on a daily basis. Backwash flow rates are measured during each backwash cycle but the flow rate is not recorded. Backwash volumes are totalized and recorded on a monthly basis.

Raw water flow is measured using a magnetic flow meter located on the influent line prior to chemical addition. The original raw water propeller flow meter was replaced with a magnetic flow meter in 2001. Finished water flow rates are measured using the original propeller flow meter located on the WTP effluent line just downstream of the high service pump station (HSPS). Backwash flow is measured in the backwash supply line using a propeller flow meter. Sludge flow rate is not measured. Filter-to-waste (FTW) flows are discharged upstream of the individual filter effluent flow meters; therefore, historically these flows have not been measured or recorded. All four filters are online during a FTW cycle so the rate through the filter in FTW can be determined by the difference between the plant flow and the measured flows through the three remaining filters. Treatment plant staff can estimate the volume of FTW from that estimated flow and the FTW duration, which is typically 10 minutes.

When the new raw water magnetic flow meter was installed, it was determined by the plant operators that the finished water flow meter was under-reading. As part of the companion long-range water supply plan, a review of the historical plant flow data was accomplished. It was determined that the City's reported finished water flows are low by approximately 6 to 7 percent due to inaccuracies introduced by the propeller meter. Comparison of raw water flows and major in-plant water uses, including backwash flows, to the reported finished water flows, highlighted these differences. For the purposes of this report, the historical reported finished water flows have been adjusted to account for this discrepancy.

Plant Production

Table 2-1 presents the historic finished water flows from 1992 to 2005 including annual average flow, average peak season and off-peak season flows, minimum and maximum monthly average flows, and maximum day flows. Figure 2-1 graphically illustrates the average annual, peak month and peak day flows.

The City has been experiencing increasing water demands over the past 10 years. Average day production increased approximately 14 percent per from 1995 to 2005 (from 4.75 mgd in 1995 to 5.4 mgd in 2003). A maximum peak day flow from the Winchester WTP of 11.6 mgd was observed on July 28, 2003. The highest average maximum monthly flow of 9.4 mgd was also observed in July 2003. Increasing demands can be attributed to steady growth and resultant increased water demands in the service area. As discussed in Section 1 of this report, the City is anticipating increased growth which will result in increasing water demands. These increasing demands result in the need to expand the Winchester WTP in the near future since its current rated capacity is 12.0 mgd.

The flow data presented in Table 2-1 were used to develop peaking factors that are useful in water supply planning efforts. The primary peaking factor is the ratio of the maximum day demand (MDD) to the annual average daily demand (ADD). This value ranged from 1.8 to 2.3 between 1992 and 2005. Another important peaking factor is the ratio of the maximum month average daily demand to the annual average daily demand. This value ranged from 1.5 to 1.8 over the past 14 years. These values are consistent with those used for demand forecasting in the City’s Long-Range Water Supply Plan. Recent studies indicate that maximum day peaking factors for systems in the Pacific Northwest typically vary from about 2.0 to 2.5. Thus the peaking factors for the City system determined from the City’s production data are consistent with these regional data.

TABLE 2-1
SUMMARY OF WINCHESTER WTP PRODUCTION (1992 – 2005)

Year	Annual Average Daily	Peak Season[1] Average Daily	Off Season[2] Average Daily	Minimum Month Average Daily		Maximum Month Average Daily			Maximum Day	
				Month	Value	Month	Value	Peaking Factor	Value	Peaking Factor
1992	5.1					AUG	7.95	1.6	9.4	1.8
1993	4.3					JUL	6.3	1.5	8.2	1.9
1994	4.9					JUL	8.3	1.7	10.3	2.1
1995	4.8					AUG	7.5	1.6	8.9	1.9
1996	4.9					JUL	8.1	1.7	9.4	1.9
1997	4.8					AUG	7.6	1.6	9.55	2.0
1998	4.9					AUG	8.5	1.7	11.5	2.3
1999	4.9					JUL	8.3	1.7	9.7	2.0
2000	5.3					JUL	8.3	1.6	9.7	1.8
2001	5.2					JUL	8.0	1.6	9.2	1.8
2002	5.3					JUL	8.8	1.7	10.1	1.9
2003	5.4	8.2	4.0	FEB	3.5	JUL	9.4	1.7	11.6	2.1
2004	5.3	7.3	4.2	FEB	3.8	JUL	8.7	1.6	10.0	1.9
2005	5.1	7.4	4.0	FEB	3.7	AUG	9.2	1.8	10.0	2.0

[1] Peak Season is June through September
[2] Off Season is October through May

Raw Water Quality

General

Five raw water quality parameters - turbidity, temperature, pH, alkalinity and organic content - were analyzed. These parameters are typically of most importance when evaluating a treatment plant's overall performance.

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, sedimentation, 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 North Umpqua 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. Figure 2-2 presents the average daily river flow and raw water turbidity, as well as the observed daily precipitation between January 2003 and December 2005. Days with no measurable precipitation are not shown in the data presented in Figure 2-2. The lowest turbidity periods occur during the warmer, drier months and the highest turbidity periods occur during the wet weather months**.**

Average turbidities are generally less than 5 NTU from June to September; minimum turbidities have been as low as 1.0 NTU during these months. Between October and May, average monthly turbidities typically range between 5 and 25 NTU. During the past 3 years, raw water turbidities approaching 500 NTU have been observed for brief periods. Occasional raw water turbidities approaching 1,000 NTU were recorded during the winters of 1995 through 1997 according to plant staff, which is when extreme precipitation occurred throughout the Pacific Northwest. May 2005 also experienced relatively high turbidities due to extended spring rains. The plant has performed well during all of these elevated turbidity events.

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 average daily temperature of the raw water entering the WTP varies by season, as shown in Figure 2-3. During the 3-year period of record considered for this evaluation, wintertime low average temperatures were approximately 50^{o}F (10^{o}C) and summertime high average temperatures were approximately 72^{o}F (22^{o}C). The lowest observed temperature was 44^{o}F (7^{o}C) in December 2005. The raw water temperature can occasionally drop below 5^{o}C during extreme cold weather events. The highest observed temperature was 77^{o}F (25^{o}C), measured in July 2003 and July 2005. The water temperature was always greater than 20^{o}C during July and August when peak water demands and maximum plant production occurs. The water temperature was usually greater than 5^{o}C during the winter months when minimum demands occur**.**

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 normally 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 perhaps some sacrifice in coagulation and disinfection performance. The addition of treatment chemicals alters the pH; the coagulant slightly depresses the pH and the hypochlorite solution slightly raises the pH.

The pH of the raw water from the North Umpqua River typically varies between 7.1 and 8.5 throughout the year, with average values between 7.5 and 8.0. Historically, pH peaks during the summer, probably corresponding to algal activity in the river. Historic minimums occur in the winter months, presumably due to heavy rainfall and snowmelt events when alkalinity is also depressed. The lowest observed raw water pH was 6.8 in mid-December 2002 and in mid-November 2004. The highest observed pH was 8.3 in mid-July 2004 and mid-July 2005.

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 coagulation and for improved pH stability in the distribution system. Alkalinity can also impact removal requirements for total organic carbon (TOC), depending on raw water organic concentrations.

Alkalinity is not measured frequently at the Winchester WTP; however, data are collected monthly or quarterly. Raw water alkalinity at the plant apparently ranges around 20 mg/L as $CaCO_3$ in the winter. Raw water alkalinity has not been measured with enough frequency to establish seasonal alkalinity trends; however, it is expected that alkalinity will decrease in the winter and early spring (corresponding to the rainy season and early snowmelt) and will increase in the summer, similar to pH trends**.**

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 decay, disinfection by-product (DBP) formation, and tastes and odors. 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 (coagulants 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 Winchester WTP has been monitoring TOC concentrations in the raw and finished water since April 2002. Monthly TOC sampling was performed from 2002 to 2005. The data from the period 2003 through 2005 for raw water TOC, finished water TOC and percent TOC removal through the plant are presented in Figure 2-4. The data suggest that the TOC concentrations in the raw water are comparable to other Pacific Northwest surface water supplies, typically ranging between 0.7 to 4.5. The highest TOC concentrations coincide with elevated turbidity in the river and are probably particulate-based. The average monthly raw water TOC concentration from 2003 to 2005 was 1.45, which is less than the 2.0 mg/L "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.

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 is a relatively inexpensive, simple and accurate alternative to lab analyses of TOC.

Taste and Odor

According to plant staff, the North Umpqua River supply has experienced some short-duration and infrequent taste and odor (T&O) events, usually in August and early September. These events have resulted in a few customer complaints. In some Pacific Northwest surface water supplies, objectionable tastes and odors can occur during the warmer summer months (August and/or September), oftentimes because of increased algal activity which results from warmer water temperatures. Geosmin and methylisoborneal (MIB) are commonly detected in surface water supplies during these events. Both of these organic compounds are naturally occurring resulting from algae metabolism and impart earthy and musty tastes and odors. Both of these compounds can cause objectionable odors at very low concentrations (0.005 ug/L or 5 parts per trillion). The City has not sampled for MIB and geosmin in the finished water nor has it monitored for algae in the raw water, but it is presumed that one or both of these compounds are the primary reason for the infrequent (T&O) problems during the late summer.

Chemical Usage

General

Chemical usage at the Winchester WTP was analyzed to determine any seasonal trends that may offer insight into the overall treatment process performance. The three major chemicals currently used at the plant are aluminum chlorhydrate (ACH) as the primary coagulant, filter aid polymer, and liquid sodium hypochlorite generated on-site. The plant also has chemical systems for pH adjustment (lime and/or soda ash) and powdered activated carbon (PAC), both of which are not currently used.

ACH is used as the primary coagulant for suspended solids and TOC removal. The filter aid polymer is used to condition the water entering the filters for improved filter performance. Sodium hypochlorite is added to the raw water and finished water as a disinfectant.

Primary Coagulant

The primary coagulant, ACH, has been used successfully at the plant since 1999 when the use of alum as the primary coagulant was discontinued. Plant trial tests determined that ACH produced more reliable treatment results over the range of raw water quality conditions and also provided the opportunity to reduce operating costs. This alternative coagulant performs well over a wider pH range than alum, has lower required dosages, and also produces less sludge. It does not significantly depress the pH compared to alum, so the plant has been able to stop practicing post-filter pH adjustment with lime or soda ash.

ACH is delivered and stored in a bulk tank as a 50 percent solution (by active product weight) and fed via a metering pump to the raw water at the rapid mix basin. The delivered solution has a bulk density of 11.1 lbs/gallon and an “active product” density of 5.5 lbs/gallon. The addition of ACH 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 ACH feed is continuous and uses carrier water. The coagulant dose is manually adjusted (from the SCADA control system) based on raw water turbidities, previous experience and results from jar tests. The ACH metering pump is automatically “flow-paced” to adjust to changing plant flows. A streaming current monitor (SCM) is also used to determine coagulant dosage.

Figure 2-5 shows the annual trends in coagulant usage between January 2003 and December 2005. The required ACH dose varies throughout the year; typical winter alum doses average between 4 and 6 mg/L (as active product) while spring and summer alum doses average between 2 and 4 mg/L. The highest coagulant doses coincide with high turbidity events; the highest reported coagulant dose of approximately 8 mg/L occurred in November 2005. The minimum daily coagulant dose has been as low as 2.0 mg/L during low turbidity, warm water.

As stated previously, the use of ACH since 1999 has resulted in significantly lower doses throughout the year compared to the previous use of alum. This has decreased plant operating costs, has decreased sludge production, and may also improve sludge dewatering properties compared to the prior use of alum.

Polymer (Filter Aid)

The Winchester WTP currently uses a nonionic polymer, Cytec N-300LWM, as a filter aid. The dry polymer is mixed and aged with water, then fed via a metering pump and carrier water to the settled water pipe approximately 32 feet upstream of the filter influent channel. 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 Winchester WTP is important. When introduced to the settled water, the polymer helps make the 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 shorter due to premature breakthrough (i.e. the filters would have to be backwashed more frequently).

As previously discussed, aluminum coagulant flocs are known to be fairly weak in terms of their 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 other plant experiences, although this is not always the case.

Figure 2-6 presents the historic average daily filter aid polymer dosages from January 2003 through December 2005. 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. During the 3-year period, the polymer dose ranged from as low as 0.02 mg/L to as high as 0.074 mg/L with typical doses in the range of 0.03 mg/L to 0.06 mg/L.

Sodium Hypochlorite

When the plant was originally built, disinfection was provided by chlorine gas. In 2000, the City converted to a dilute liquid sodium hypochlorite solution generated on-site. The City reports that customer complaints regarding chlorinous taste and odor diminished after the conversion from gas.

The hypochlorite solution, approximately 0.44 percent, is stored in two 500-gallon polyethylene tanks located within the chlorine generator room. An 8,225 gallon insulated, cross-linked polyethylene tank stores salt and brine solution. The salt is delivered in bulk truckloads as required. The on-site hypochlorite generation system was installed in 2001 to replace the original gas chlorine injection system. The City added another generator in 2007 to increase chlorination capacity. Further information about the on-site hypochlorite generation system is presented in Section 4.

Hypochlorite is added to the raw water ("pre-chlorination") to assist in coagulation, control of biological growth through the sedimentation basins and filters, 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. The City does not need to "boost" the chlorine once the water leaves the WTP.

The operators dose pre-chlorine at 0.9 to 1.1 mg/L to maintain a low chlorine residual within the flocculation/sedimentation basin. The operators dose post-chlorine at 0.8 to 1.3 mg/L to achieve a target chlorine residual of approximately 0.80 mg/L leaving the high service pump station. The doses vary seasonally depending on water quality and plant flow conditions. Figure 2-7 presents the daily minimum free chlorine residual leaving the plant from 2003 to 2005. The measured and target residuals are fairly consistent. The City does not record the daily settled water chlorine residual.

The maximum chlorine usage at the plant of approximately 180 lbs/day, including both pre-chlorine and post-chlorine, occurred in July 2003 during the historical peak day flow of 11.5 mgd. This corresponds to a total chlorine dose for pre- and post-chlorination of 1.8 mg/L. According to the manufacturer, the on-site sodium hypochlorite generation process requires approximately 15 gallons of softened water, 3.5 pounds of salt (NaCl) and 2.5 kW-hr of electricity to produce 1.0 pound of available chlorine, in an approximate 0.44 percent NaOCl solution. The plant operators report that the brine tank receives two salt deliveries per year, each totaling approximately 34 tons of salt. The average daily production for 2005 was 5.1 mgd (see Table 2-1). If the on-site generator actually uses 3.5 pounds of salt to produce 1.0 pound of free chlorine, then the average chlorine dose for 2005 would calculate as 2.5 mg/L. The total reported chlorine dose varies from about 1.7 to 2.4 mg/L.

Using an average chlorine dose of 2.0 mg/L, the total chlorine added in 2005 would be 31,100 pounds. This indicates that the on-site generator uses about 4.4 pounds of salt for each pound of free chlorine produced. Given that the water softener also consumes salt, there does not appear to be excessive salt consumption in the production of the hypochlorite. Therefore, it appears reasonable to conclude that the on-site generation system, including the water softener, uses between 3.5 and 4.4 pounds of salt per pound of free chlorine produced and that the average total chlorine dose is between 2.0 and 2.5 mg/L.

Additional Chemicals

As mentioned above, the Winchester WTP has a system to add pH adjustment chemicals (lime or soda ash). The pH adjustment chemical system has not been used since the plant stopped using alum as the primary coagulant in 1999. Based on the data from January 2003 through December 2005, the finished water pH in the winter occasionally drops below 7.5 but normally stays at or above 7.2. During December 2003 and most of January 2004, the finished water pH was below 7.5; however, the pH only fell below 7.2 for one day during this period. The regulatory implications of low pH are discussed in Section 3. Since the City converted from alum to ACH and from chlorine gas to dilute hypochlorite generated on site, it has reported no finished water pH below 7.0.

The plant also has a chemical system to add powdered activated carbon (PAC). PAC is a dry chemical used in many treatment plants for control of objectionable tastes and odors (T&O), usually on a seasonal or temporary basis. The PAC system at the Winchester WTP has never been used since the plant staff has never had to respond to any severe T&O episodes.

PAC in high doses (greater than 20 mg/L) has the ability to remove MIB and geosmin via adsorption. At low doses, it has marginal benefit for MIB and geosmin removal, but can improve general T&O if needed or desired. The equipment at the Winchester WTP is rated for feed rates of 0.01 to 0.6 cubic feet per hour. Depending upon the density of the PAC, this equates to a maximum rate of 400 to 490 ppd. At the rated plant capacity of 12 mgd, the maximum PAC dose would be 4.0 to 4.9 mg/L; therefore, the existing equipment can feed PAC at rates capable of improving general tastes and odors but can only provide marginal removal of MIB and geosmin.

Plant Performance Data

General

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 flocculation/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.

Coagulation Performance

The North Umpqua River water quality presents some treatment challenges at the Winchester WTP resulting from: diurnal variations of approximately 0.2 pH units; wide swings in pH seasonally; seasonal variations in turbidity, temperature, and color; and occasional taste and odor events. With the exception of taste and odor, this variable raw water quality can significantly impact coagulation performance at the plant. Prior to 1999, these challenges were successfully met using a relatively high dosage of alum combined with post-filter pH adjustment using soda ash.

As discussed above, the WTP switched from alum to ACH as the primary coagulant and this has proven to be very successful for the City. Coagulant doses have been reduced and stabilized, resulting in lower solids production and lower and more consistent settled water turbidities, which in turn have resulted in stable and consistent filter performance. Also, since ACH does not significantly depress the pH compared to alum, the need for post-filter pH adjustment has been eliminated.

Pretreatment Performance

The Winchester WTP relies on a single flocculation/sedimentation basin that was constructed in 1992. A mechanical rapid mixer is also installed at the front end of the flocculation/sedimentation basin. The plant’s site plan allows for a second, parallel basin to be added in the future for expanded capacity while continuing to use the upstream rapid mixer for both basins.

Flocculation consists of a three-stage basin (38.2 feet long by 27.5 feet wide) with baffles separating the three individual horizontal flocculators. Each flocculator is driven by an independent motor and gearing which allows a wide range of flocculation speeds for variable mixing intensity. The nominal flocculation time at 12 mgd is approximately 11.2 minutes. Flocculated water enters the sedimentation basin via a wooden diffuser wall that allows full-depth flow. Maximum water depth is approximately 12 feet.

The sedimentation basin is approximately 85.7 feet long by 32 feet wide with a maximum 12 foot water depth. The latter 60 feet of the sedimentation basin (67 percent of total basin surface area) contains plastic tube settlers (1.75 foot nominal depth) immediately below the effluent launders. The nominal sedimentation time at 12 mgd is approximately 29.5 minutes. The nominal surface loading rate of the area covered by the tube settlers (1,600 sf) is 5.2 gpm/sf. Settled water flows from the launders to a central collector channel to the settled water channel at the end of the basin, and then to the filter inlet channel via a 42-inch diameter settled water pipeline.

Coagulant, chlorine and pH adjustment chemical (if needed) are added at or immediately upstream of the rapid mixer. Filter aid polymer is injected into the 42-inch diameter settled water pipeline approximately 32 feet before that pipeline empties into the filter inlet channel. The PAC injection point, which has never been used thus far, is at the same location as the filter aid polymer injection point.

The sludge collected within the sedimentation basin is removed automatically via a chain-and-flight collector system and discharged to the recently-completed washwater/sludge system. The system operates via a timer, with the frequency of cleanings and sludge discharge adjusted seasonally depending on the solids production rate. Since the WTP only has a single basin currently, the ability to take the basin off-line for inspections and maintenance are extremely limited. Having a second flocculation/sedimentation basin will improve the reliability and maintenance of the pretreatment system.

The plant does not have an on-line turbidimeter sampling water from the 42-inch diameter settled water pipeline; therefore, settled water turbidity measurements are made by grab sample and are recorded daily. Figure 2-8 presents the daily settled water turbidity between March 2001 and December 2003. Figure 2-9 presents the monthly average settled water turbidity for that same period. The figures show that the pretreatment system consistently produces low-turbidity settled water. Daily settled water turbidities during the summer-time typically range from about 0.5 to 0.8 NTU. Even when operating at peak day flows of 9 to 11.6 mgd, the settled water turbidities have remained less than 1.0 NTU. The lowest daily settled water turbidity values are measured in the fall, typically in the range of 0.3 to 0.6 NTU. Higher settled water turbidities occur during the winter and spring, during high raw water turbidity events. Average monthly settled water turbidity ranges from 1.0 to 2.0 NTU for the months of December through April and from 0.50 to 1.0 for the months of May through November.

A well-designed and well-operated conventional water treatment plant should be able to consistently achieve settled water turbidities less than 2.0 NTU; the Winchester WTP has been able to meet this goal with its single flocculation/sedimentation basin at flows up to 11.5 mgd. Figure 2-10 presents a probability distribution of the settled water turbidities in 2001-2003. For that period, 96 percent of the daily settled water turbidities were below 2.0 NTU. This evaluation demonstrates that the plant's pretreatment system has been able to successfully treat the North Umpqua River supply under a wide range of water quality and flow conditions. This excellent performance has provided the filters with a stable, low-turbidity feedwater which has resulted in excellent filter performance, as discussed below. This performance is expected to continue, and perhaps improve, with the addition of a second basin to expand the plant's capacity.

Filter Performance

The plant has four mixed-media gravity filters which were constructed in 1992. Each filter has two bays, each 11' x 19' (209 sf per bay, 418 sf per filter) for a total filtering area of 1,672 sf. The nominal maximum filtration rate at 12 mgd with all 4 filters in service is 5.0 gpm/sf. When one filter is out of service for backwashing, the other three on-line filters continue to process the entire plant flow, meaning that the filtration rate increases by 33 percent during a filter backwash, for a maximum filtration rate of 6.65 gpm/sf with one filter out of service.

Each filter was designed to hold a 30-inch tri-media configuration with the following specifications:

- Top: 16-1/2 inches of 0.9 to 1.2 mm effective size anthracite
- Intermediate: 9 inches of 0.45 to 0.55 mm effective size sand
- Bottom: 4-1/2 inches of 0.25 to 0.30 mm effective size garnet
- Support: 12 inches of graded gravel, including 6 different sizes

This media design results in an "L/d ratio" (depth (L) to diameter (d)) of approximately 1,248 (=375 [for 16.5" of 1.05mm anthracite] + 457 [for 9" of 0.50mm sand] + 416 [for 4.5" of 0.275mm garnet]). This dimensionless parameter provides a basis of comparing differing media types and sizes based on the depth and average diameter of the media. Filter media with L/d ratios in excess of 1,000 are usually capable of performing well under low-to-moderate filtration rate and low solids loading conditions. Deeper media with higher L/d ratios are usually required for higher filtration rates and/or higher solids loading conditions.

The lips of the 18-inch deep filter washwater troughs are approximately 4.0 feet above the top of the media. The water submergence over the top of the media is typically 5 to 6 feet. The filters were originally constructed with Leopold Type S underdrains that are capable of utilizing air plus water backwash. These underdrains require the use of support gravel as the current version of "gravel-less" underdrains was not developed until after the filter construction was completed. Approximately five years after initial plant startup, the filter underdrain and support gravel system in all four filters was replaced. This re-build was completed in 1997 and there have been no changes to the filter system since then.

The filters are operated by rate of flow control. Butterfly valves on individual filter effluent pipes modulate to maintain a constant filter rate and to maintain a constant water level in the filter influent channel.

Backwash water is provided by two constant-speed 50 hp pumps, one service and one standby. Each of the two filter bays is backwashed separately and sequentially after a filter is taken out of service. A master backwash control valve (in conjunction with a propeller flow meter) modulates to achieve the desired backwash rate. The backwash rate is adjusted seasonally to account for water temperature differences. The maximum backwash flow is approximately 3,890 gpm which results in a maximum backwash rate of 18.6 gpm/sf.

The filters use a single constant-speed 60 hp air scour blower to provide auxiliary air for cleaning the media. The nominal capacity of the blower is 1,050 standard cubic feet per minute (scfm) which results in an air wash rate of 5.0 standard cubic feet per minute per square foot (scfm/sf). Air flow modulating capability could be improved by installation of a modulating valve on the air waste vent. There is currently no back-up blower.

Each filter is currently backwashed once per day unless terminal headloss or turbidity breakthrough is observed prior to the end of the 24 hour run time. Terminal headloss is 7.0 feet of head and turbidity breakthrough occurs when turbidity exceeds 0.15 NTU. The filter backwash program includes: crust disruption with water only at the low rate; air scour alone; concurrent air and water at low water rate; high rate water only; and a water ramp-down to reclassify the media. General durations for each step are presented below. These durations were in effect during site visits conducted on June 29, 2006 and on September 14, 2006. Actual durations may vary seasonally or between filters.

- 0 – 7 seconds – water only at low rate to disrupt the crust on the media
- 7 – 97 seconds – air scour alone with water level ±6-inches inches above the media
- 97 – 107 seconds – concurrent backwash at low water rate
- 107 – 227 seconds – backwash "ramp-up" period to high rate BW flow
- 227 – 677 seconds – high rate backwash flow
- 677 – 737 seconds – settling rate backwash ramp down
- close backwash waste valve and refill the filter with filtered backwash water

The maximum backwash rate is varied seasonally to account for temperature and viscosity effects to achieve adequate bed expansion. The total backwash volume used per filter during colder water periods is about 64,000 gallons, which is 153 gallons per square foot (gal/sf) of filter area. The total backwash volume used per filter during warmer water periods is about 76,000 gallons (182 gal/sf). This represents a seasonal difference in rates of 19 percent.

As a rule-of-thumb, the backwash rate should be varied by 2 percent for every 1ºC change in water temperature above or below 20ºC (68ºF). With normal winter water temperatures in the range of 48ºF (9ºC) and normal summer water temperatures in the range of 70ºF (21ºC), this represents an approximate 24 percent difference in optimum backwash rates seasonally.

Filter-to-waste is employed after each backwash to ensure the filter has been adequately rinsed, and typically lasts 5 to 10 minutes. The 8-inch filter-to-waste pipes limit the maximum filter flow during this period to approximately 2 mgd. The common filter effluent pipes from each filter to the under-filter clearwell are 18-inch.

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.

Turbidity

Each filter at the Winchester 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 process monitoring and for regulatory reporting.

Figure 2-11 presents a summary of average daily finished water turbidities between January 2003 and December 2005, 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 0.15 NTU, and is usually less than 0.05 NTU. Figure 2-12 presents a statistical summary of average daily finished water turbidities during the 3 year period. From the figure, the plant has produced 0.04 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, especially since ACH has been used as the primary coagulant since 1999.

Historical filtered water turbidities from individual filters were reviewed, but cannot be easily summarized in a figure. In general, each of the filters has performed well in terms of overall particulate removal and there do not appear to be any "problem" filters. All filters appear to be producing filtered water turbidities less than 0.10 NTU for at least 95 percent of the time.

Filter Production Efficiencies

To evaluate overall plant efficiency, a relationship between a filter's production, run lengths and backwash volume requirements is required. The concept of unit filter run volume (UFRV) is 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 area, typically expressed in gal/sf, and also referred to as the unit backwash volume (UBWV)

The following relationship can be developed for these parameters as follows:

$$R_e = R_d \text{ x } [(\text{UFRV} - \text{UBWV})/\text{UFRV}]$$

Figure 2-13 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 determining the volume of backwash water used from a reliable flow meter (or by multiplying the backwash flow rate (gpm) by the duration of backwash (min)) and dividing by the total filter surface area. For reference, the current UBWV for the filters ranges from 153 gal/sf to 182 gal/sf as discussed above.

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 percent (with UBWV = 150 gal/sf). As a result, treatment plants 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, it is recommended that filters be designed 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 (which results in a production efficiency of 98.5 percent with a UBWV = 150 gal/sf), there is little increase in production efficiency, so major efforts are not usually taken to achieve very high UFRVs. Also, most treatment plants 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 at many plants is set for 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 (1,440 minutes) at a filtration rate of 5.0 gpm/sf produces a UFRV of 7,200 gal/sf. Table 2-2 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 4 filters on-line and with one of the 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, which is considered to be the minimum desirable filter production efficiency.

At the current rated maximum plant capacity of 12 mgd, the filters should operate for a minimum of 17 hours between backwashes to meet the 5,000 gal/sf UFRV criteria. During times when the plant is operating at its typical peak season production rate (7.5 to 8.0 mgd), the filters should operate for a minimum of 24 to 28 hours between backwashes. During the low demand period of the year when the plant is operating at 4.0 to 5.0 mgd, the filters should operate for a minimum of 40 to 48 hours between backwashes. It should be noted that the filtration rates required to produce flows in excess of 12 mgd are relatively high for the tri-media installed in each of the filters. Higher filtration rates will result in high incremental head loss and short filter runs.

TABLE 2-2
MINIMUM FILTER RUN LENGTH TO ACHIEVE 5,000 GAL/SF UFRV

Filtration Rate (gpm/sf)	Average WTP Flow with all 4 filters on-line (mgd)	Average WTP Flow with one filter off-line (mgd)	Minimum Filter Run Length to Achieve UFRV = 5,000 gal/sf (hours)
1.0	2.4	1.8	83.2
2.0	4.8	3.6	41.6
3.0	7.2	5.4	27.8
4.0	9.6	7.2	20.8
5.0	12.0	9.0	16.7
6.0	14.4	10.8	13.9
7.0	16.8	12.6	11.9

Plant operating records between January 2003 and December 2005 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 the monthly raw water flows and monthly backwash water usage. Backwash volumes were determined from the backwash flow meter recordings as summarized by plant staff. Figure 2-14 presents monthly average backwash volumes as well as the monthly filter production efficiencies. Also shown on the figure is the 97 percent production efficiency target for a UFRV of 5,000 gal/sf, and the 98.5 percent production efficiency target for a UFRV of 10,000 gal/sf.

In general, the average plant filter production has been less than 97 percent, but never less than 92 percent. It can be seen that the efficiency of the filters is closer to 97 percent during the peak production months of June through September and generally drops during the off-peak periods when total production is lower. This analysis indicates that the filters are not being operated as efficiently as possible due to excessive backwashing. This inefficiency is created by the City’s standard operating practice of limiting the maximum filter run time between backwashes to 24 hours, even if terminal headloss or turbidity breakthrough has not occurred. While a 24-hour filter run at plant flows at or above 8.0 mgd will result in greater than 97 percent efficiency, a 24-hour hour filter run at a 4.0 mgd plant flow will result in a 93 percent efficiency. Since the filters appear to be in decent condition and there are no “problem” filters, the City may be willing to consider extending filter run lengths during the months of October through May to a maximum of 36 to 48 hours in order to improve the annual production efficiency.

Special Filter System Analyses

Based on discussions with plant staff and visual observations during plant visits, there do not appear to be any filters which are behaving erratically or poorly in terms of higher effluent turbidities or unusual appearance or behavior during backwash. The filter media and support gravel are approximately 10 years old, having been installed in 1997 after the filter re-build. Therefore, no special filter analyses were conducted as part of this plant evaluation.

With older filters and/or with filters that have apparent problems, it is common practice to perform detailed filter investigations including the following:

- Filter coring and sampling, including: distance from trough lip to top of media, total media depth, and depth of different media layers
- Filter media size (by sieve analysis) and specific gravity measurements
- Support gravel depth and whether the gravel depth is uniform throughout the filter
- Visual observations of the filter before, during and after backwash to look for potential problems with the underdrain system, support gravel and media, including: surface cracking; mud balls on the surface of the filter; non-uniform backwash and/or air flow; “boiling”; etc.
- Backwash efficiency measurements including: bed expansion; dirty backwash water turbidity profiles; and floc retention analyses to determine how well the backwash process is removing particulates that are captured within the media

It is recommended that the City perform these tests on the existing filters within the next one to two years to determine the remaining useful life of the filter media and to determine if improvements can or should be made to the backwash procedures. A well-designed and properly maintained granular media filter system should have a useful life of 15 to 25 years before replacement is required.

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, since plant startup in 1992; however, plant production efficiencies are typically less than 97 percent throughout the year, and generally decrease to as low as 92 percent when plant production is lower. Plant efficiencies can be improved through longer filter run times. This would reduce the cost for each unit of water produced by reducing pumping, chemical costs and washwater production per unit volume.

An efficiency of 97 percent is considered the minimum desirable filter production efficiency and up to 98.5 percent is often achievable with conventional plants. Plant efficiencies can probably be improved by increasing the filter run times between backwashes to allow greater than 24 hour filter run lengths, up to 48 hour maximum run lengths. Of course, the plant should still terminate filter runs if terminal headloss and/or turbidity breakthrough occurs before the maximum run time occurs.

Presented below is a summary of historical plant performance and analyses presented in this section, along with observations of potential improvements and/or further study.

- The plant has performed well and reliably over a range of flows and water quality conditions.

- The plant has produced up to 11.5 mgd during peak demand periods.

- The plant has successfully treated the North Umpqua River supply even during extremely high turbidity events (up to 1,000 NTU) during 1995 to 1997.

- Coagulation chemistry seems to have improved with the use of ACH since 1999. This chemical produces excellent settled water and filtered water turbidities while also reducing plant operating costs through lower chemical costs and less sludge production compared to alum.

- The single flocculation/sedimentation basin uses a high-rate design with tube settlers to minimize space requirements, but appears to provide excellent pretreatment per historical data. This performance benefited from low turbidity and warm water during the high production periods in June through September. If the basin had to produce higher flows during the higher turbidity, colder water periods of the year, the basin might not perform as effectively.

- The City should consider installing on-line turbidimeters to continuously monitor settled water turbidity. This work can be accomplished as part of the proposed plant expansion.

- The plant has four mixed-media gravity filters with the support gravel and media being almost 10 years old since the 1997 re-build. The filters produce excellent filtered water and do not have any apparent problems at this time.

- The filter backwash procedures appear to be acceptable in maintaining clean filter beds.

- The City should perform detailed filter investigations, including corings, sieve analyses and backwash efficiency analyses, in the next one to two years to assist in the determination of the remaining useful life of the filter media and support media. This work can be accomplished as part of the proposed plant expansion.

- The City should install on-line particle counters for each of the filters to operate in parallel with the on-line turbidimeters. This work can be accomplished as part of the proposed plant expansion.

- The City should consider extending the maximum filter run length to as long as 48 hours to achieve better production efficiencies.

- The City should purchase a bench-top ultraviolet (UV) spectrophotometer to monitor total organic carbons. This should be accomplished independent of the proposed plant expansion.

- The finished water pH occasionally drops below 7.5 during late fall and winter conditions. The City may wish to consider seasonal use of the existing chemical feed system with a pH adjustment chemical such as lime or soda ash to maintain a minimum finished water pH of 7.5 to 8.0. This would provide a more-consistent and less corrosive finished water quality throughout the year.

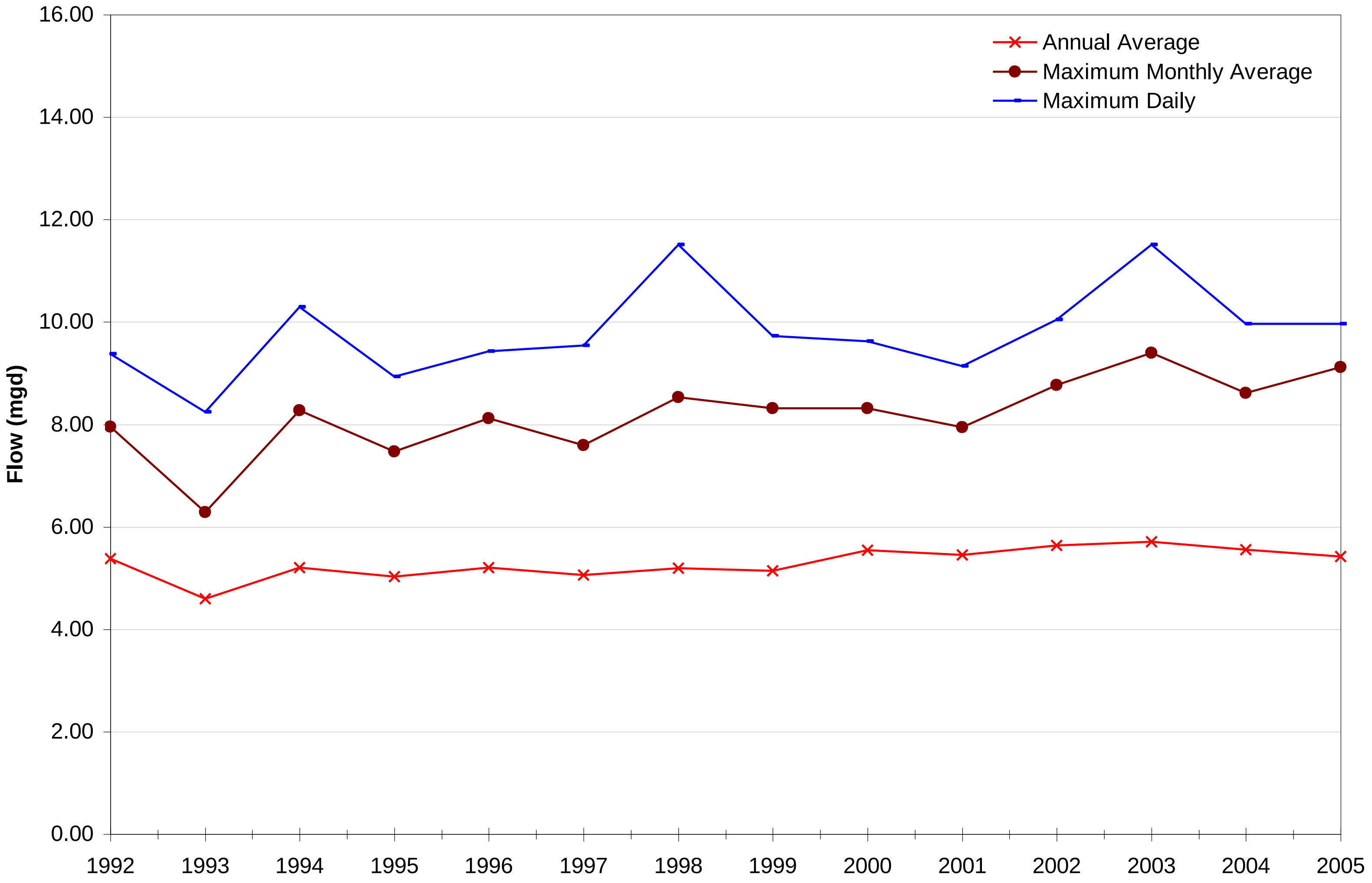


Figure 2-1
Historical Plant Production for the Winchester WTP

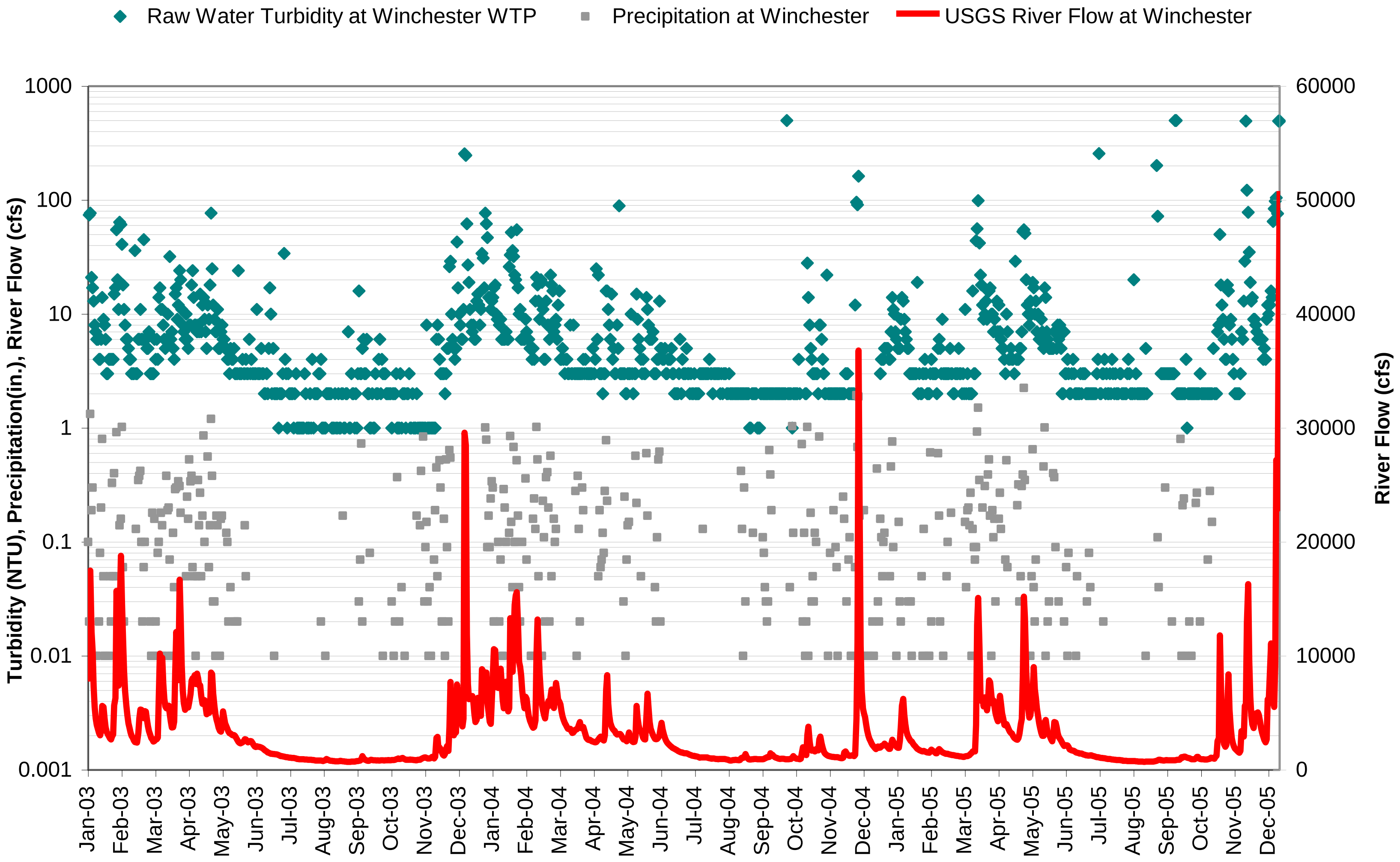


Figure 2-2
Average Daily Raw Water Turbidity, Precipitation and River Flow (2003-2005)

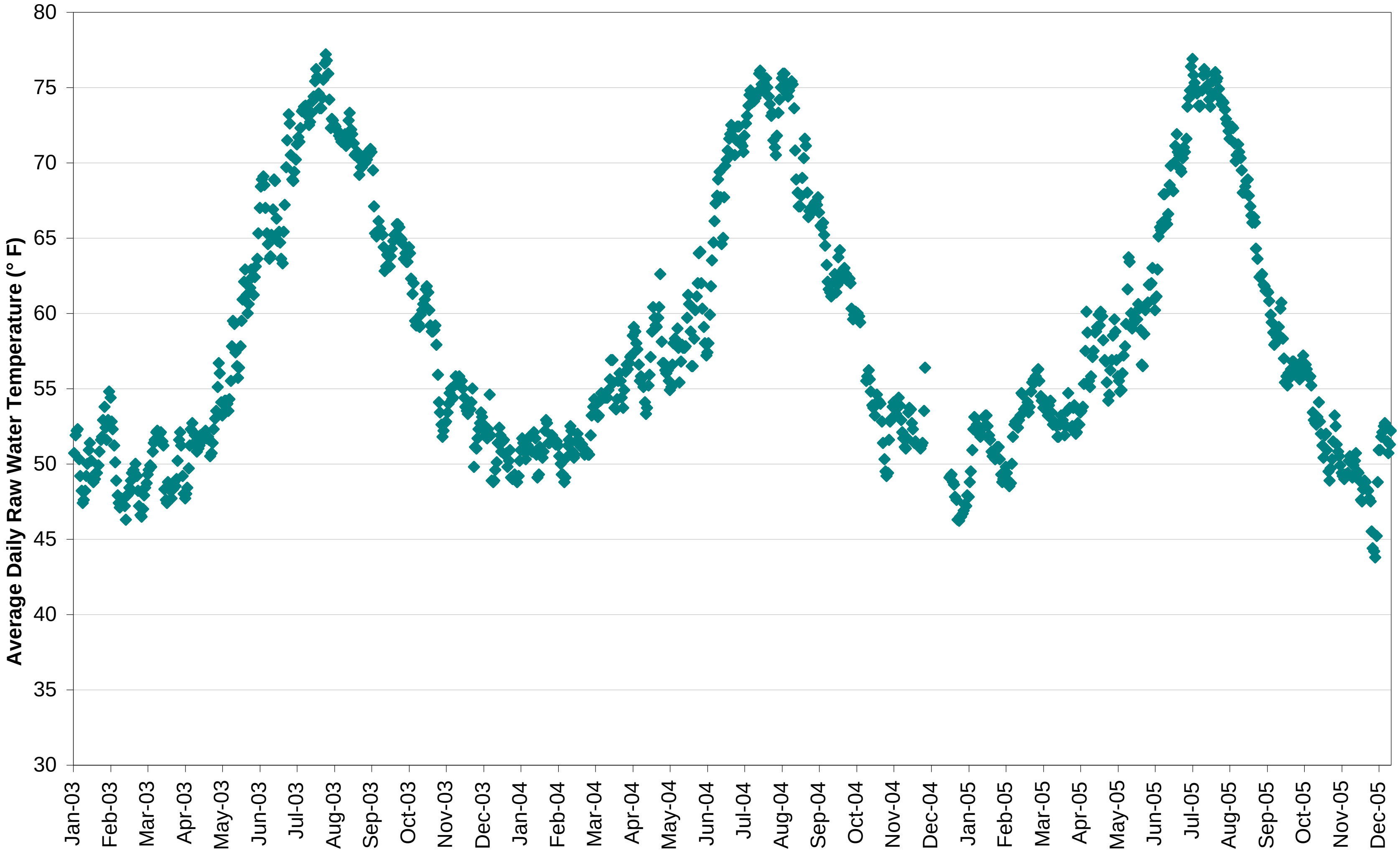


Figure 2-3
Raw Water Temperature at Winchester WTP (2003-2005)

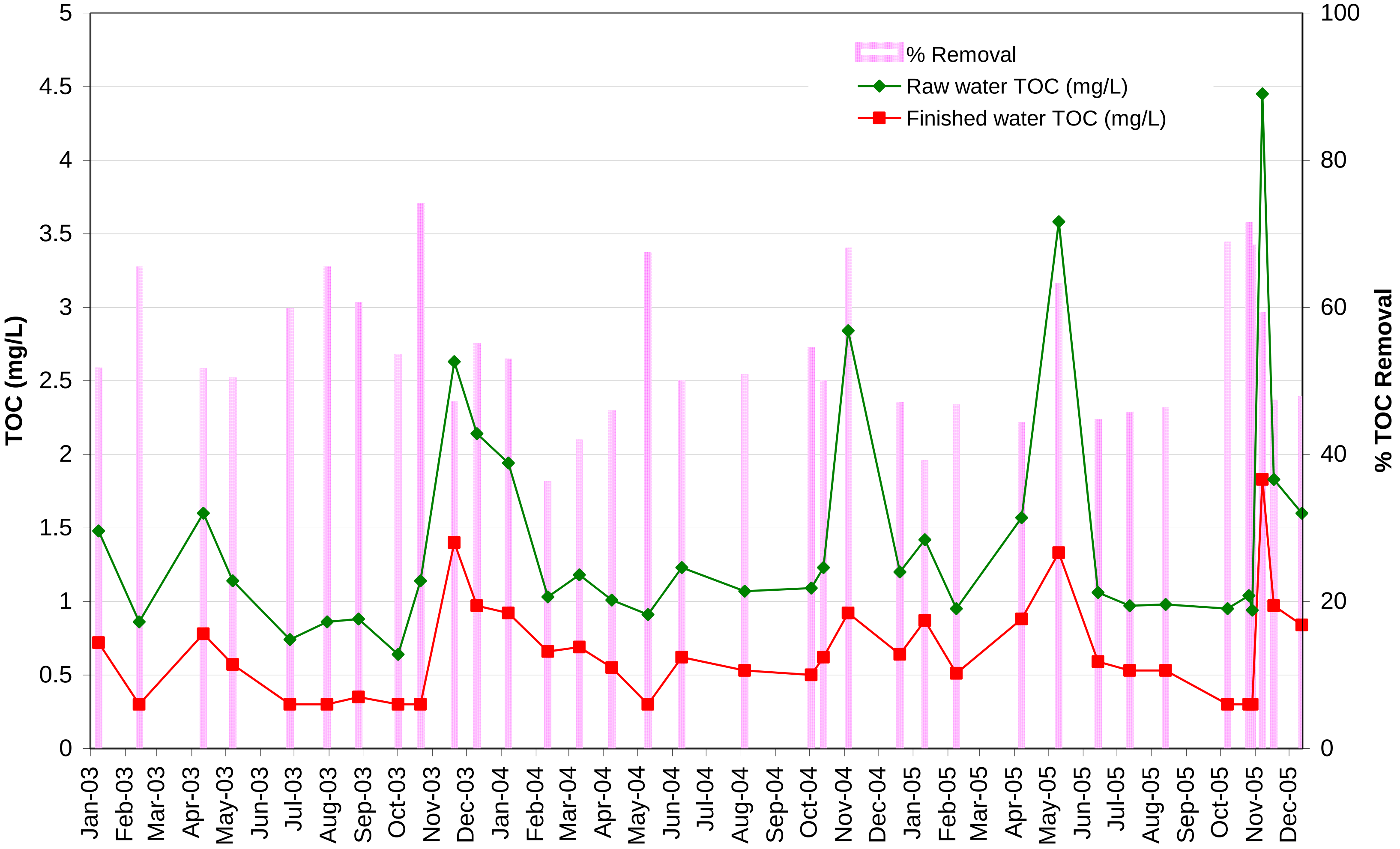


Figure 2-4
TOC Removal through the Winchester WTP

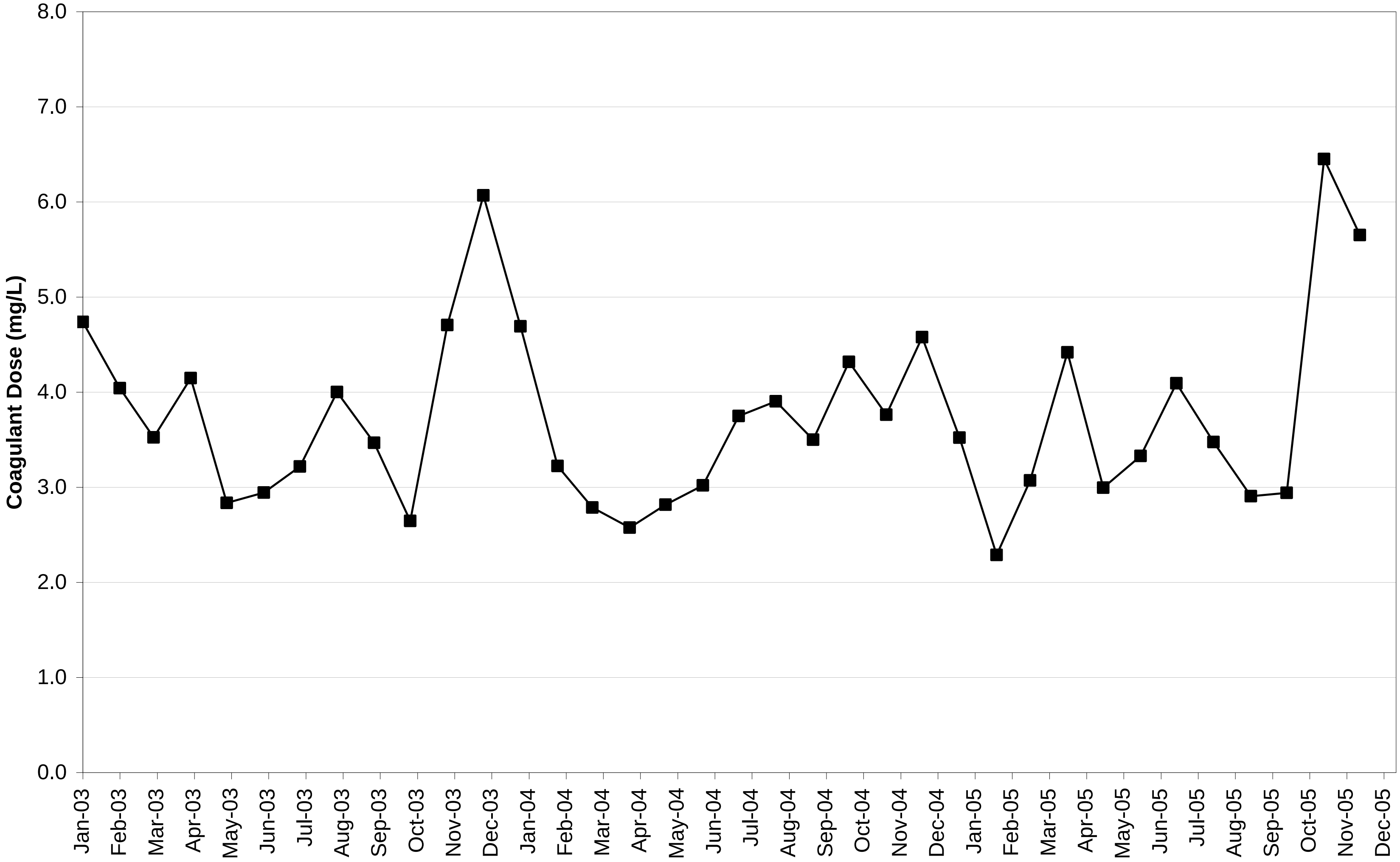


Figure 2-5
Average Monthly Coagulant Dose at Winchester WTP (2003-2005)

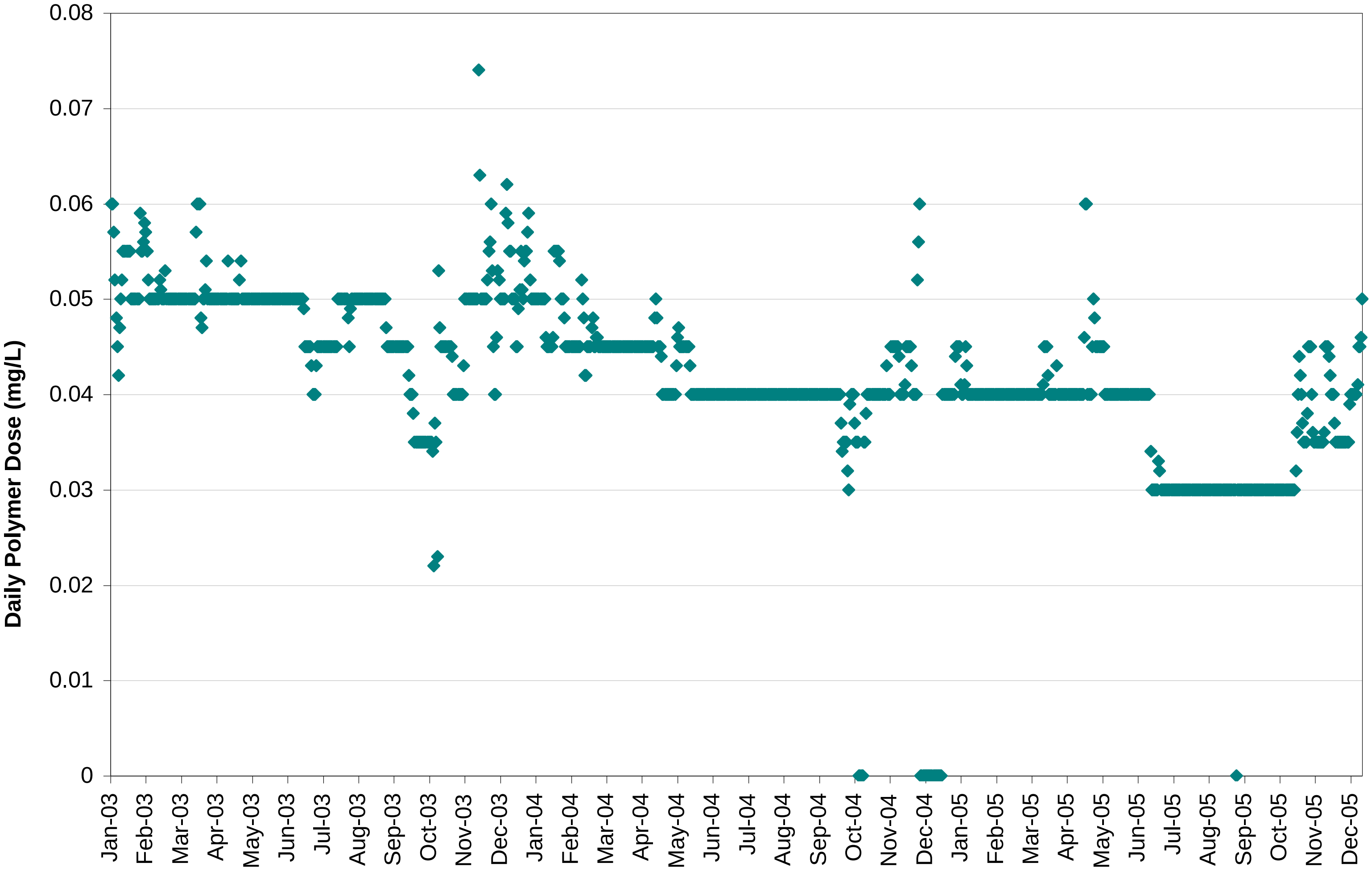


Figure 2-6
Filter Aid Polymer Dose at Winchester WTP (2003-2005)

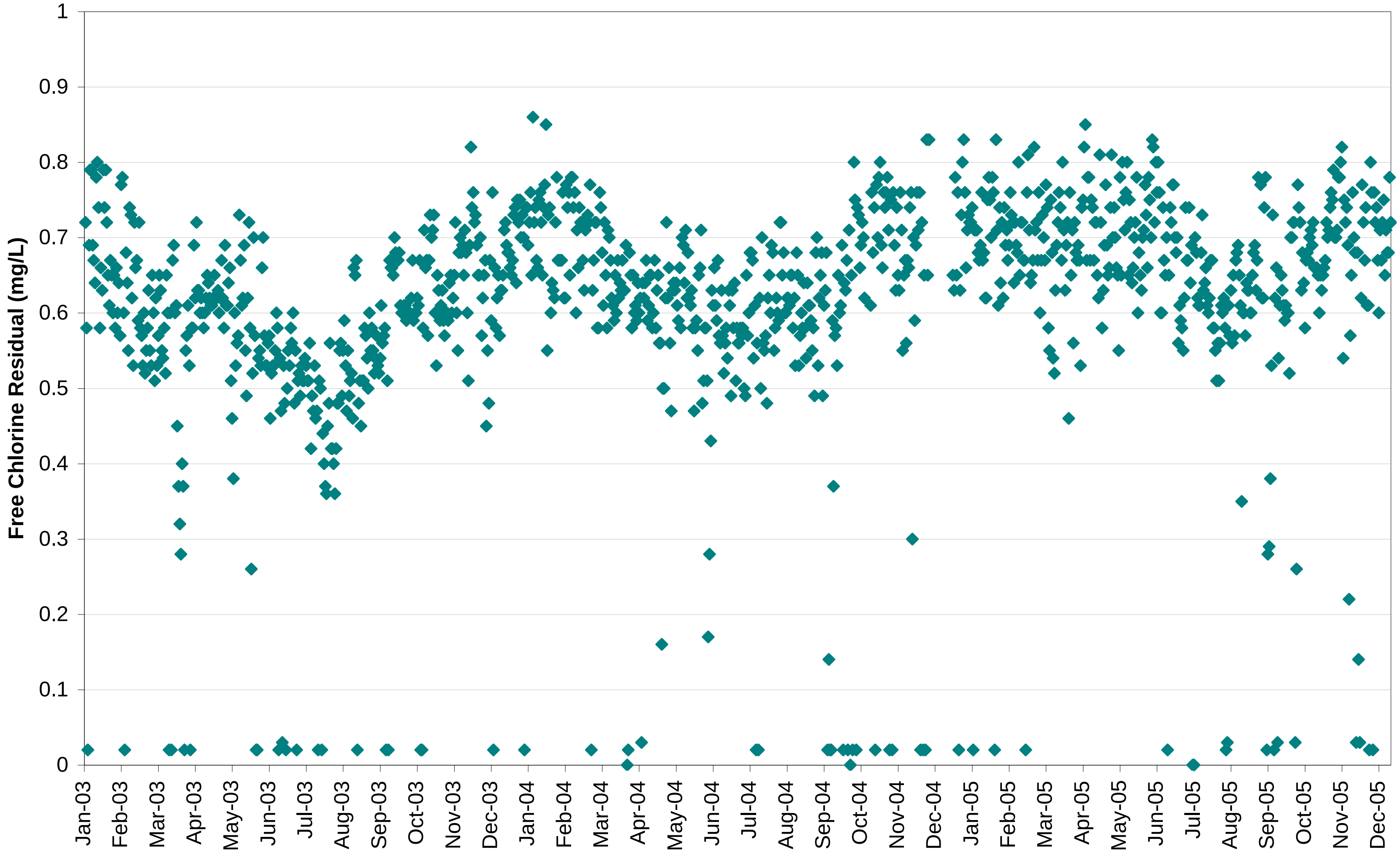


Figure 2-7
Minimum Free Chlorine Residual Leaving the Winchester WTP (2003-2005)

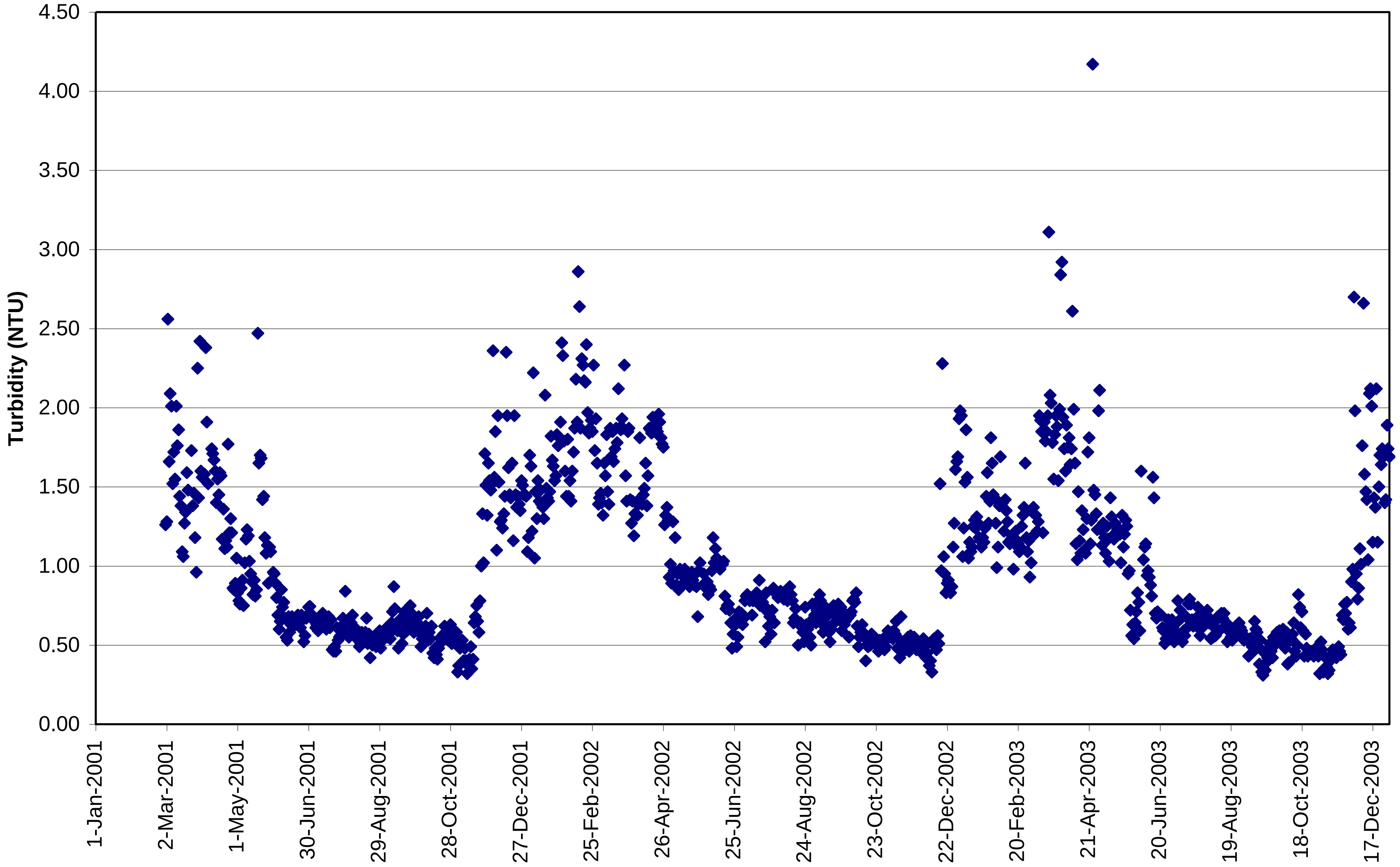


Figure 2-8
Daily Settled Water Turbidity at Winchester WTP (2001 - 2003)

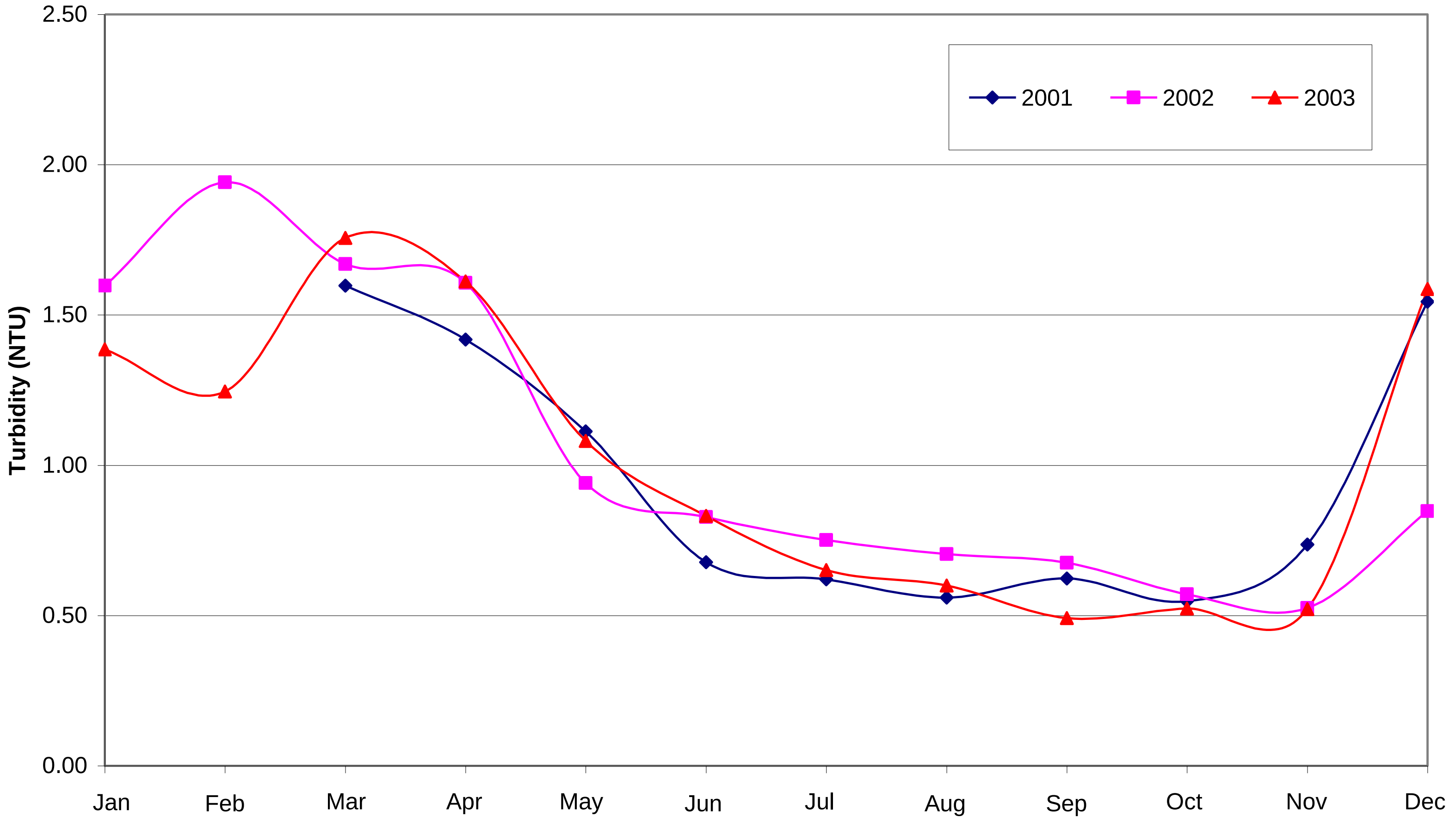


Figure 2-9
Monthly Average Settled Water Turbidity at Winchester WTP (2001 - 2003)

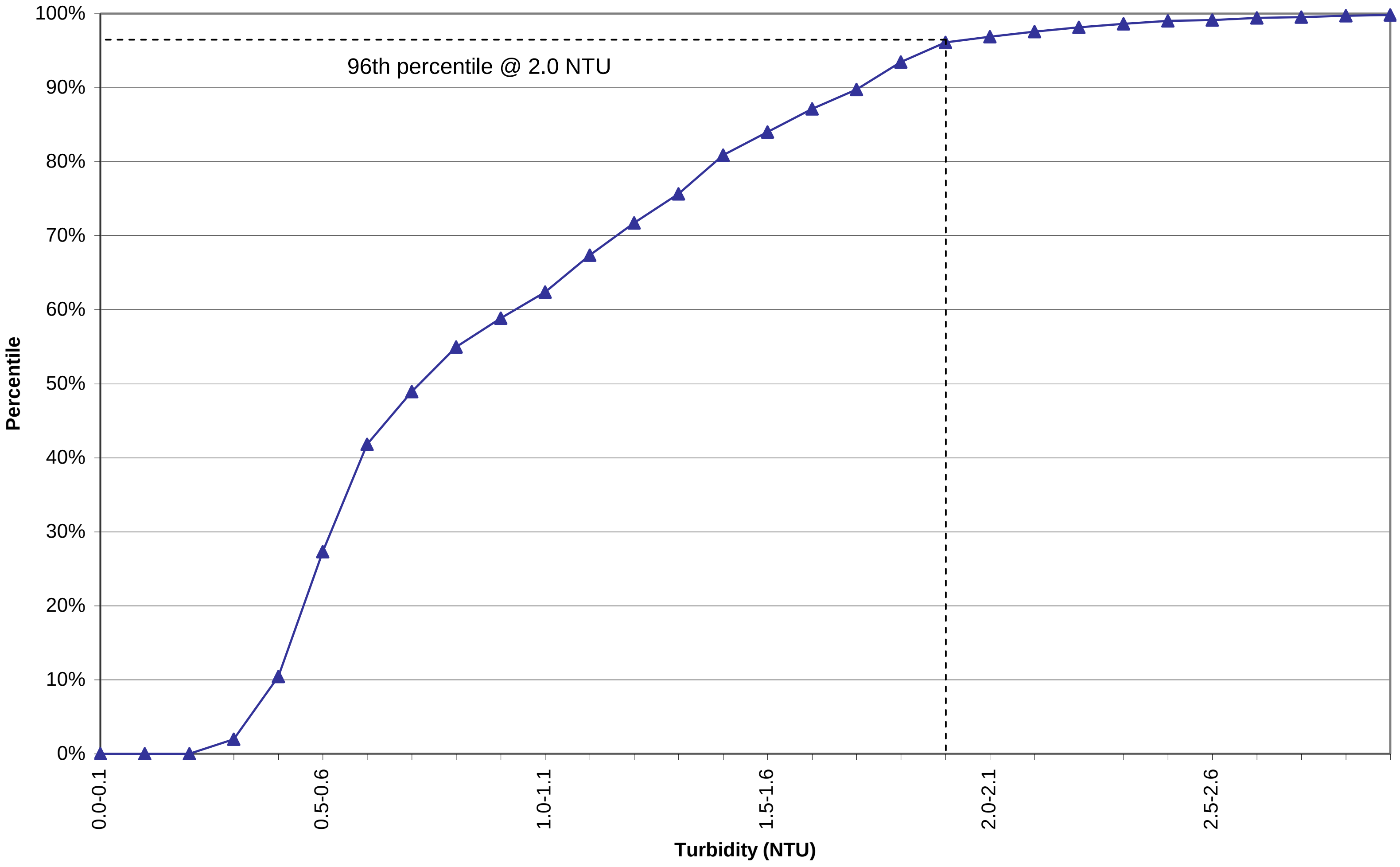


Figure 2-10
Statistical Summary of Settled Water Turbidities at Winchester WTP (2001 - 2003)

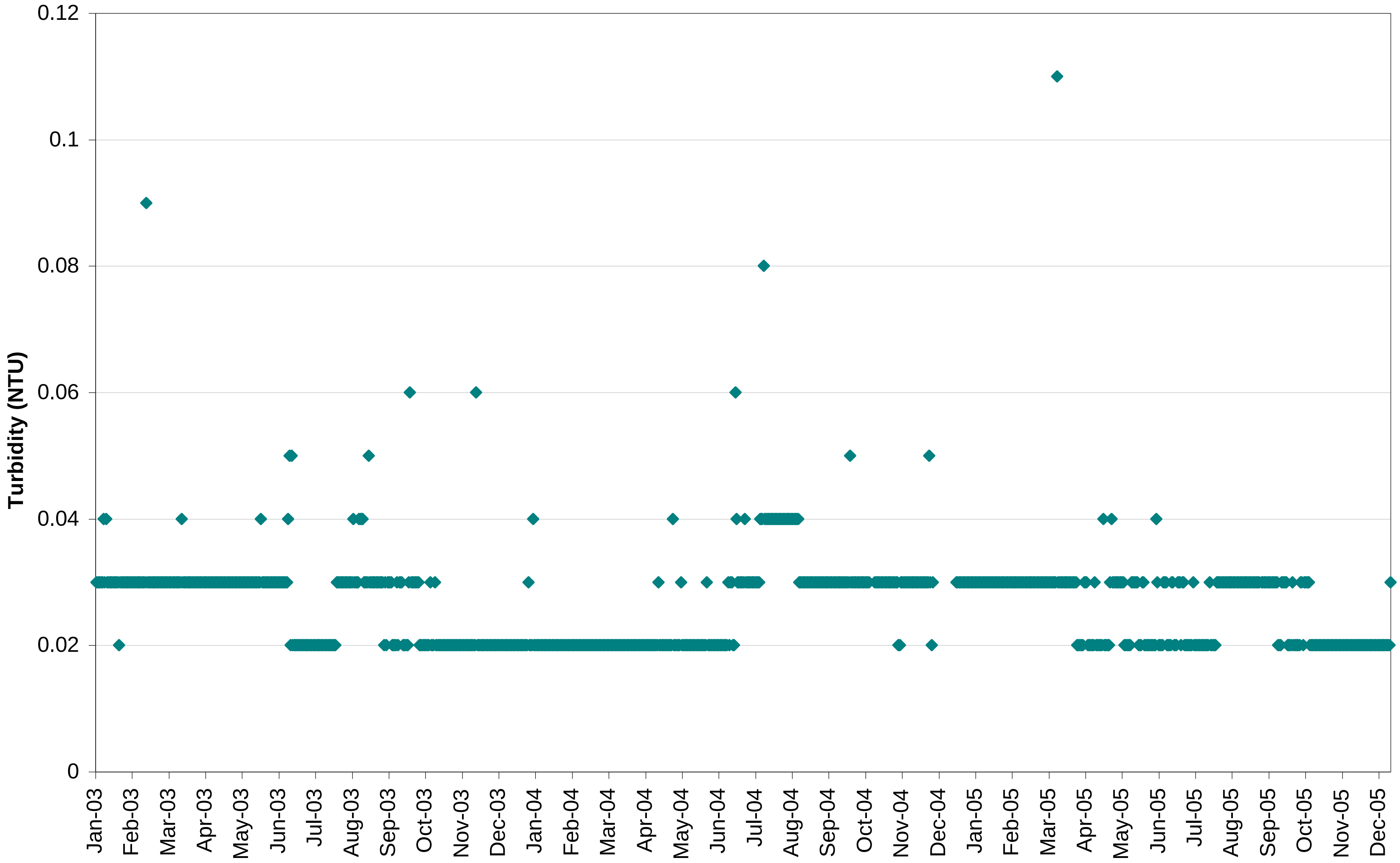


Figure 2-11
Average Daily Finished Water Turbidity at Winchester WTP (2003-2005)

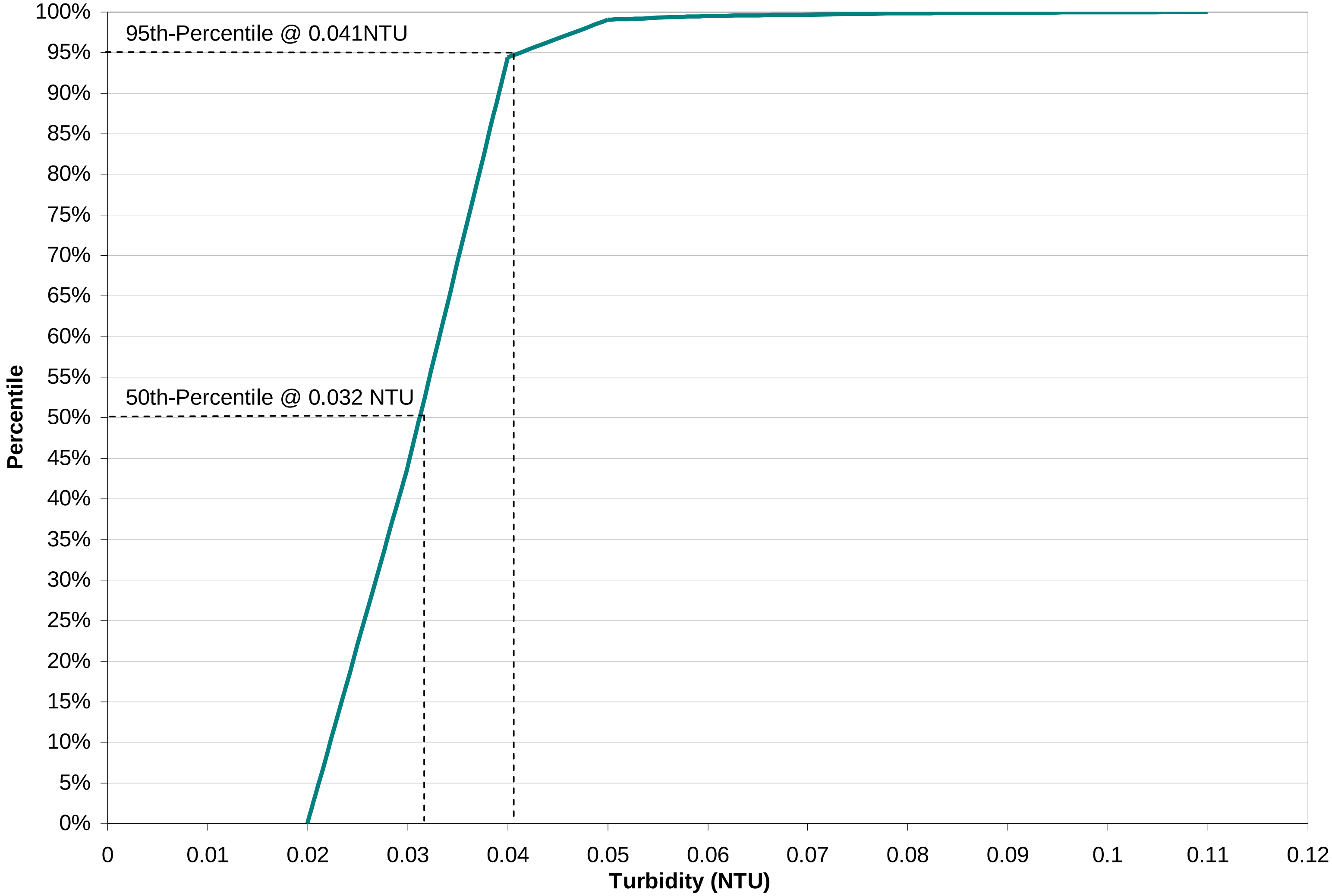


Figure 2-12

Statistical Summary of Average Daily Finished Water Turbidities (2003-2005)

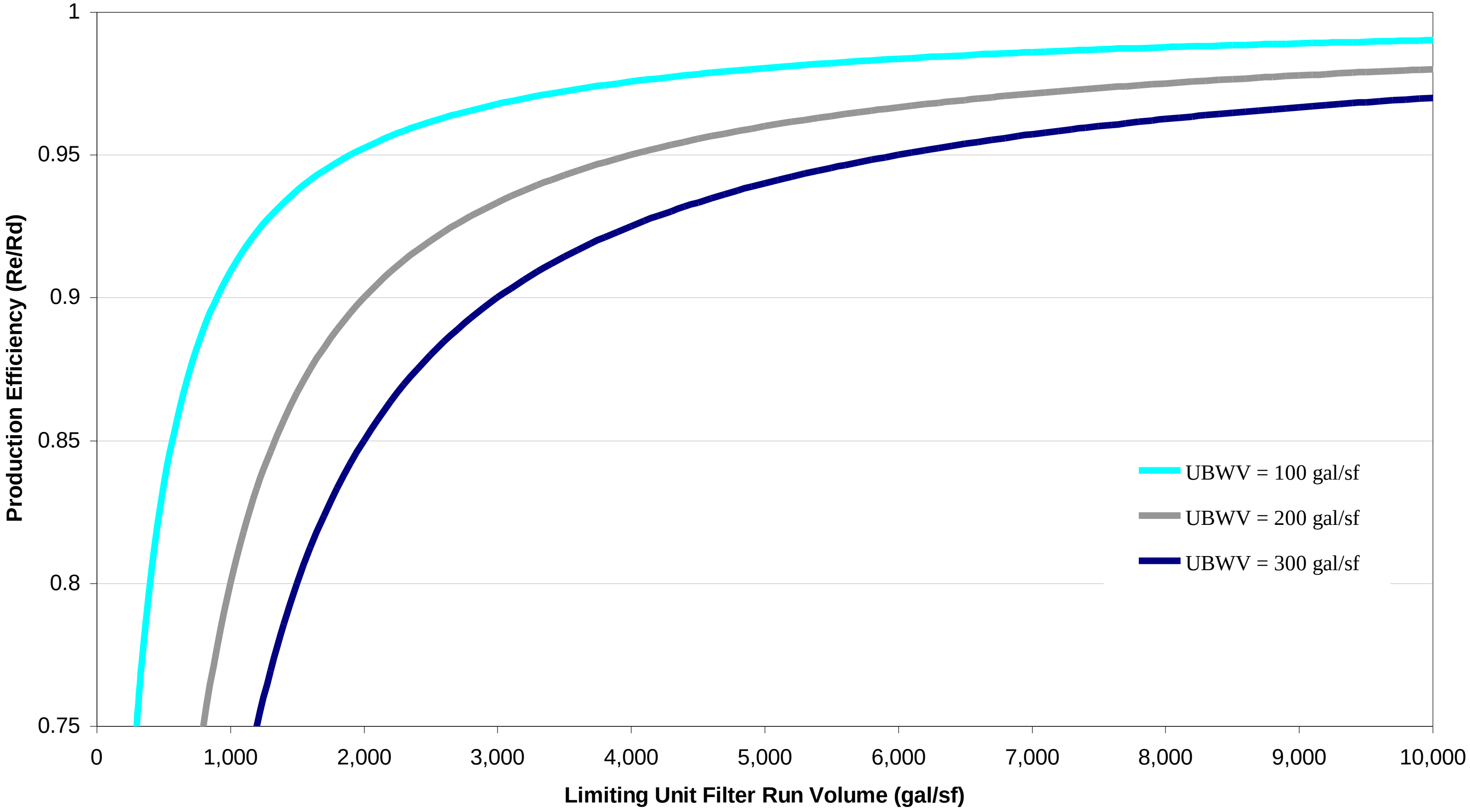


Figure 2-13
Influence of Limiting Unit Filter Run Volume and Unit Backwash Volume on Production Efficiency at Winchester WTP

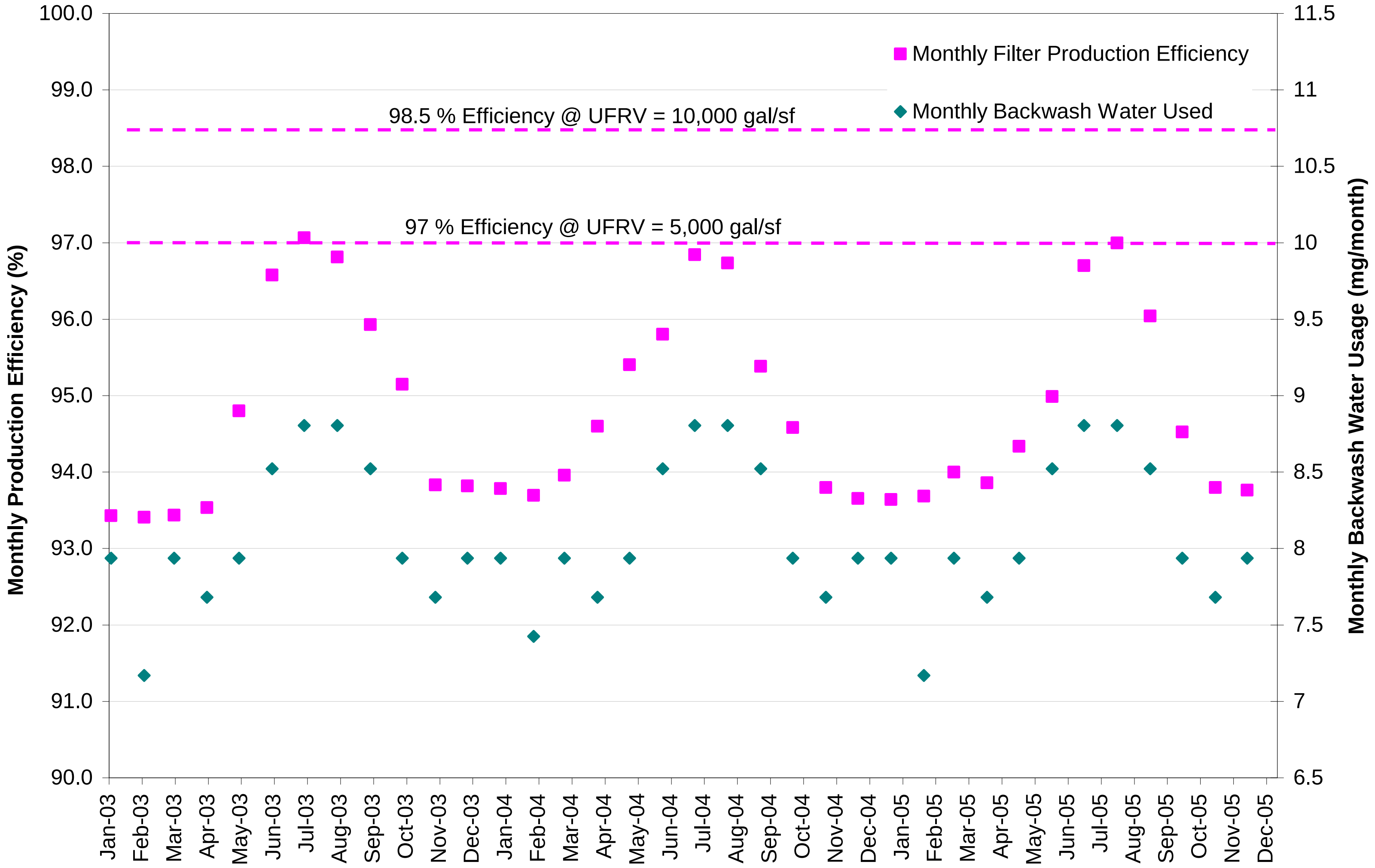


Figure 2-14
Monthly Backwash Water Usage and Production Efficiencies at Winchester WTP (2003-2005)

SECTION 3

Preface

The original version of this section was completed in the fall of 2006 and, at that time, was current as of June 2006. The final report was submitted in June 2008. Subsequently, the City requested an update of the report after it revised its long-term population forecasts which resulted in revised water demand forecasts.

Since 2006, there have been some changes to the Drinking Water Regulations which are discussed in this section. Specifically, the two regulations which are discussed as "Future Drinking Water Quality Regulations" now have compliance deadlines and/or other requirements. The Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) has a compliance deadline of October 2013. The City of Roseburg has completed almost all of the *Cryptosporidium* monitoring required by the LT2ESWTR and this data is included in this updated section. The State of Oregon published draft OARs for the Stage 2 Disinfectant/Disinfection By-Product Rule in February 2009 and is expected to take primacy in the summer/fall of 2009. Neither of these changes has a material effect on the conclusions or recommendations for the Winchester WTP which were made in the original version of this section.

General

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 June 2006. See Preface above.

Existing Drinking Water Regulations

Currently enforced national drinking water regulations that have implications for the City of Roseburg's Winchester WTP 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) – supersedes 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 Drinking Water Program of the Department of Human Services (DHS) (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 a 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.

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 on September 24, 1982. However, the microbial requirements outlined in the NPDWR have since been superseded 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, supersedes 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 supersede the NPDWR and outline improved filter monitoring and performance, as well as disinfection requirements. The recently promulgated Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR) will supersede the IESWTR. This future regulation is discussed later in this section.

Microbial Contaminants

Regulatory History - 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).

Regulatory History - 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.

<u>Overall Filtration Performance:</u> 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 treatment plants based on the types of processes provided by the plants. For conventional plants with filter-to-waste capabilities, such as the Winchester 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 the Winchester WTP, 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 compared to viruses. In Oregon, the DHS has also enacted a requirement that a minimum of 0.5-log inactivation of *Giardia* must be achieved following filtration and prior to the first customer. Hence, the only disinfection credit which the Winchester WTP can, and must, achieve is following filtration. No disinfection credit can be taken prior to filtration even if a chlorine residual is carried through the unit operations preceding filtration.

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 running annual average (RAA) exceed 80 percent of the new MCLs ($\geq$0.064 mg/L and/or $\geq$0.048 mg/L for TTHM and HAA_5, respectively), a disinfection profile will need to be developed. The historical DBP data collected by the City 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 one year of *Giardia* inactivation data tabulated by month from January 1999 to December 1999 and submitted it to DHS on May 8, 2000 in compliance with the rule**.**

Analysis of Roseburg's Compliance History, Coliform Rule

Coliform Bacteria: Historic microbial testing results for the City were obtained through the DHS. These results date back to January 1999. The City has two coliform violations on record at the DHS. The most recent is dated June 1, 2006; this violation corresponds to an inadequate number of samples submitted to the State.

The City had several samples test positive for total coliforms in 1999. A single sample tested positive for total coliforms in sampling done on each of the following dates: May 26, August 25, and November 17. Repeat sampling for all these positive results found no coliform present. However, on November 24, 1999, one sample tested positive for total coliforms and repeat sampling conducted on November 30 also had a single positive test result. The positive result was for total coliforms, not fecal coliforms. Repeat testing done on December 2 found no contamination.

Archived alerts on the DHS website indicate there were coliform alerts issued for samples taken on: July 9, 1997; May 29, 1996; January 11, 1995; and December 14, 1994. Details of these alerts are not available because the DHS coliform sample archive for the Winchester WTP has no results prior to January 1999. Since 1999, no samples have tested positive for coliforms. Historic treatment data indicates consistent compliance with the Oregon Drinking Water Quality Act’s coliform bacteria requirements.

Analysis of Roseburg's Compliance History --Surface Water Treatment

Overall Filter Performance: Combined (finished) filtered water turbidity is measured prior to the point of entry into the distribution system. A statistical analysis was performed on the average daily finished water turbidity data collected from January 2003 through December 2005 to determine regulatory compliance. Figure 2-13 presents the results of this statistical analysis.

From Figure 2-13, the finished water turbidity was less than or equal to 0.04 NTU 95 percent of the time over the 3-year period. Consequently, the City has met and/or exceeded all regulatory filtration standards in place at the time the data were collected.

Individual Filter Performance: On-line turbidimeters necessary for monitoring the individual filtered water turbidity have been used at the Winchester WTP for many years. Review of recent individual filter turbidity data indicate that there are no “problem” filters, as noted in Section 2; all filters have been performing well with regard to the regulatory requirements. In addition, none of the individual filter effluent turbidity thresholds which would result in a reportable violation per State law have ever been exceeded.

Disinfection Performance: CT-achieved through the WTP is calculated daily based on a formula and reporting method established by the State in 1992. 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 determined by the CT tables presented in the SWTR Guidance Manual for 0.5-log inactivation of *Giardia* with free chlorine (included in Appendix C of this report). The value is a function of minimum daily chlorine residual, minimum daily raw water temperature and maximum daily pH. Using this methodology, CT was consistently met at the WTP during the January 2003 to December 2005 period evaluated for this study. Also, the Winchester WTP has no violations with regard to disinfection residual monitoring or residual concentrations in the distribution system. The State conducted a Sanitary Survey of the Winchester WTP on August 24, 2006 during which no significant deficiencies were identified.

The following equations have been historically used to calculate daily CT-achieved through the plant:

1. T (min) = $\dfrac{\text{510,000 gallons*}}{\text{Plant Flow (gpm)}}$
2. C (mg/L) = Minimum In-plant Chlorine Residual
3. $CT_{achieved}$ (mg/L-min) = C x T

*Effective volume that the State identified for use in calculating CT.

The City has never conducted a tracer study through the plant or clearwell. The State has been performing Sanitary Surveys of drinking water treatment plants throughout Oregon over the past few years. As part of these survey recommendations, a requirement to perform a tracer study and to re-evaluate the disinfection CT calculations is often required.

The State requires that a minimum of 0.5-log inactivation of *Giardia* must be achieved following filtration and prior to the first customer. Hence, the only disinfection credit which the Winchester WTP can, and must, achieve is following filtration. No disinfection credit can be taken prior to filtration, even if a chlorine residual is carried through the entire plant. As noted in Section 2, the chlorine residual in the sedimentation basin at the Winchester WTP is so low that there would be no disinfection credit available even if the State were to allow for such pre-filtration credit.

It is unknown for certain what assumptions are embedded in the volume used for the above equation; however, some handwritten notes attached to a letter dated June 28, 1993 from DHS to the City indicate that the volume was arrived at by applying a 70 percent hydraulic efficiency to the entire storage volume beneath the filters, the main operations building and the high service pump station assuming the water surface is at the overflow depth of 11.5 feet.

For reasons explained below, the volume beneath the filters and the high service pump station cannot be used to calculate CT and the baffled clearwell probably has an efficiency of approximately 50 percent rather than 70 percent. As discussed below and in Section 4, the clearwell is typically operated between about 8 and 10 feet depth, rather than 11.5 feet. It appears likely, therefore, that the City must modify its historical CT calculation methodology. This change could potentially create a regulatory compliance challenge which needs to be discussed with the State before any further action is taken by the City.

CT Recommendations: The City will need to modify its CT calculation methodology in coordination with DHS. CT-achieved through the treated water storage and clearwell will need to be calculated daily to determine if it exceeds the CT-required for 0.5-log *Giardia* inactivation following filtration. Further analysis is required to determine if the existing clearwell design (volume and hydraulic efficiency (T_{10}/T_{th}) and operating conditions (flow, minimum operating level, chlorine residual, and pH) allow daily compliance with the State's disinfection requirements. It may be required to modify the treated water storage and clearwell design as part of the plant expansion (such as improved baffling), as well as modify some operating conditions (such as maintaining a minimum water level or increasing the chlorine residual), to ensure that CT is always met in the future. This is discussed further in Section 4.

CT Calculation and Optimization: To assist the City in understanding the ramifications of changing its CT calculation methodology as discussed above, an analysis was performed to estimate CT-achieved in the treated water storage and clearwell sections of the existing plant. Table 3-2 presents a summary of this analysis which indicates CT-achieved and CT-required over a range of flow and water quality conditions. Major assumptions used for this analysis include:

- The clearwell always remains at a water depth of 10.5 feet.
- The storage volume under the filters (approximately 170,000 gallons) is not available for chlorine contact time due to the lack of baffling. The effluent from Filter No. 1 can flow directly to the baffled clearwell with little or no contact time, and this represents the worst-case condition.
- The total volume of the baffled clearwell is approximately 324,000 gallons. A portion of the baffled area is bypassed by filtered effluent from Filter Nos. 1 and 3. Only that portion of the baffled area not bypassed is used to determine the chlorine contact time.
- The presumed hydraulic efficiency of the baffling is 0.5.
- The storage volume under the high service pumps (approximately 133,000 gallons) is not available for chlorine contact time due to lack of baffling.
- The minimum chlorine residual is 0.7 mg/L per historical records.

The CT-required values were taken from the tables in the Surface Water Treatment Rule Guidance Manual as presented in Appendix C of this report.

The information presented in Table 3-2 suggests that the City cannot meet the CT requirements prior to the high service pumps under many water quality and flow conditions, even with the assumption that the clearwell always remains full at 10.5 foot water depth. The clearwell water level normally varies between 7.6 feet and 9.2 feet above the bubbler tube, according to plant operators. The existing bubbler tube terminates about 6-inches above the invert of the clearwell, so the normal operating range for the clearwell is between 8.1 and 9.7 feet depth. The clearwell level decreases approximately one foot during each filter backwash and then slowly refills prior to the next backwash. Levels below 10.5 feet further reduce the values of CT-achieved compared to those shown in Table 3-2.

Table 3-2
CT ACHIEVED IN CLEARWELL AND CT REQUIRED UNDER VARIOUS CONDITIONS

Flow (mgd)	$\text{CT}_{\text{Achieved}}$ (mg/L*min)	$\text{CT}_{\text{Required}}$ (mg/L*min) pH = 8.0, T = 20° C Residual Cl_2 = 0.7 mg/L **Typical Summer Conditions**	$\text{CT}_{\text{Required}}$ (mg/L*min) pH = 8.0, T = 15° C Residual Cl_2 = 0.7 mg/L **Typical Spring & Fall Conditions**	$\text{CT}_{\text{Required}}$ (mg/L*min) pH = 7.5, T = 10° C Residual Cl_2 = 0.7 mg/L **Typical Winter Conditions**	$\text{CT}_{\text{Required}}$ (mg/L*min) pH = 8.5, T = 15° C Residual Cl_2 = 0.7 mg/L **Worst Case Warm**	$\text{CT}_{\text{Required}}$ (mg/L*min) pH = 8.0, T = 5° C Residual Cl_2 = 0.7 mg/L **Worst Case Cold**
3	47	13	17.5	21.5	20.5	34.5
4	35	13	17.5	21.5	20.5	34.5
5	28	13	17.5	21.5	20.5	34.5
6	24	13	17.5	21.5	20.5	34.5
7	20	13	17.5	21.5	20.5	34.5
8	18	13	17.5	21.5	20.5	34.5
9	16	13	17.5	21.5	20.5	34.5
10	14	13	17.5	21.5	20.5	34.5
11	13	13	17.5	21.5	20.5	34.5
12	12	13	17.5	21.5	20.5	34.5

Assumed depth in Clearwell =	10.5	Feet
Volume under HSPS =	132,515	Gallons
Volume under the 4 filters =	170,140	Gallons
$^{\mathbf{A}}$ Volume of Baffled Clearwell =	324,021	Gallons
$^{\mathbf{B}}$ Bypassed Clearwell Volume =	42,417	Gallons
$^{\mathbf{(A-B)}}$ **Active Volume for CT =**	**281,604**	**Gallons**
Assume Hydraulic Efficiency =	0.5	($T_{10}/T_{\text{theoretical}}$)
Typical Cl_2 Residual from data =	0.7	mg/L
Total Treated Water Storage =	626,676	Gallons

Notes : Full depth of 10.5 ft is considered for volume calculations
Shaded cells represent conditions where $\text{CT}_{\text{Achieved}}$ is equal to or greater than $\text{CT}_{\text{Required}}$
Unshaded cells represent conditions where $\text{CT}_{\text{Achieved}}$ is less than $\text{CT}_{\text{Required}}$

Figure 3-1 is a plot of finished water pH and temperature from January 2003 through December 2005. During typical winter conditions with cold water and lower pH, the plant can only meet CT at flows of about 6 mgd or lower. During the typical spring and fall conditions with warmer water but higher pH, the plant can only meet CT at flows of about 8 mgd or lower. During the typical summer conditions with high pH but even warmer water, the plant can only meet CT at flows of about 11 mgd or lower. It does not appear possible to meet CT at the plant design flow of 12 mgd under any conditions.

This evaluation indicates that the City is currently at risk of non-compliance with the State's disinfection requirements, and therefore action needs to be taken. Initially, the City should discuss this situation further with the State to determine an appropriate plan of action, probably in coordination with the plans for plant expansion. There are a variety of potential plant and/or system improvements which can allow the City to meet CT requirements following filtration. A review of improvement options for current conditions, as well as for the future expanded plant, is presented in Section 6.

Disinfectants and Disinfection By-Products

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 formed during 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 superseded 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 0.08 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. The Stage 2 D/DBPR will soon supersede the Stage 1 rule, as discussed later in this section.

Monitoring Requirements

The Oregon Drinking Water Quality Act requires monitoring of disinfection by-products. For the Winchester WTP, the current sampling number/frequency requirements for DBPs have been reduced to one sample/site per quarter representative of the maximum residence time. This reduction was allowed by the State due to low DBP concentrations during the first few years of sampling since 2000 when four samples per quarter were collected, 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.

As mentioned, for both TTHM and HAA_5, the monitoring frequency has been reduced since samples representing the longest system detention times contained 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 Roseburg 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 treatment plants 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.

Compliance with this treatment requirement must be calculated as a running annual average on a quarterly basis, after 12 months of data are available. Systems having raw water TOC concentrations less than 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 later in this section.

Table 3-4
TOC REMOVAL REQUIREMENTS (PERCENT)

Raw Water TOC (mg/L)	Alkalinity (mg/L as CaCO3)		
	0 – 60	60 – 120	> 120
2.0 – 4.0	35	25	15
4.0 – 8.0	45	35	25
> 8.0	50	40	30

Historic Compliance

On average, the reported running quarterly annual averages for TTHM and HAA_5 were 0.02 to 0.03 mg/L and 0.02 mg/L, respectively, for the period 2003 through 2005. Figure 3-2? presents DBP concentrations from 2003 through 2005. These low TTHM and HAA_5 concentrations were well below the thresholds of 0.064 mg/L and 0.048 mg/L, respectively, and support the reduced monitoring requirements granted by the State. No instances of

TTHM or HAA_5 MCL exceedances are on record. The City should continue to request reduced monitoring as long as low DBP concentrations are measured.

Historical raw and finished water TOC sampling between January 2003 and December 2005 indicate that TOC levels in the North Umpqua River may occasionally exceed the "trigger" level of 2.0 mg/L during the winter months. However, the annual average TOC concentrations were 1.45 mg/L, significantly less than 2.0 mg/L. Figure 2-4 presents the monthly raw and finished water TOC during the past 3 years. TOC removal has averaged over 50 percent, which would meet the requirements for enhanced coagulation (a minimum of 35 percent TOC removal with alkalinity less than 60 mg/L as $CaCO_3$) per Table 3-4.

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.

Compliance with the Stage 1 D/DBPR has been successfully achieved by the Winchester WTP and it appears as though the City is well-positioned to remain in compliance with the Stage 2 Rule when it is promulgated. This compliance is possible due to low TOC concentrations in the source water, coupled with excellent removal of TOC through the plant.

Lead and Copper and Corrosion Control

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.

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 Roseburg (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.

Historic Compliance

Initial lead and copper sampling began in Roseburg in 1992. Since then, 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. The City has used ACH as the primary coagulant since 1999 and sodium hypochlorite as the disinfection chemical since 2002. Neither chemical depresses the pH. Thus, WTP operators have for the most part maintained finished water pH values above the minimum pH of 7.2 required for LCR compliance without requiring post-filter pH adjustment. However, as noted in Section 2, there have been rare occasions when the finished water pH fell to 7.0 for a period of one day.

The most recent measurements, taken on August 18, 2005 report 90th percentile values of 0.0 mg/L for lead and 0.058 mg/L for copper. These values are well below the current action levels for lead and copper. The raw water and finished water pH vary seasonally from minimum values in the low 7.0's in winter/spring to maximum values in the mid 8.0's during summer and early fall.

Inorganic Contaminants

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. The rule reduced 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.

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 was 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.

Historic Compliance

Finished water from the Winchester WTP has remained in compliance with regard to all IOC MCLs during the period evaluated. The only detection of IOC's on record at the DHS since 1992 have been one detection of lead and several detections of fluoride. Lead was detected at a concentration of 0.003 mg/L on January 10, 1996. Fluoride has been detected five times since 1992, with the concentration ranging from 0.11 to 0.20 mg/L.

Roseburg installed asbestos cement (AC) pipe during between 1966 and 1982; 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 Winchester WTP.

Organic Contaminants

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.

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.

Historic Compliance

No concentration of regulated VOCs or SOCs above the detection limit is on record between April 2000 and March 2003.

Radiologic Contaminants

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.

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.

Historic Compliance

The most recent radiologic samples were taken by the City August 27, 2003. No radiological contaminants were present at concentrations above the detection level. Roseburg has fully complied with all DHS radiologic standards.

Federally Monitored Unregulated Contaminants

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

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 3-5. 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 Roseburg monitor List 3 contaminants.

Table 3-5

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	

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 2007.

Historic Compliance

The City was required by the EPA to sample for List 1 and List 2 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 or List 2 constituents were detected in the Roseburg water system.

Future Drinking Water Quality Regulations

General

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 recently promulgated rules, estimates of the timetables for promulgation, and projected effects on the City of Roseburg 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)

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 two 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 affect Roseburg. 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 2006 and 2011.

Compliance Requirements

Many revisions to the LT2ESWTR have been made since the first publication. The most recent requirements that apply to the City of Roseburg 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 2006-2007 (The City started this sampling and testing program in December 2006and completed most of it in late 2008. Two additional samples are required to have 24 months of data). 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 3-6 presents the proposed treatment requirements for conventional plants based on results from the monitoring program.

Table 3-6
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	0.075 – <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 Combined Filter Effluent of 0.15 NTU (95 percent of the time), or 1.0 log for individual filter effluent less than/equal to 0.15 NTU (95 percent of the time). Cannot get credit for both.
- Membranes - Challenge test.

Surface water systems serving greater than 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").

In addition to raw water monitoring requirements, the LT2ESWTR will require all systems to perform disinfection profiling. Disinfection profiling was required for public water systems who measured TTHM or HAA_5 levels in excess of 80 percent of the new MCLs (≥0.064 mg/L and/or ≥0.048 mg/L for TTHM and HAA_5, respectively), during preliminary testing as part of the Interim ESWTR. The specific requirements for disinfection profiling are discussed above in this section. 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 modifications are made.

Implications for the Winchester WTP

It is not anticipated that the North Umpqua River contains *Cryptosporidium* oocysts at concentrations above the upper limit for Bin #2 classification (1.0 oocysts/L); instead, it is expected that the Winchester 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.

The sampling and testing results performed by the City to date for the required 24-month *Cryptosporidium* monitoring program on the North Umpqua River (22 months of data collected so far) indicate that the WTP will fall into the Bin #1 classification. The City's sampling and testing program is anticipated to be completed in the summer, 2009 by taking two additional samples. Only 2 of the 22 samples had a detectable oocyst reported, and both of these were a single detection. The average oocyst concentration to date is well below the 0.075 oocyst/L concentration threshold for Bin #1 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 (less than 0.10 NTU 95 percent of the time) will account for the required 1.0-log additional removal treatment requirement. Currently, individual filter effluent turbidities average 0.03 NTU (see Figure 2-12). Filter improvements may be required to enhance filter performance in the future, depending on the condition of the filter media. 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 very unlikely. However, if Roseburg 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. It is assumed for this report that no major changes will be needed at the Winchester WTP.

Stage 2 Disinfection By-Products Rule

The purpose of the Stage 2 Disinfection By-Products Rule (D/DBPR)is to further reduce health risks associated with disinfection by-products by requiring utilities to meet DBP maximum contaminant levels (MCLs) at each monitoring site within the distribution system. The final rule was released in January 2006. The largest utilities were required to begin conducting additional monitoring in October 2006 and achieve compliance with the new MCLs by 2012. Requirements for smaller utilities follow six to eighteen months later, depending on the population served.

For Roseburg, compliance with the Stage 2 D/DBPR has two stages, as described below:

- Monitoring: The City must prepare an initial distribution system evaluation (IDSE) monitoring plan that includes locations near the entry point, with average residence time, and anticipated to have high total trihalomethane (TTHM) or haloacetic acid (HAA5) levels. The monitoring was to have begun by October 2007, and be completed by September 2009, with a final report submitted by January 2010. Systems with very low DBP levels may qualify for reduced monitoring, if every sample taken for eight consecutive quarters contains no more than 40 µg/L TTHM and 30 µg/L HAA5.

- Compliance: After October 2013, the City will begin compliance monitoring under the Stage 2 D/DBPR requirements. For each monitoring location, the City must calculate a locational running annual average (LRAA). These LRAAs are then used to determine compliance with the MCLs for HAAs, and DBPs, which remain at 80 µg/L and 60 µg/L, respectively.

Implications for the Winchester WTP

Based on historical low DBP concentrations within the Roseburg system (as presented in Figure 3-2), it is anticipated that the City will have no major compliance issues for the Stage 2 D/DBP Rule.

Other Compliance Issues

NPDES Discharge Permit

Plant solids from waste washwater, filter-to-waste and the sedimentation basins are collected in two holding/settling basins and the thickened sludge is then transferred to one of two solids handling and drying beds. The concrete basins and concrete drying beds were recently constructed to replace old earthen lagoons and drying beds. The holding basins discharge clarified overflow, and the beds discharge decant, back to the North Umpqua River downstream of the intake. The City discharges under NPDES permit No. 200-J, which is a general permit covering discharge or land application of filter backwash, settling basin and reservoir cleaning water as well as the flushing of raw water intakes after storm events and spring runoff. This general permit requires a pH between 6.0 and 9.0 and total settleable solids less than 0.1 ml/l. It also requires that the chlorine residual be monitored if chlorinated water is used for backwash.

Historic compliance with NPDES permit requirements has been maintained during the three-year period evaluated for this report. As long as the City continues to receive extensions for this NPDES permit from the State, there are no immediate or long-term improvements required at the Winchester WTP. If this permit were to be terminated for some reason in the future, the City would need to evaluate its solids and liquid residual handling options. This could result in the need for an in-plant recycling program to avoid direct discharges to the river. However, given that there is no reason to anticipate termination of the NPDES permit, in-plant recycling is not considered for this report.

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 North Umpqua River. These 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: an approach velocity at or below the maximum; a screen opening size less than the maximum; a sweeping velocity at the face of the screen to ensure that juvenile fish are not trapped in front of the intake; and, where trash racks are used, a hydraulic gradient to route juveniles fish from between the trash rack and screen to safety. For the North Umpqua River, the maximum approach velocity for an intake structure with an automatic screen cleaning system is 0.4 fps. The sweeping velocity must be sufficiently high to ensure that juvenile fish are not exposed to the screen for a period of time exceeding 60 seconds.

At the existing intake, the maximum velocity occurs at the 4’ x 4’ sluice gates at the entrance to the entry sump. The top of each gate is about six inches below the ordinary low water, so the gates typically remain submerged. With a maximum approach velocity of 0.4 fps with the gates submerged, the maximum flow into each sluice gate is 4.14 mgd and the maximum flow into the intake with all four gates open is 16.5 mgd. Thus, as long as the WTP operates at less than 16 mgd, the maximum approach velocity will not be exceeded when all four gates are opened.

The existing travelling screen opening size of 3.18 mm is greater than the maximum allowable opening size of 2.38 mm, and the gaps between the opening sidewalls and the travelling screens are also 3.18 mm. Although the existing screen opening size of 3.18 mm is greater than the maximum allowable opening under current regulations, it is in compliance with the previous criterion. The regulatory framework allows facilities that were designed in compliance with the previous screen opening size to continue operating without upgrading until either significant changes are made to the structure (such as replacement of screens, trash racks or structural elements) or until flows into the intake begin to exceed the existing criterion for maximum approach velocity.

To meet all of the fish protection criteria, it may be necessary to replace the travelling screens with an alternative screening system such as fixed screens located on the exterior of structure. Such a modification would locate the screens at the diversion entrance, which is currently the preferred location for fish screens. With fixed screens installed where the trash racks are currently located, the sluice gates would be on the interior side of the screens; thus, the approach velocity would not be determined at the sluice gates, but rather at the exterior face of the screens. With two fixed screens, each covering one of the two 10.83 ft wide openings into the structure, the intake could draw 22 mgd without exceeding the 0.4 fps approach velocity as long as the screens maintain a submergence of 4.5 ft. The ordinary low water level is 4.5 ft above the bottom of opening where the screens would be installed. Thus, installation of fixed screens could enable the intake to withdraw up to 22 mgd from the river without violating current fish protection criteria.

It may be necessary to attach trash racks to the exterior of the intake to protect the fixed screens. A new automatic screen cleaning system would be required. Options include: mechanical cleaning; water nozzles; and air burst. Further discussion regarding improvements to the intake is presented in Section 6.

Summary and Recommendations

In general, the Winchester WTP has consistently met all existing water quality regulations since it started operations in 1992. The two biggest drinking water regulatory issues of concern at this time are:

1. Ability to consistently meet 0.5-log *Giardia* inactivation following filtration under all current and future plant flows and under a wide range of plant operating conditions (clearwell level, finished water temperature, finished water pH and finished water chlorine residual), and

2. Bin classification per the LT2ESWTR depending on raw water *Cryptosporidium* concentrations.

The City should evaluate post-filtration CT compliance as part of the preliminary design effort for the upcoming plant expansion, as well as discuss the CT calculation methodology further with DHS. The evaluation should include a tracer study to determine the actual baffling efficiency in the existing clearwell. The City anticipates completing a tracer study in the fall of 2009. These data can then be used for updating the City's calculations of CT and for designing the improvements to the clearwell baffling that will be needed to increase contact time for the plant expansion. The City should complete its 24-month monitoring program for *Cryptosporidium* to verify that it falls into Bin #1. This classification requires no capital investment for facilities to inactivate *Cryptosporidium*.

Historical DBP concentrations are low enough to allow the City to continue its reduced monitoring requirements with agreement from DHS; however, as part of the Stage 2 D/DBP Rule, the City will have to complete the required IDSE to determine the appropriate DBP sampling location(s) and frequency. The IDSE should ideally be completed by early 2007. The City should discuss the IDSE requirements further with DHS before initiating this effort.

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.

The City should consider adding on-line particle counters to each filter effluent, in parallel with filtered water turbidimeters, to better understand filter performance and to anticipate turbidity breakthrough and other performance problems earlier.

As noted in Section 2, the finished water pH occasionally drops below 7.5 during late fall and winter conditions, although the pH has not dropped below 7.0 since the City changed coagulation and disinfection chemicals. Even though the City has remained in compliance with the Lead and Copper Rule, the City may wish to maintain a minimum finished water pH of 7.5 to 8.0 to provide a more-consistent and less corrosive finished water quality throughout the year. This would require seasonal use of the existing chemical feed system to add a pH adjustment chemical such as lime or soda ash.

A regulatory issue that may be dealt with on a longer time frame is modification of the fish screens to meet current requirements. Currently the only clear compliance issue is the screen size. However, as the intake flow increases the maximum approach velocity will limit the flow to 16.5 mgd. The City may decide to modify the intake when the plant is expanded to 18 mgd, or it may choose to wait to address the issue of the fish screens later, either when directed to do so by the regulating agency or when doing so is required due to increased demand approaching 16 mgd.

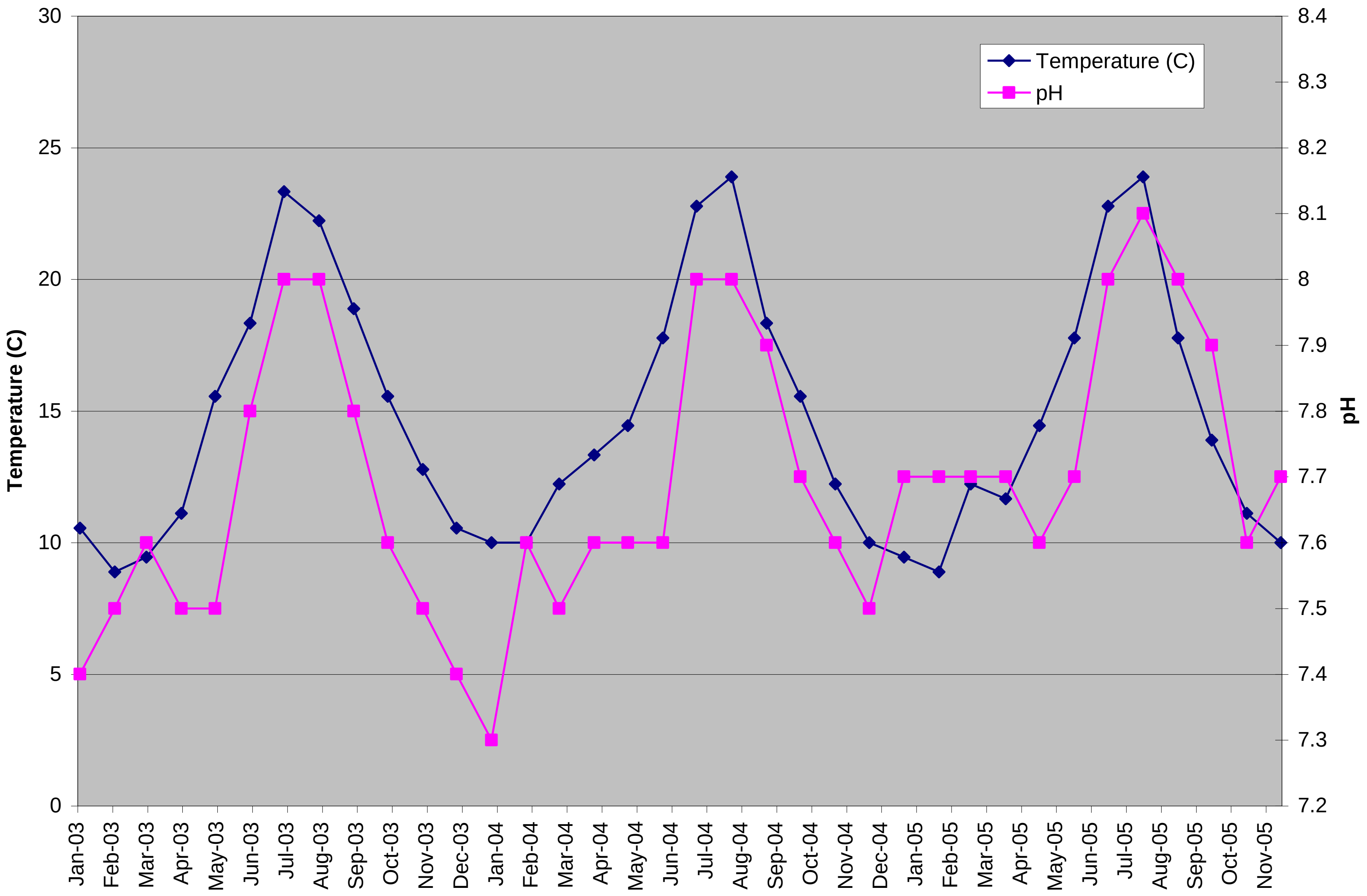


Figure 3-1
Finished Water pH and Temperature at Winchester WTP (2003 - 2005)

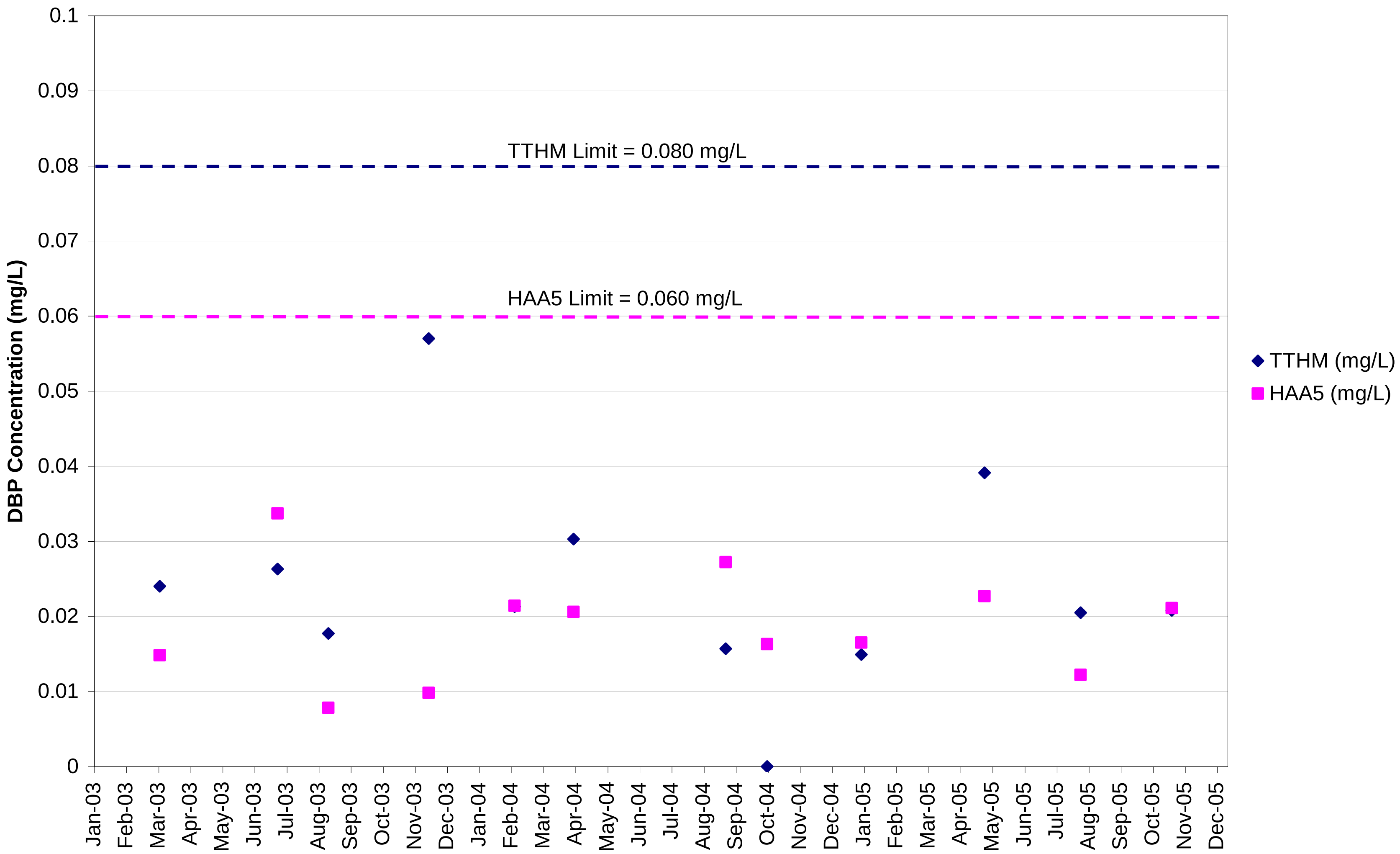


Figure 3-2
Historical Disinfection By-Products in the Distribution System (2003-2005)

SECTION 4

General

A review of the hydraulic and treatment capacities of the Winchester WTP was performed to determine the current capacity and possible future capacity given the limitations of each process and the interconnected system as a whole. The hydraulic capacity is related to the piping, pumping, volume and flow control systems, which limit the ability of the water to flow through individual unit operations and through the interconnected system as a whole. The treatment capacity is related to the ability of each treatment unit process to meet regulatory requirements or generally accepted industry standards, whichever is applicable.

The Winchester WTP has a current rated plant capacity of 12 mgd. This is a treatment capacity limitation determined by the four existing filters. Most of the existing equipment and treatment processes already have capacities exceeding that value. The plant was designed to be expanded to a nominal capacity of 18 mgd. This is also a treatment capacity limitation based on an ultimate build out of six filters using the same media as the existing four filters. Several unit operations will require modification to achieve this capacity. This section of the report evaluates the existing plant capacities and identifies which unit operations will require modification to achieve 18 mgd. Hydraulic and treatment capacities are discussed and evaluated separately.

The City currently has water rights on the North Umpqua River totaling 20 mgd and is investigating the possibility of obtaining additional rights from that source. This topic is discussed in greater detail in the Long-Range Water Supply Plan for the City of Roseburg. Since the existing water rights exceed 18 mgd, and because there is the possibility of obtaining additional rights, this section also analyzes the ability of individual unit operations to accept higher flow rates, between 18 and 24 mgd. For each unit operation, the likely ultimate hydraulic and treatment capacity is analyzed, modifications required to achieve the higher rate are identified, and suggested improvements are noted if they appear to be feasible. Figure 4-1 at the end of this section shows the general site plan for the existing Winchester WTP.

Hydraulic Capacity Evaluation

General

The maximum raw water pumping rate with all raw water pumps operating is nominally 22 mgd; however, during a test on June 24, 2003, the total combined flow of all four raw water pumps was 20.9 mgd. The maximum instantaneous flow rate of finished water from the high service pump station is nominally 16 mgd with all finished water pumps operating; however, the maximum actually achieved is about 14 mgd.

The plant currently operates continuously throughout each day on a “24/7” basis to keep the terminal reservoirs on Reservoir Hill full to meet demands in the City’s distribution system. The plant flow rate is typically adjusted on a daily basis, and occasionally more frequently when required, as water demands fluctuate. As discussed in Section 2, the plant flow rate has ranged from approximately 3.5 mgd during low demand periods to 11.6 mgd during a recent peak summer day.

Existing Hydraulic Profile

Figure 4-2 at the end of this section presents the hydraulic profile of the plant. The hydraulic profile was developed during the original design of the Winchester WTP for a maximum instantaneous flow of 18 mgd. Figure 4-1 has been updated, based on field tests conducted for this analysis, for a maximum flow of 22 mgd. The key hydraulic control features of the plant include:

- River water levels and intake pumping capacity
- Hydraulic capacity of the 30-inch raw water pipeline to the flocculation/sedimentation basin
- Hydraulic capacity of the overflow weir at the outlet of the flocculation/sedimentation basin
- Hydraulic capacity of the 42-inch settled water pipeline delivering water 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
- Hydraulic capacity of the overflow weir in the clearwell
- High service pumping capacity from the clearwell into the transmission system
- Finished water transmission pipeline capacity
- Backwash piping and pumping capacity
- Wastewater pump station and force main capacity
- Washwater and solids handling system capacity

Hydraulic capacity issues associated with each of these features are described in detail below.

Intake and Raw Water Pumping Capacity

The existing intake is equipped with 4 vertical turbine pumps, one with a capacity of 2,800 gpm (4.0 mgd) at 49 feet of total dynamic head (TDH) and three, each with a capacity of 4,200 gpm (6.0 mgd) at 49 feet of TDH. The smaller pump, Pump No. 1, has a 50 hp motor and the larger pumps, Pump Nos. 2, 3, and 4, have 75 hp motors.

All of the spaces provided for pumps are filled at the Winchester WTP intake. According to current planning and operating conventions within the water industry, firm pumping capacity is defined as the maximum flow achievable with one of the largest installed pumps out of service; therefore, the nominal firm capacity of the intake is approximately 16 mgd. Testing has demonstrated that firm capacity is actually less than the nominal value. On September 14, 2006 with Pump Nos. 1, 2 and 4 operating, the intake provided only 14 mgd. Testing of the station in 2002 determined that the firm capacity of the station at that time was approximately 15.0 mgd. The 2002 test results appear to indicate that Pump No. 3 is pumping substantially below its rated capacity; however, Pump No. 3 was not operating during the test on September 14, 2006 and the capacity was even lower. Further testing should be conducted to determine whether one or more of the pumps is failing to pump at its design capacity or whether low river and/or high dynamic head conditions impacted the flows. If it is the former, the malfunctioning pump or pumps should be replaced or repaired as part of the plant expansion. Before any future raw water pump testing, the entire intake should be cleaned.

The City recently installed a VFD on Pump No. 2. Ultimately, at least two pumps should have VFDs for increased reliability. Increased flow control through the installation of VFDs will allow for greater operational flexibility and treatment optimization in the future.

The intake structure has four gated openings to the North Umpqua River, each 4 feet square. There is a trash rack on the outside of structure in front of these openings. These openings are of adequate size to supply the intake pumps with up to 24 mgd or more of raw water. There are two traveling screens which screen the raw water before it enters the pump bay. These screens will pass up to 18 mgd or more of raw water. There are two 5 feet square gated openings between the traveling screens and the pump bay. These openings are of adequate size to supply the intake pumps with up to 24 mgd or more of raw water.

The maximum velocity in the intake outside the traveling screens occurs at the 4 feet square gated openings. As discussed in Section 3, because these openings are located outside the fish screens, the maximum approach velocity for protection of fish applies at these openings. With a maximum approach velocity of 0.4 fps and with the gates fully submerged, the maximum flow into each sluice gate is 4.14 mgd and the maximum flow into the intake with all four gates open is 16.5 mgd; therefore, although the intake structure has a hydraulic capacity up to 24 mgd, the structure as currently configured is limited to 16.5 mgd by current regulatory constraints. To achieve greater intake capacity, it will be necessary to locate fish screens outside these four gated openings. A discussion of options for improving the intake is presented in Section 6.

When the plant is expanded to 18 mgd, a second VFD should be installed on Pump No. 3 or 4. Pump No. 1, a 4 mgd pump, should be replaced with a 6 mgd pump, bringing the firm capacity to 18 mgd. New fish screens should be installed also.

If the plant is expanded beyond 18 mgd capacity, then replacement of some of the existing four 6 mgd pumps will be required to achieve higher firm capacities. Replacement of two 6 mgd pumps with two 8 mgd pumps will result in a firm capacity of 20 mgd. Replacement of three 6 mgd pumps with three 8 mgd pumps will result in a firm capacity of 22 mgd. Replacement of four 6 mgd pumps with four 8 mgd pumps will result in a firm capacity of 24 mgd.

Raw Water Pipeline Capacity to the Flocculation/Sedimentation Basin

The raw water pumps discharge into an underground 30-inch diameter steel raw water pipeline which delivers raw water to the flocculation/sedimentation basin, entering on the northeast corner of the structure. The steel pipe has a ¼-inch wall thickness. A 30-inch magnetic flow meter was installed on this pipeline in a below-grade vault in 2002, replacing a meter previously located at the inlet of the raw water pipeline into the rapid mix basin. The velocity, velocity head and head loss for the existing raw water pipeline at various flows are presented in Table 4-1.

TABLE 4-1
30-INCH RAW WATER PIPELINE VELOCITIES AND HEADLOSS

Plant Flow (mgd)	Velocity (fps)	$V^2/2g$ (ft)	Headloss (ft/100 ft)
8	2.6	0.116	0.05
12	3.9	0.24	0.11
18	5.9	0.54	0.25
20	6.5	0.66	0.32
24	7.8	0.90	0.44

As shown in the table, velocities through the 30-inch raw water pipeline exceed 6.0 fps at flows slightly over 18 mgd. The maximum desirable velocity in a transmission main of this type is typically 6.0 fps. The short length of the pipe coupled with the valve and fitting configuration of the pipe indicate that there should be minimal surge control concerns, even at the higher velocities that will exist above 18 mgd. The headloss associated with higher flow through the existing pipeline will ultimately raise the system total dynamic head (TDH), thus reducing the capacity of the raw water pump station. To account for this, replacing pumps with a higher pumping head may be necessary to compensate for this increased headloss. If the plant is expanded beyond 18 mgd, there will be an opportunity to adjust the design head as necessary at that time.

Rapid Mix and Flocculation/Sedimentation Basin

Raw water flows first into a rapid mix basin and then into the flocculation/sedimentation basin. There are two rapid mix basins in parallel, but currently only one rapid mixer is operated at any given time. The basin design provides for the addition of a second, parallel flocculation and sedimentation basin to increase the plant capacity to a nominal 18 mgd. Under those conditions, both rapid mix basins will be operated in parallel at the higher flows.

Section 2 discussed the design features of the plant's pretreatment system including rapid mix, flocculation and sedimentation. The single existing basin is rated at 12 mgd and was designed to be expanded with a parallel basin for an ultimate nominal treatment capacity of 18 mgd, but with approximately 24 mgd of hydraulic capacity. As previously discussed, this basin has been able to provide excellent pretreatment prior to filtration at sustained flows up to 11.5 mgd during the warm weather, low turbidity raw water conditions. Normal plant flows during high turbidity, colder water conditions have ranged from 3.5 to 5.0 mgd.

Settled solids from the basin are continuously removed by a chain-and-flight system. Settled water flows from the basin via the launders into a settled water channel located at the south end of the basin. The settled water then flows to the filters via a 42-inch pipeline.

The maximum water elevation in the basin is in the raw water influent channel. At 12 mgd, the maximum water surface elevation with one rapid mix basin operating and with the slide gates into and out of that basin fully open should be approximately 466.0 feet. There is a triangular launder weir in the sedimentation basin with invert elevation of approximately 465.83 feet. The bottoms of the 12-inch deep launder troughs in the sedimentation basin are approximately elevation 464.63 feet. The freeboard from the top of maximum water surface to the top of walls is approximately 1.5 feet which is adequate.

The basin has an overflow weir to accommodate unplanned filter influent valve shutdowns or raw water flows into the basin that exceed the filter capacity. The overflow weir is located in the settled water collection channel at the south end of the basin above a disposal channel drained by a pipeline. The overflow weir elevation is at 465.0 feet, approximately 1.0 feet higher than the normal filter operating level. The weir is approximately 9 feet long with a capacity of approximately 12 mgd, assuming a 9-inch water depth over the weir. The drain pipe is 30-inches in diameter, has a capacity of approximately 10 mgd, and delivers this flow to a diversion manhole with a weir. Under normal conditions sedimentation basin overflows are directed to the wastewater pump station, but if the pump station cannot handle the flow the water flows over the weir inside the diversion manhole and then to the river. A second overflow weir will be constructed along with the second flocculation/sedimentation basin when the plant is expanded; thus, the overflow system is adequate for an ultimate maximum plant capacity of 24 mgd.

The settled water pipeline to the filters is a 42-inch diameter steel pipe. At 18 mgd the pipeline velocity will be 2.9 fps and at 24 mgd the pipeline velocity would be 3.9 fps. Although these velocities are not excessive, this pipe could be the largest contributor to headloss at high flows. During a flow test conducted on September 14, 2006, the head loss through the pipe was measured at 2.8-inches at a flow rate of 13.1 mgd. Under these conditions, the water surface in the settled water channel was approximately 6-inches below the basin overflow weir. Depending upon where the new settled water pipe connects to the existing settled water pipe, the head loss at 24 mgd would be several inches greater. It will be necessary during design to carefully layout the yard piping to reduce the length of 42-inch pipe that carries the full plant flow thereby reducing headloss in the settled water pipe.

Through such careful design, it may be possible to avoid raising the overflow weir on the existing basin. The ultimate hydraulic capacity of the two basins in parallel is limited primarily by the settled water pipeline and that ultimate capacity is 22 to 24 mgd.

Filters, Filter Effluent and Filter-to-Waste Piping

Section 2 provides basic information on the design and performance of the filters. Settled water flows through the filter influent channel and enters each filter via a 20-inch pipe with isolation valve. The normal filter operating level ranges from 463 to 464 feet as controlled by the filter effluent flow control system. This water level provides 5 to 6 feet of submergence over the top of the filter media. A filter effluent modulating valve is used to maintain this water level in each filter. As headloss increases, the valve opens further. Each filter consists of two bays. The water flows down through the media, the support gravel and the underdrain of each filter bay. The water then flows into the filter piping gallery through 16-inch pipes that individually drain each filter bay and is combined into an 18-inch filter effluent pipe that is common to the two bays within a filter. The filtered water flows through the 18-inch effluent pipe, a propeller flow meter, the filter effluent modulating butterfly valve and then into the unbaffled clearwell area underneath the filters. Each filter empties directly into the clearwell through its own 18-inch effluent pipe, thus each filter's effluent enters the clearwell at a different location.

The normal maximum water level in the treated water storage area and clearwell is 449.0 feet, six-inches below the elevation of an overflow weir located beneath the high service pump station. The clearwell is normally operated between about 446.5 feet and 448 feet, which provides a total filter driving head of approximately 15 to 17.5 feet from filter water level to treated water storage level. The alarm signifying terminal filter headloss is currently set at 11.0 feet. Plans were made with the original design to add two more filters to expand capacity.

As discussed below, the current 3 mgd filter rating is a treatment capacity limitation rather than a hydraulic limitation. Hydraulically, the piping into and out of each basin can handle 4 mgd when the filter is filtering to the clearwell. With six filters, the total hydraulic capacity of the filters will be 24 mgd. Individual filter effluent flows into the clearwell are measured; however the requirements for straight-pipe both upstream and downstream of each filter effluent flow meter are not met, possibly reducing the accuracy of the meters. During a plant tour on September 14, 2006 the sum of the individual filter effluent meters was approximately 7 percent less than the flow determined by the more accurate magnetic flow meter measuring raw water flow.

The filter effluent piping arrangement currently prohibits the metering and modulating of filter-to-waste flows. Filter-to-waste flows can be estimated by calculating the difference between the raw water flow and the sum of the individual filter effluent flows. Based on this, it appears that the existing 8-inch filter-to-waste piping is too small to convey 3 mgd when filtering to waste. On a site visit to the plant on June 29, 2006, with filter No. 3 offline for cleaning and a raw water flow of 9.1 mgd, the flow through filter No. 1 in filter-to-waste was estimated by calculating the difference between the raw water flow and the combined filter effluent flows through filters No. 2 and No. 4. The filter-to-waste flow was approximately 2 mgd while the flow through the other two filters was greater than 3.4 mgd.

It is desirable to modulate filter-to-waste flow to maintain relatively constant flow through all filters at all times. Rapid increases in flow through the media induce shear forces on the retained floc which can cause premature breakthrough. Inducing rapid flow changes in a conventional filter when the filter is operating near its treatment capacity is particularly problematic. When filter-to-waste piping is unable to pass the design filter flow, then the flow through all other filters on line increases while one filter is in filter-to-waste. This problem is more pronounced with fewer filters, when the filters are operating near their design flow and when the difference between the filter design flow and the filter-to-waste flow is high.

In addition to the hydraulic capacity limitations of the filter-to-waste piping, the current location of the filter effluent sample point prohibits measurement of turbidity during filter-to-waste. For all the above stated reasons, changes in the filter effluent and filter-to-waste piping are desirable.

Options for this work vary depending upon the extent to which the City desires to monitor and control filter-to-waste. It may be possible to achieve more accurate effluent flow measurement by simply installing a different meter while using the existing piping configuration. In addition, relocating the sample point would enable measurement of filter-to-waste turbidity; thus, the lowest cost option would be to change the type of effluent meter and the location of the sample point without reconfiguring the filter effluent piping. This option would not provide for measurement and control of filter-to-waste flow and it would not provide for the increased piping size needed to pass more than 2 mgd during filter-to-waste. To provide measurement and control of filter-to-waste flow, and to allow for filter-to-waste at the same rate as filtering to the clearwell, it will be necessary to reconfigure the filter effluent and the filter-to-waste piping. Although this is the more expensive alternative, it is the one that brings the greatest benefit.

Given that the existing filter-to-waste piping does not appear to convey more than 2 mgd, any plan to increase the WTP capacity beyond 18 mgd may require reconfiguring the piping since the difference between the existing filter-to-waste flow and the higher filter design flow would be even more pronounced.

Clearwell

The clearwell and treated water storage at the Winchester WTP are comprised of three interconnected areas described as follows:

1. Unbaffled area under the four filters with a maximum storage volume of approximately 170,000 gallons at 10.5 foot water depth.

2. Baffled clearwell area underneath the chemical rooms and the blower room with a maximum storage volume of approximately 324,000 gallons at 10.5 foot water depth.

3. Unbaffled area underneath the high service pump room with a maximum storage volume of approximately 133,000 gallon at 10.5 foot water depth.

The total maximum treated water storage volume of 627,000 gallons represents approximately 75 minutes (1.26 hours) of storage at the current maximum plant flow of 12 mgd. This assumes that all the treated water in the clearwell is available as storage; however, in addition to providing finished water storage, the clearwell serves as the wetwell for the high service and backwash pumps and provides disinfection contact time. During a backwash, the level in the clearwell drops about 1.0 feet. To ensure disinfection contact time, a minimum level must be maintained, thus the actual treated water volume available as storage is less than the total clearwell volume. Disinfection contact time is a treatment unit operation, so it is discussed elsewhere in this section under Treatment Process Capacity Evaluation. In addition, a discussion of the clearwell's current ability to meet disinfection (CT) requirements at various flows is presented in Section 3.

There are two clearwell overflows, one located in the high service pump station and the other located below Filter No. 3. The high service pump station clearwell overflow weir is at elevation 449.5 feet with a weir length of approximately 9 feet. The overflow is located in the northwest corner of the high service pump station. At 1 foot of water depth over the weir, the overflow capacity is approximately 19 mgd. The overflow water is discharged into an overflow chamber with inside dimensions of 4.5 feet by 4.5 feet. The water then flows into an 18-inch diameter pipeline which directs emergency overflows to an outfall on the bank of the North Umpqua River. The overflow weir and piping is sufficient to handle clearwell overflows up to 19 mgd.

The clearwell overflow located below Filter No. 3 is at an elevation of 450.00 feet with a weir length of approximately 15 feet. This overflow discharges to the backwash waste channel which flows to the wastewater pump station and to the river if the station cannot handle the flow. The high service pump station clearwell will overflow first until the water level exceeds 450.00 feet, then water will overflow both weirs. Combined, the two overflow weirs can accommodate an ultimate plant flow of 24 mgd.

During current normal operating conditions, the high service pumps operate to maintain the clearwell level relatively constant between approximately 8 feet and 10 feet depth, which is 3 to 1 feet below the overflow weir. The level drops by about 1-feet when a filter is backwashed. One filter backwash uses 65,000 to 75,000 gallons of water with the higher backwash volume used during the summer. Since all four filters are backwashed once per day, approximately 260,000 to 300,000 gallons of finished water is used daily for backwashing. The clearwell level control is accomplished by manually throttling valves on the discharge piping of a raw water pump and a finished water pump to achieve somewhat stable operating conditions.

As discussed above in this section under the river intake and high service pump station paragraphs, the City is in the process of installing new variable-frequency drives (VFDs) on one raw water pump and one finished water pump to more accurately balance the flows to maintain a constant clearwell level while avoiding the need for valve throttling. These drives will also improve energy efficiency. The finished water pumping rate can then be decreased during backwashes to maintain the clearwell level as close to full as possible at all times.

It is generally considered desirable that a WTP maintain a minimum clearwell volume of approximately 60 minutes of detention time at the plant's peak flow rate. This criterion is currently being met at the 12 mgd plant capacity, although only if one assumes the entire clearwell volume is available as storage. When the plant is expanded with two additional filters, an additional 85,000 gallons of treated water storage will be added under Filters 5 and 6, bringing the maximum storage volume to 712,000 gallons. At the expanded plant capacity of 18 mgd, the maximum detention time would then be 57 minutes, which approximates the minimum desirable time. At plant capacities greater than 18 mgd, the treated water storage at the plant would be considered less than optimal. At 24 mgd using the 1-hour criteria, the suggested minimum clearwell volume is 1,000,000 gallons which is 288,000 gallons more than will be available when the ultimate buildout of six filters has been achieved. There is extremely limited space on the existing site to construct additional treated water storage. A detailed discussion regarding clearwell improvements for future expansion that may provide the ability to meet disinfection requirements is presented in the treatment capacity evaluation portion of this section.

High Service Pump Station

The station is equipped with 4 vertical turbine, high service pumps including:

- One pump (No. 1) rated at 1,400 gpm (2.0 mgd) at 305 feet TDH, with a 150 hp motor
- Three pumps (Nos. 2, 3 and 4) each rated at 2,800 gpm (4.0 mgd) at 305 feet TDH, with 300 hp motors.

All four of these constant speed pumps were installed in the Phase 2, High Service Pump Station phase, of the WTP project in 1987-1988. There is space provided in the pump station and on the discharge header for two additional pumps. The City recently installed a VFD on Pump No. 2. Ultimately, at least two pumps should have VFDs for increased reliability and greater operational flexibility. The current firm capacity of the high service pump station is nominally 10 mgd. Since the three largest pumps operate at approximately 3,300 gpm each under current hydraulic conditions, the current true firm capacity is about 11.5 mgd.

Table 4-2 summarizes the proposed high service pump capacities considering a plant expansion to 18 mgd as well as expansion scenarios up to 24 mgd. For expansion to 18 mgd, Pump Nos. 1 through 4 should remain. A new 6 mgd pump with a VFD drive and a new 4 mgd pump should be added in the current available spaces in the station.

TABLE 4-2
HIGH SERVICE PUMP CAPACITIES

Nominal Firm Pumping Capacity (mgd)	Pump No. 1 (mgd	Pump No. 2 (mgd)	Pump No. 3 (mgd)	Pump No. 4 (mgd)	Pump No. 5 (mgd)	Pump No. 6 (mgd)
10 (Current)	2	4 (VFD)	4	4	--	--
18	2	4 (VFD)	4	4	6 (VFD)	4
20	4	4 (VFD)	4	4	6 (VFD)	4
22	4	4 (VFD)	4	4	6 (VFD)	6
24	4	4 (VFD)	6	4	6 (VFD)	6

Finished Water Transmission Pipeline - WTP Site

The high service pumps located in the high service pump station discharge into a steel header located above the pump room floor. This header varies in diameter from 18-inch diameter at the east end to 30-inch diameter at the west end. This header exits the room at the west end above grade with a vertical bend that transitions the pipeline to a buried condition. The finished water transmission pipeline wall is 3/8-inch thick and the inside diameter is 29.25-inches. The buried finished water transmission main then continues westerly and southerly to its connection to two transmission mains located directly north of the Pioneer Way right-of-way. These two mains, which are discussed below, transmit finished water to the City's terminal reservoirs on Reservoir Hill. There is a 30-inch diameter propeller meter installed in a vault on the transmission main within the WTP site. This meter is reading low and should be replaced with a magnetic-type flow meter similar to the existing raw water meter. Pertinent design factors for the existing finished water transmission main are presented in Table 4-3.

While the maximum desirable velocity in a main of this type is approximately 6.0 fps, the short segment of the pipe allows this higher velocity to be acceptable for this portion of the facility. As shown in the table, velocities through the 30-inch raw water pipeline exceed 6.0 fps at approximately 18 mgd. The headloss associated with additional flow through the existing pipeline will ultimately raise the system TDH slightly, thus reducing slightly the capacity of the high service pump station. This small headloss increase is not significant within the total finished water transmission system headloss. Replacing pumps with a higher pumping head may be necessary to overcome the entire system headloss if an ultimate plant flow of greater than 18 mgd is to be provided, but not because of the small added headloss associated with this short transmission main segment.

TABLE 4-3
30-INCH FINISHED WATER TRANSMISSION MAIN VELOCITIES AND HEADLOSS

Plant Flow (mgd)	Velocity (fps)	$V^2/2g$ (ft)	Headloss (ft/100 ft)
8	2.65	0.109	0.05
12	3.98	0.246	0.11
18	5.97	0.553	0.25
20	6.63	0.682	0.32
24	7.96	0.984	0.44

There is a hydropneumatic surge tank connected to the finished water transmission main on the plant site. The 1,900 cubic feet (14,200-gallon) tank was designed to provide both upsurge and downsurge protection on the finished water transmission system between the plant and the terminal reservoirs on Reservoir Hill caused by power outages. The system performance criteria are as follows:

- Surge protection is provided for pumping rates up to 9,700 gpm (14.0 mgd)
- The maximum upsurge in the transmission pipeline between plant and terminal reservoirs is controlled to 150 psi.
- The minimum allowable downsurge at the high service pump station is controlled to an HGL of 630 feet (approximately 175 feet of head or 75 psi assuming plant ground elevation of 455 feet.)

With respect to surge protection, any expansion beyond the existing total pumping capacity will require replacing the existing tank with a larger tank. The addition of a second hydropneumatic surge tank is likely not practical due to space constraints at the site.

Finished Water Transmission Pipelines – WTP to Reservoir Hill

The 30-inch finished water transmission main from the plant connects to two transmission mains on the plant site. These mains are a 20-inch outside diameter steel main with a 3/16-inch thick wall and a 30-inch diameter Class 50 ductile iron main. These mains constitute the northerly portion of the City's transmission system to terminal reservoirs on Reservoir Hill. This system was the subject of a study entitled "Preliminary Engineering Study for Water Transmission System for City of Roseburg, Oregon, July, 1986". This study evaluated the hydraulics of the existing system, evaluated the condition of the existing pipelines, analyzed alternative sizes and routes for pipelines for improved transmission system capacity, recommended improvements and the phasing for those improvements, and prepared estimates of cost. The study provided key hydraulic information needed for the design of the treatment plant's finished water pumping system (pumps and piping) and the surge control facilities.

The study recommended improvements in two phases to achieve a hydraulic capacity of 18 mgd in the transmission main. Phase 1 consisted of construction of approximately 9,400 feet of new parallel transmission piping and demolition of an existing booster pumping station. This recommended work was completed by the City resulting in a two-pipe transmission system between the plant and a location near the intersection of Walnut Street and West Avenue. Phase 2 consisted of the easterly and southerly extension of a new 27-inch diameter transmission main approximately 1,600 feet long from the above intersection to the reservoir complex on Reservoir Hill. This main would parallel an existing 24-inch main. A development is being constructed over the routes of the existing 24-inch main and the proposed 27-inch parallel main. Based upon recent studies by the City and consultations with the project developer, it is anticipated that a new 30-inch main will be constructed initially as part of the development to replace the existing 27-inch main. A second parallel 30-inch main would be constructed in the future, thus completing the two-pipe transmission system between the plant and the terminal reservoirs. Further analysis currently being conducted as part of the updating of the City's water system master plan will determine if the second main should be completed prior to or concurrently with the plant capacity expansion to 18 mgd.

Further analysis and modeling outside the scope of this study will be needed to determine the ability of the existing pumps, plus additional pumps with the same TDH, to meet higher flow rates if the plant is expanded beyond 18 mgd capacity without increasing the capacity of the transmission system. If future transmission main capacity improvements beyond those recommended in the 1986 report were to be accomplished and the pumping head at peak flows were reduced, then additional pumping capacity could potentially be achieved without changing out pumps to higher capacities and pumping heads. It is recommended that the transmission main study be updated if ultimate plant design flows exceed 18 mgd.

Air Scour System

The WTP has an air scour system for filter backwash. The system is supplied with low pressure air from a multi-stage centrifugal blower. The blower has a capacity of 1,050 scfm at 6.4 psi and is driven by a 60 hp motor. Low pressure air is directed to each of the filters through an 8-inch diameter air supply system. Air flow is controlled by an electrically operated butterfly valve. Air flow control using a control valve on a relief vent may provide for more precise air flow control.

Backwash Pumping and Piping

The WTP is currently equipped with two constant-speed vertical turbine backwash pumps, each with a 50 hp motor, rated at 3,250 gpm at 41 feet TDH. Only one pump is required for backwashing each of the filter bays. The second pump serves as a spare/backup for emergencies. The backwash pumps discharge into a 16-inch diameter header that feeds backwash water to the individual filters. These pumps and the associated piping and valving are adequately sized for the existing filter configuration and filter type. The existing backwash control valve operator has been troublesome and may need to be replaced with an electric motorized valve operator.

If the filter media is changed in the future to deep bed filters to achieve production rates higher than 18 mgd, higher backwash rates than these pumps can achieve may be required. It is estimated that the optimum backwash rate for these filter bays outfitted with deep bed media (up to 1.1 mm effective size anthracite) would be approximately 4,400 gpm. When the pumps were tested on September 14, 2006 with the modulating butterfly valve at approximately 45° open, the pumps produced 4,060 gpm, so the existing pumps may be able to achieve acceptable backwash rates with deep bed media. It is also possible that the existing pumps may be nearing their expected lifetime by the time the need for deep bed media occurs and could therefore be replaced.

Filter backwash water from each filter discharges through a 16-inch backwash drain pipeline into the backwash conveyance channel under the filter gallery floor slab. From the channel, flow is conveyed via a 24-inch diameter pipeline to a drain manhole and then through a 30-inch pipeline to the wastewater pump station. The 24-inch diameter pipeline will be replaced with the addition of filters and can be upsized to 30-inch diameter. The new, larger pipeline will be capable of conveying a higher backwash flow in the future if a higher rate is needed to backwash a larger filter media. As part of the analysis to determine the optimum media size for production rates greater than 18 mgd, the hydraulic capacity of the downstream waste channel (as well as the wastewater pump station and solids handling capacities) should also be assessed.

Wastewater Pump Station

The plant’s wastewater pump station collects filter backwash and other plant drainage discharges and pumps them to the plant’s solids handling system. The pump station consists of a wetwell containing three 35 hp submersible solids handling pumps, each with a capacity of approximately 1,500 gpm at 40 feet of TDH. Each pump has a 10-inch diameter discharge with 10-inch isolation and check valves. A 16-inch diameter transfer main carries the wastewater to backwash basins 1 and 2. The approximate firm capacity of this station is 3,000 gpm with one of the pumps out of service.

Solids and Washwater Handling

Two new washwater and solids equalization/settling basins, plus the two new solids drying beds, were recently constructed by the City to replace the old lagoon and drying bed. Each basin has a maximum volume of approximately 300,000 gallons. Filter backwash, settled sludge and other plant drainage pumped from the wastewater pump station can be directed to either basin. A decanting structure is located in each basin. This structure allows decant to be discharged to the river. A solids removal pump station is located between the two basins. This station receives flow from each basin through a slotted drain pipe entering the station wetwell. Knife gates are installed on each of the two drain pipes at the wetwell. Two 2-hp submersible solids handling pumps discharge into a common 4-inch diameter header to transfer pumped flow to the drying beds.

Drying Bed No. 1 has an approximate total surface area of 9,500 square feet. Drying Bed No. 2 has an approximate total surface area of 10,000 square feet. The beds have concrete bottoms and sidewalls and slope from east to west to a channel located at the west end of the two beds. A decanting structure with stop logs for each bed is located in the channel at the divider wall between the two beds at the west end. This structure drains to the plant drainage system which flows to the wastewater pump station. Vehicle access ramps are provided to each bed at their east end. An equipment building is located at the northeast corner of Drying Bed No. 1.

The system was designed to handle the volumes generated by operation of the WTP at the plant’s design capacity of 18 mgd. It is likely that the system will not be capable of handling the solids generated at higher flows. After the system has been operational for some time it should be possible to evaluate the system’s performance and to project the system’s capacity to handle higher flows. If the newly completed system is determined to be inadequate for flows above 18 mgd, the need for additional land for a subsequent plant expansion will be greatly enhanced.

Summary of Hydraulic Capacity Evaluation

The plant has the hydraulic capacity to handle 18 mgd and may be capable, with appropriate modifications, of achieving a hydraulic capacity of up to 24 mgd. Unit processes that need modifications to achieve a hydraulic capacity of 18 mgd are:

1. Fish Screening at Intake: While the hydraulic capacity of the existing fish screen configuration is 18 mgd, the approach velocity through the slide gates to the existing screens will exceed regulatory requirements at flows above 16.5 mgd.

2. Raw Water Pumping: The low head pumps currently have a firm capacity of about 14 to 15 mgd. Replacement of the 4 mgd pump with a 6 mgd pump will provide firm capacity of 18 mgd. A VFD should be installed on the new 6 mgd pump.

3. Flocculation/Sedimentation: Hydraulically, the existing basin can handle flows somewhat greater than 12 mgd; however, as noted below in the discussion on treatment process capacity, a second basin will need to be added in parallel to the existing basin to treat flows greater than 12 mgd. When that is accomplished, the two basins will have a hydraulic capacity of 24 mgd.

4. Settled Water Transmission: The alignment and configuration of the pipeline from the second flocculation/sedimentation basin will need to be carefully evaluated in conjunction with the existing basin overflow weir elevation. The ultimate hydraulic capacity of the two sedimentation basins in parallel is limited by the settled water pipeline and that ultimate capacity is 22 to 24 mgd.

5. Filtration: Hydraulically, each of the existing filter basins can handle flows of 4 mgd when filtering to the clearwell. This is greater than the treatment capacity of each filter. With the addition of two more filters, the total hydraulic capacity of the six filters will be 24 mgd when filtering to the clearwell. In filter-to-waste, the hydraulic capacity is about 2 mgd. The two new filters should have a different piping design that provides for 4 mgd hydraulic capacity through the filter-to-waste piping. The piping for the existing four filters should be modified during the plant expansion to accommodate 4 mgd.

6. Clearwell: The volume of storage in the clearwell is adequate for existing conditions and for flows up to 18 mgd. As discussed below, the clearwell also provides treatment capacity in addition to storage. To the extent that the clearwell level must remain constant to achieve treatment for disinfection, the effective storage capacity of the clearwell is reduced.

7. Finished Water Pumping: The firm capacity of the existing high service pump station is nominally 10 mgd but the three largest pumps operate above their rated capacity at current plant capacity, so the true firm capacity is currently about 11.5 mgd. When the plant is expanded to 18 mgd, a new 4 mgd pump and a new 6 mgd pump should be installed in the two remaining open spaces. This will provide a firm capacity of 18 mgd. The new 6 mgd pump should be equipped with a VFD. The hydropneumatic surge control tank will need to be replaced with a larger tank to accommodate pumped flows greater than 14 mgd.

The unit processes that will need to be modified or replaced subsequently to achieve an ultimate plant capacity greater then 18 mgd are:

1. Fish Screening at Intake: The structure can be modified to accommodate flows up to 22 mgd when modifications are made to accommodate flows greater than 16.5 mgd.

2. Raw Water Pumping: If the plant is expanded beyond 18 mgd, replacement of some or all of the 6 mgd pumps with 8 mgd pumps will be required to achieve higher firm capacities.

3. Flocculation/Sedimentation: Construction of the second basin will provide hydraulic capacity for flows up to 24 mgd.

4. Settled Water Transmission: The alignment and configuration of the pipeline from the second flocculation/sedimentation basin will need to be carefully designed. Even with careful alignment, the settled water pipeline will likely limit the hydraulic capacity of the two sedimentation basins to 22 to 24 mgd.

5. Filtration: Each filter basin has the capacity to handle 4 mgd; thus with six filters, the hydraulic capacity will be 24 mgd. Hydraulically, the filter inlet channel appears capable of accommodating flows up to 24 mgd; however, as discussed below, the hydraulic capacity of 24 mgd exceeds the treatment capacity of the existing filter media. The filter media treatment capacity can be increased to about 22 mgd by changing the media. The backwash pumps appear capable of supplying the additional water that will be needed to clean the new media and the backwash waste piping appears capable of handling higher backwash flows; however, selection of the media size when the filter media is changed to increase treatment capacity should consider the capacity of the backwash waste system to accommodate the flow necessary to clean the media.

6. Clearwell: The volume of storage in the clearwell will be lower than industry standards for flows of 24 mgd. In addition, as discussed below, the clearwell also provides treatment capacity in addition to storage. To the extent that the clearwell level must remain constant to achieve treatment for disinfection, the effective storage capacity of the clearwell is further reduced.

7. Finished Water Pumping: If the plant is expanded beyond 18 mgd, replacement of some or all of the six high service pumps will be required to achieve higher firm capacities up to 24 mgd as shown in Table 4-2. The hydropneumatic surge control system as modified to accommodate flows greater than 14 mgd can be designed to handle flows up to 24 mgd. If ultimate plant design flows are expected to exceed 18 mgd, it is recommended that the transmission main study be updated.

8. Washwater and Solids Handling: Although the new equalization/settlings basins and drying beds are likely to have sufficient capacity to handle the washwater and solids generated by 18 mgd, it is unlikely that the system will have the capacity to handle flows higher than 18 mgd. The capacity of this system to handle higher WTP flows can only be evaluated after the system has operated for some time.

Treatment Process Capacity Evaluation

General

Each of the key plant processes was evaluated for its ability to meet current and possible future conditions based on the process's past proven performance and also on the basis of professional judgment gathered through design of new plants and plant expansions and from observations made at other operating plants.

Summary of Chemical Feed Systems

The primary chemical storage, metering and feed systems at the plant include:

- Liquid aluminum chlorhydrate (ACH) (50 percent solution) for primary coagulation
- On-site generated sodium hypochlorite (0.44 percent solution) for disinfection (pre- and post-chlorination)
- Dry polymer for filter aid
- Dry powdered activated carbon (PAC) for taste and odor control
- Hydrated lime or soda ash for pH adjustment

Only three of the systems (ACH, hypochlorite and polymer) are used continuously whenever the plant is in operation. Lime or soda ash addition has not been used for pH adjustment since ACH was introduced as the coagulant. The PAC system has not been used as the plant has not experienced raw water quality conditions that necessitated its use. The doses of each chemical vary depending on plant flow and raw water quality.

Liquid ACH

ACH is stored on the first floor of the plant in two 6,000 gallon fiberglass tanks for a total storage volume of 12,000 gallons. The tanks and metering pumps were previously used for liquid alum storage. The plant currently adds ACH to the raw water for primary coagulation prior to rapid mixing. The chemical metering system consists of two positive displacement diaphragm pumps, both rated at 20.8 gallons per hour (gph). The ACH feed is continuous using carrier water; carrier water flow rates are estimated at 14 gpm.

At the current maximum instantaneous plant flow of 12 mgd, the estimated maximum ACH usage rate is 500 pounds per day (ppd) at an ACH dose of 5 mg/L. This equates to a maximum chemical pumping rate of 3.8 gph using 5.55 pounds of ACH per gallon of solution. The pumping rate of 3.9 gph is well below the rated pumping capacity of the existing ACH feed pumps. Assuming a dose of 5 mg/L, the existing metering pump system is capable of reliably meeting plant demands up to 24 mgd or more if maximum ACH doses during peak summer flows remain similar. Replacement of existing metering pumps with larger capacity pumps will not be required to achieve reliable ACH feed capacity at flows in excess of 24 mgd.

Depending on location, access to deliveries, potential winter delivery outages and other factors, 15 to 30 days of chemical storage is typically recommended. The recommended storage is typically calculated at a maximum dosage and an average day demand. At the plant’s existing average daily demand of 5.0 mgd and a maximum ACH dose of approximately 8 mg/L, the 12,000 gallons of total ACH storage represents approximately 200 days storage. At a plant capacity of 24 mgd, there will be approximately 100 days storage.; thus, storage capacities at the plant are sufficient for the near term and for expansion up to 24 mgd.

Sodium Hypochlorite

Liquid sodium hypochlorite with a solution strength of approximately 0.44 percent is generated on site and is stored in two HDPE tanks, each with a capacity of 500 gallons for a total solution storage capacity of 1,000 gallons. The tanks are located inside the hypochlorite feed room, along with the hypochlorite generators and metering pumps. The 8,225 gallon salt/brine tank and the brine pumps which feed the generators are located outside in a space with containment, adjacent to the hypochlorite room.

There are currently three MIOX hypochlorite generators, each with a rated capacity of 100 ppd. There are three positive displacement motor driven diaphragm-metering pumps, two rated at 300 gph at 50 psi and one rated at 147 gph at 100 psi. Under normal operating conditions, one pump is dedicated for pre-disinfection (with injection into the rapid mix basin), the second for post-disinfection (with injection into the clearwell), and the third pump serves as backup. Space and a piping connection have been included for a future pump addition.

The combined pre- and post-filtration chlorine dose varies from a low of 1.8 mg/L to a high of 2.5 mg/L. Doses above 2.0 mg/L typically occur in the lower demand months from November through May. The combined dose during the high flow months is typically about 2.0 mg/L. At the current maximum instantaneous plant flow of 12 mgd and a combined pre- and post-filtration chlorine dose of 2.0 mg/L, the estimated hypochlorite usage is 200 ppd. The current total installed generator capacity is 300 ppd and the firm generator capacity, with one generator out of service, is 200 ppd, which provides firm capacity for 12 mgd. An additional generator or generators will need to be added for flows in excess of 12 mgd. For a plant capacity of 18 mgd, a fourth 100 ppd generator will be required. For capacities greater than 18 mgd and up to 24 mgd, a fifth 100 ppd generator will be required.

The 200 ppd current maximum chlorine usage at 12 mgd equates to a total liquid hypochlorite pumping rate of approximately 5,440 gallons per day or approximately 227 gph total. This is well below the existing firm pumping capacity of 447 gph. Assuming a dose of 2.0 mg/L, chlorine solution would need to be pumped at 455 gph to treat 24 mgd. This approximately equal to the existing firm capacity and there is space to add a fourth pump to increase the firm capacity well above that amount; thus, the existing pumping system is capable of reliably meeting plant demands up to 24 mgd.

The plant’s current hypochlorite solution storage volume of 1,000 gallons provides only about 4.4 hours of solution at 12 mgd with a dose of 2.0 mg/L. The on-site generator manufacturer recommends a solution storage volume sufficient for 24 hours at peak demand. To achieve 24 hours storage at the existing 200 pound per day peak chlorine demand would require about 5,400 gallons. The existing storage volume is low even for existing peak conditions. For peak demands of 18 mgd, the existing volume would only provide about three hours.

The existing three on-site generators each have independent power supplies. This fact mitigates somewhat the need for a full 24 hours of storage; however, the volume of storage should be expanded, either when the plant is expanded to 18 mgd or soon thereafter.

It may be difficult to locate sufficient space to store 5,400 gallons within the existing chlorine room. There is, however, ample space available elsewhere within the WTP. The feasibility of separating the storage tanks from the on-site generators will need to be coordinated with the generator manufacturer. The solution is transferred to the hypochlorite tanks directly from the electrolytic cells by water system pressure provided by dilution water that is mixed with the pumped brine. If higher pressure is needed to convey the fresh solution to more distant storage tanks, the manufacturer must be consulted to ensure that the pressure does not exceed the limitation of the electrolytic cells and other generator equipment.

Currently, the City adds 34 tons of salt to the brine tank about twice per year while treating an average of 5.1 mgd. Using a peaking factor of 2.0, when the peak day flows are 18 mgd, the tank will need to be refilled between three and four times per year. When the peak day flows are 24 mgd, the tank will need to be refilled between four and five times per year; thus, the brine tank appears sufficiently sized to handle up to 24 mgd. It may desirable to install a small brine tank capable of holding 500 to 1,000 pounds of salt so that hypochlorite can still be produced when the brine tank is taken down every few years for cleaning. This is particularly true as long as the solution storage volume remains less than the manufacturer's recommended 24 hours.

Polymer

The plant currently adds non-ionic polymer to the settled water as a filter aid to improve filter performance. A dry feed system, including a 26 ppd automatic polymer solution mixing/aging unit and feed tank and two diaphragm positive displacement metering pumps rated at 69 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. Maximum polymer doses range up to approximately 0.05 mg/L.

At the possible maximum future plant flow of 18 mgd, the estimated maximum polymer usage is 7.5 ppd, assuming a maximum polymer dose of 0.05 mg/L. At 24 mgd capacity, the estimated maximum polymer usage is 10.0 ppd. The existing polymer feed system and storage capacity has capacity for plant flows up to and beyond 24 mgd.

Lime/ Soda Ash

The Winchester WTP was originally equipped with two dry chemical feed systems for pH adjustment. Chemicals can be fed to the raw water to increase the pH/alkalinity for coagulation optimization and to the finished water to increase the pH/alkalinity for corrosion control purposes. The City originally fed soda ash to control pH. As described in Section 2, the City stopped feeding pH adjustment chemical in 1999 when it started using ACH as the primary coagulant. ACH does not depress the pH compared to alum, and therefore it was determined that finished water pH adjustment was no longer required to maintain the minimum pH of 7.2.

There are two identical volumetric feeders, each with a capacity of up to 3.0 cubic feet/hour. The feeders each have a 75-gallon solution tank and fixer. A hopper is installed above each feeder unit. An elevator is installed to provide for loading of each hopper from the dry chemical storage room on the plant's first floor. Since the City is not presently using this equipment and there is no indication at this time that the pH adjustment with lime or soda ash will be required, it is recommended that the equipment remain available for service but not be modified or increased in capacity at this time, regardless of the size of any plant expansion.

Powdered Activated Carbon (PAC)

The plant currently has a dry chemical feed system which is capable of adding PAC to the raw water for taste and odor (T&O) control. As described previously, this system has never been used since the raw water quality conditions have not been such as to require its use; however, for completeness and in case the City wants to use this chemical in the future, the capacity of the system is reviewed.

The dry feeder is a volumetric type with hopper that discharges to a mix tank with a flushing funnel. An eductor carries the resulting slurry/solution to the application point. Prior to application, the solution is further diluted. Dilution water is controlled by a solenoid valve. Dry PAC is shipped in 50-pound bags and stored in the PAC storage area.

The dry feeder and dissolution system is rated at a maximum PAC usage rate of 400 to 490 ppd (16.7 to 20.4 lbs/hr). At a future maximum plant flow of 18 mgd, the estimated maximum PAC dose usage is 3.2 mg/L. This dose may be able to help reduce some T&O, but is incapable of removing much, if any, geosmin or MIB as described in Section 2. Since the City has not experienced any raw water conditions that would necessitate the use of this equipment over the plant's history, it is recommended that the equipment remain available for service but not be modified or increased in capacity, regardless of the size of any plant expansion.

Coagulation Performance

The North Umpqua River water is generally considered a low turbidity/high quality supply, but some treatment challenges exist at the plant, 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 mild taste and odor events. Excepting taste and odor, this variable raw water quality can significantly impact coagulation performance at the plant.

Prior to 1999, these treatment challenges were met using a relatively high dosage of alum. This strategy resulted in relatively high solids production, depressed pH (corresponding to an increase in pH adjustment chemical usage/costs), and decreased overall plant efficiencies. In

1999, the plant switched from alum to ACH as discussed in Section 2. This switch has resulted in significantly lower coagulant doses, improved plant performance, and reduced operating costs. The plant has been able to successfully treat a wide range of water quality conditions at plant flows up to 11.5 mgd since 1999 using ACH and the City expects to continue successful performance with ACH in the future.

Based on this performance, it is assumed that the coagulation system at the Winchester WTP is optimized and does not need to be modified in the future. Coagulation does not appear to be a capacity-limiting process at the plant.

Rapid Mix and Flocculation/Sedimentation Basin

A summary of historical performance from the single basin is summarized in Section 2. The flocculation/sedimentation basin provides contact time for disinfection and solids removal prior to filtration. The flocculation/sedimentation basin is considered a high-rate pretreatment process due to the relatively low flocculation time and the high surface loading rate which was designed to minimize the basin’s footprint. The fact that the flocculation process provides three sequential stages with baffle walls between each stage improves performance and reduces short-circuiting potential. The tube settlers installed in the latter part of the sedimentation basin allow the basin to perform well under the range of water quality and flow conditions experienced at the plant since 1992. The conversion from alum to ACH in 1999 has improved pretreatment performance as well. The warm, low turbidity water in the summer allows good pretreatment during the peak production periods. The water temperature does not get extremely cold during the winter, and this has helped the pretreatment process perform well during the lower production periods, even with turbidities that have infrequently approached and exceeded 500 NTU.

The original plant design included space and features to allow the construction of a second parallel flocculation/sedimentation basin. With the addition of this second basin, the performance of the plant’s pretreatment system should improve compared to existing conditions at an expanded plant capacity of 18 mgd. With the second basin installed, at flows of 18 mgd the flocculation and sedimentation times will increase by at least 33 percent and the surface loading rate will decrease. Based on the single basin’s performance at flows up to 11.5 mgd during peak production periods in the summer, it is believed that the addition of a second parallel flocculation/sedimentation basin will provide adequate pretreatment for up to 24 mgd.

Filtration

Section 2 presents a detailed evaluation of historical filter performance and a discussion of the production inefficiencies due to short filter runs during the non-peak production periods. Filtered water quality has been excellent, due in part to the good pretreatment performance. The existing four filters have been able to perform well at flows up to 11.5 mgd during peak summer production periods.

The addition of two more filters with similar media design characteristics as the original four filters will be able to support an expanded 18 mgd capacity, since the nominal filtration rate of 5.0 gpm/sf will be the same as the existing filtration rate at 12 mgd and pretreatment will improve as noted above. In addition, the net filtration rate will decrease during backwashes compared to existing conditions (5 filters operating during backwash compared to 3 filters operating during backwash), so filter performance will be somewhat improved due to smaller flow changes to the filters.

The existing mixed media filter design is considered to be inadequate to support capacities in excess of 18 mgd due to its higher headloss characteristics at filtration rates in excess of 5.0 gpm/sf. An alternative filter media design will be necessary to treat flows greater than 18 mgd. With an alternative media configuration, the loading rate could be increased to about 6 gpm/sf, providing a total treatment capacity of 22 mgd for six filters. It is not considered feasible to increase treatment capacity to 24 mgd using granular media filtration since this would require loading rates of 6.7 gpm/sf. A discussion of alternative filtration improvements for increased capacity is presented in Section 6.

As noted above in the discussion on hydraulic capacity, the existing filter-to-waste piping has a hydraulic capacity of only 2 mgd. If the filter-to-waste piping is not reconfigured when the plant is expanded to 18 mgd, it will be necessary to reconfigure the piping when the media is changed to increase treatment capacity to 22 mgd.

Clearwell Contact Time

The clearwell was designed to provide sufficient contact time to meet disinfection requirements for flows up to 18 mgd under the regulatory framework that existed at the time it was constructed. At that time, credit for inactivation achieved in the sedimentation basin could be added to the inactivation achieved in the clearwell. As discussed in Section 3, the State subsequently changed its regulations to require at least 0.5-log of *Giardia* inactivation following filtration. As a result, the existing clearwell/treated water storage configuration coupled with current plant operating parameters such as water depth and chlorine residual limit the plant's ability to meet this requirement, even under current flow conditions as shown in Table 3-2. To meet disinfection requirements at an expanded capacity of 18 mgd, additional clearwell improvements are necessary. Achieving adequate disinfection within the available treated water contact volume appears to be infeasible at flows beyond 18 mgd.

Several potential improvements have been identified. One improvement is to increase the hydraulic efficiency of the existing baffled area from the assumed present value of 50 percent (used in Table 3-2) to approximately 70 percent by installing additional baffle walls. Another improvement is to increase the baffled volume when the two new filters are constructed. This can be done by routing the filtered water beneath the south filters to the west then to the north and baffling the area beneath the north filters to route the water from each north filter to the west. This would extend the baffled area to include half the area beneath the north filters. It would also eliminate the ability of the south filters to bypass the northwest section of existing baffled area.

In addition to improving the clearwell, changes in operating parameters will also be required to achieve adequate disinfection at 18 mgd. The required changes in operating parameters include maintaining a consistently higher clearwell level and maintaining a consistently higher chlorine residual. Table 4-4 estimates the maximum flow achievable during each month after the plant is expanded to 18 mgd. The data for minimum monthly temperature and maximum monthly pH are based on WTP data from 2003.

Table 4-4
MAXIMUM MONTHLY FLOWS MEETING CT REQUIREMENTS
AFTER PLANT EXPANSION AND CLEARWELL MODIFICATIONS

Month	Min Finished Water Temp[1] (ºC)	Max Finished Water pH[1]	Chlorine Residual (mg/L)	$CT_{Required}$ (mg/L*min)	Maximum Flow Achievable (mgd)
Jan	8.3	7.6	0.75	25.9	**10.7**
Feb	7.2	7.6	0.75	27.9	**9.9**
Mar	7.8	7.6	0.75	26.8	**10.3**
Apr	8.9	7.6	0.75	24.9	**11.1**
May	11.1	7.7	0.75	22.3	**12.4**
Jun	16.1	8.2	0.75	19.2	**14.4**
Jul	20.0	8.3	0.75	15.4	**18.0**
Aug	20.0	8.3	0.75	15.4	**18.0**
Sep	16.7	8.2	0.75	18.4	**15.0**
Oct	12.8	7.8	0.75	20.6	**13.4**
Nov	9.4	7.7	0.75	25.0	**11.1**
Dec	8.9	7.5	0.75	24.0	**11.5**

Notes:
[1] Min Temperature and Max pH based on data from 2003
Assumed depth in Clearwell = 10.0 Feet
Volume under HSPS = 126,204 Gallons
Volume under the filters 1 through 4 = 162,038 Gallons
[A] Volume of Existing Baffled Clearwell = 308,591 Gallons
[B] Bypassed Existing Clearwell Volume = 0 Gallons
[C] Baffled volume beneath north filters = 60,764 Gallons
(A - B + C) Active Volume for CT = 369,355 Gallons
Assumed Hydraulic Efficiency = 0.7 - ($T_{10}/T_{\text{theoretical}}$)
Total Treated Water Storage = 677,861 Gallons

The table assumes that the hydraulic efficiency of the baffled area is improved to 70 percent and that the clearwell is modified as described above. Under these conditions, maintaining a clearwell depth of 10.0 feet and a chlorine residual of 0.75 mg/L will meet the 0.5-log *Giardia* inactivation disinfection requirement at flows up to the maximum flow indicated. If the chlorine residual were increased, the clearwell level necessary to meet CT requirements would decrease. For example, at a chlorine residual of 0.84 mg/L, the clearwell depth needed to treat the flows shown in Table 4-4 would be 9.0 feet.

The City will be completing the required 24-month sampling and testing program for *Cryptosporidium* per the LT2ESWTR in the summer of 2009. Test results to date indicate that the North Umpqua River has very low concentrations of *Cryptosporidium*. In the highly unlikely event that this supply is determined to have excessive concentrations of *Cryptosporidium*, the LT2ESWTR may require other, non-chlorine based forms of disinfection, such as UV irradiation or ozone. These would result in significant plant modifications. A discussion of these alternative disinfection processes for possible compliance with future regulations is presented in Section 6.

The addition of VFDs to the raw water and finished water pumping systems will allow the clearwell water level to be maintained at a consistent and high level. Additional plant operations, such as reducing the finished water pH in the summer and early fall, could help reduce the chlorine residual or clearwell level needed to fully comply with disinfection requirements. The City could also consider use of a portion of the finished water transmission pipelines to meet CT.

Clearwell Storage Volume

As discussed previously in this section, the current treated water storage volume along with the addition of approximately 85,000 gallons under the new Filters 5 and 6 is considered adequate for plant capacities up to 18 mgd, however, the volume may be inadequate to support higher capacities. In addition, as noted above, the ability to meet CT after the plant expansion will be dependent upon maintaining a consistent, high level in the clearwell to ensure sufficient contact time; thus, the volume actually available as storage, given that a minimum level must be maintained for disinfection contact time, is much less than the "treated water storage volume" shown in the notes of Table 4-4. If the minimum clearwell level needed to provide disinfection contact time is 9.0 feet, then the effective storage volume available is only the volume between 9.0 feet and the overflow level; thus, the effective storage volume at the site is not even adequate for flows of 12 mgd let alone 18 mgd and higher. It should be noted that the plant operates to maintain appropriate water levels in the terminal storage reservoirs on Reservoir Hill. The plant is not relied upon for any distribution system storage capacity. The primary concern with respect to clearwell storage volume is meeting plant operational requirements such as for filter backwash operations. The potential to add more treated water storage volume on the existing site is extremely limited due to space constraints.

Disinfection By-Products

Existing DBP concentrations within the City's distribution system are relatively low and are compliant with current regulations. The source water TOC concentration is low and the plant's coagulation/sedimentation/filtration process removes a significant percentage of TOC, resulting in low chlorine demand and low DBP concentrations. It is expected that compliance with the Stage 2 D/DBP Rule will continue with the expanded plant, up to 24 mgd, while using similar treatment practices. An alternative disinfection scheme is not required for DBP compliance. Once plant improvements have been made to meet the disinfection requirements discussed above, the City should review its DBP concentrations and conduct another disinfection profile if required by the State.

Washwater and Solids Handling System

As noted above, the two new washwater and solids equalization/settling basins and the two new solids drying beds were recently constructed, thus no operational data are available to evaluate their performance. The basin design appears to be substantially in line with the plant's original design criteria which envisioned flows of up to 18 mgd treated with alum coagulant. Given that the plant now uses ACH coagulant which produces less sludge than does alum, there is no reason at present to anticipate that the system cannot handle the solids generated by treating flows up to 18 mgd, however, it is not yet clear what solids concentration will be achieved by the new system. This factor will determine how the solids will be handled and where they can be disposed. After the system has been operational for some time, it should be possible to evaluate the system's performance and to project the system's capacity to treat the solids generated by flows greater than 18 mgd. It is likely that the system may be unable to treat flows as high as 22 to 24 mgd. If that is true, then either additional space or alternative technology will be required to treat the solids generated by flows greater than 18 mgd.

Summary of Treatment Process Capacity Evaluation

The following is a summary of the treatment process capacity evaluation. The conclusions of this evaluation are:

1. Chemical Feed Systems: With the exception of the sodium hypochlorite system, the existing chemical systems are adequate without modification to serve the next 10 to 20 years for the expanded plant capacity to 18 mgd and up to 24 mgd. The three sodium hypochlorite generator provide firm generator capacity for the 12 mgd current capacity. A fourth 100 pound per day unit will be needed to ensure 300 ppd firm capacity for a plant expansion to 18 mgd. A fifth 100 ppd unit will be needed if the plant is expanded beyond 18 mgd and up to 24 mgd. Although it may not be possible or necessary to store a full 24 hours of hypochlorite solution under peak demand conditions, additional hypochlorite solution storage volume should be considered as part of the plant expansion since the current volume is less than one fifth the recommended storage. The addition of a small brine tank may be desirable in case the existing brine tank needs to be taken out of service.

2. Flocculation/Sedimentation: Constructing a second flocculation and sedimentation basin in parallel to the existing basin should achieve up to 24 mgd of pretreatment

capacity. Pretreatment performance will improve at 18 mgd capacity compared to the existing performance with the single basin. At 24 mgd, flocculation/sedimentation should approximate existing performance.

3. Filtration: Adding two new filters with the same mixed media configuration as the existing four filters will provide for treatment of up to 18 mgd. For treatment of flows in excess of 18 mgd, the existing mixed media configuration is not recommended. Use of a deep media design in the six filter basins that comprise the plant's ultimate build out would provide for a maximum treatment capacity for filtration up to 22 mgd.

4. Clearwell: Clearwell storage is insufficient for flows above 18 mgd. For flows up to 18 mgd, the storage volume may also be insufficient since the minimum water level needed for disinfection contact time requires that the majority of the treated water in the clearwell not be available as storage. The operation of the City's water distribution system does not rely on the plant to provide for system storage. The plant is currently pushing the limits regarding the ability to meet post-filtration CT requirements under existing seasonal flow and water quality conditions. Improvements to the clearwell such as baffling can be made to improve CT compliance, but there may still be challenges in meeting CT at 18 mgd. Use of a portion of the finished water transmission pipeline may help achieve sufficient CT for flows up 18 mgd. Alternative disinfection strategies may be needed to treat flows greater than 18 mgd.

5. Disinfection By-Products: DBP formation is low and is expected to remain low in the future. Alternative disinfection strategies are not required to meet future regulations.

6. Washwater and Solids Handling System: The new equalization/settlings basins and drying beds are likely to be adequate for treating the solids generated by flows up to 18 mgd. The effectiveness of this system for treating the solids generated by higher flows can only be evaluated after the system has operated for some time, but it is considered unlikely that the system will be able to treat the solids generated by flows greater than 18 mgd.

Conclusions and Recommendations

The ultimate plant capacity for the Winchester WTP will depend upon whichever is limiting among the following factors:

1. The extent to which the existing configuration and treatment unit operations can be economically upgraded to increase hydraulic and treatment capacities within the existing property.

2. The hydraulic or treatment capacity of alternative treatment unit operations that could be economically located within the existing property.

3. The economic feasibility of obtaining additional, adjoining property to ensure sufficient space for treatment of higher flows using either an expansion of existing technology or replacement of existing technology with alternative treatment technology.

4. The maximum withdrawal from the North Umpqua River that can be achieved combining the City's current water rights with any additional rights that the City may be able to secure on the North Umpqua River.

Typically, it is more economical to expand the capacity of an existing treatment facility than to construct a new facility on a site with no existing treatment infrastructure; therefore, it is desirable in the long term to expand the plant capacity to treat all the water from the North Umpqua for which the City has or can obtain rights. This is preferable to forfeiting any opportunity to obtain additional water rights because of capacity limitations at the Winchester facility.

Using the existing four filters and the two filters to be added during the upcoming expansion, the treatment capacity limitation using the current treatment technology is 22 mgd. This capacity limitation is based on filtration and assumes that the following improvements can be made to other treatment units:

1. The clearwell volume available for disinfection contact time can be increased to provide sufficient CT, or alternative disinfection technology is added when the plant is expanded from 18 to 22 mgd.

2. The recently constructed washwater and solids handling system is determined to be capable of handling the solids generated at 22 mgd or additional solids handling capacity is constructed. This can be done using additional space to expand the existing system or by replacing the existing system with alternative technology having a smaller footprint.

It is recommended that the City proceed to expand the plant to 18 mgd in the near term and to plan for an ultimate capacity of 22 mgd in the long term.

To ensure that there is maximum flexibility and economy in selecting the best technology to be employed for expansion of the plant beyond 18 mgd and to ensure that the City can address any new regulations that may be promulgated in the intervening years, it is recommended that the City purchase additional land adjoining the WTP property to the west for potential future plant expansion to 22 mgd and potentially beyond this capacity. The additional property could be utilized for some or all of the following processes:

- Additional clearwell volume for disinfectant contact time.
- Additional clearwell volume for treated water storage.
- Additional washwater and solids handling system facilities.

- Additional or modified process technologies driven by new regulations.
- Additional potential future unidentified treatment facilities for potential expansion beyond 22 mgd if additional water rights and/or raw water supplies can be acquired

Figure 4-3 at the end of this section shows the WTP and adjoining properties. Table 4-5 provides information regarding ownership, size and assessed value for the properties identified in Figure 4-3.

Table 4-5
SUMMARY INFORMATION ON PROPERTIES WEST OF WINCHESTER WTP

Tax Lot No.	Area, acres	Assessed Value - Land	Assessed Value –Improvements	Total Assessed Value	Owner
500	0.07	$3,350	$0	$3,350	Schumacher Investments, LLC
600	1.1	$52,635	$209,166	$261,801	Schumacher Investments, LLC
800	3.52	$168,432	$627,499	$795,931	Schumacher Investments, LLC
700, 801, 900	Tax lots for sewage collection system facilities.				Roseburg Urban Sanitary Authority

Assuming that an additional sludge drying bed and a 1.25 mg clearwell will be constructed on the property to be acquired, it is recommended that the easterly 200 feet of Tax Lot 800 be acquired. This portion of the tax lot is approximately 1.54 acres and it has an estimated total assessed value of approximately $350,000 based upon a direct proportion of areas to the total parcel valuation.

The ability of the City to protect its existing water rights and to acquire additional water rights and potentially other raw water supplies for treatment at the Winchester WTP site are not known at this time. The Long-Range Water Supply Plan recommends further actions and studies to address these questions. The City may desire to defer acquiring the additional property pending development of further information on water rights and other water supplies that will more fully support the need to acquire the additional property.

G:\09\1015\401\CAD\09-1015-401-OR-FIG 4-X.dwg FIGURE 4-1 6/23/09 15:04 (DAK)

Figure 4-1

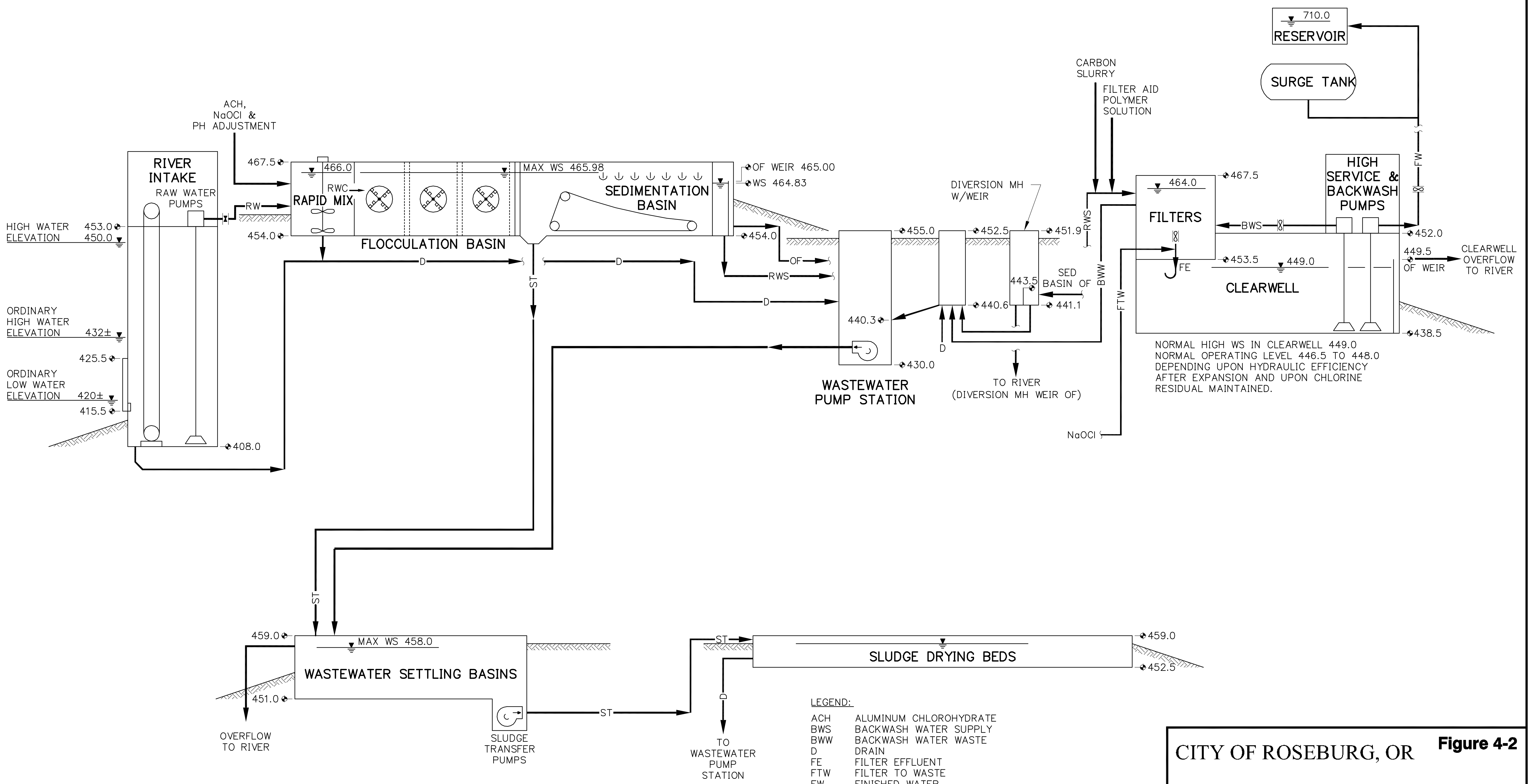
CITY OF ROSEBURG, OR

WATER TREATMENT FACILITIES PRELIMINARY DESIGN REPORT

EXISTING WINCHESTER WTP SITE PLAN

JULY 2009

Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 PHONE 503-225-9010
Portland, Oregon 97204 FAX 503-225-9022



LEGEND:

ACH	ALUMINUM CHLOROHYDRATE
BWS	BACKWASH WATER SUPPLY
BWW	BACKWASH WATER WASTE
D	DRAIN
FE	FILTER EFFLUENT
FTW	FILTER TO WASTE
FW	FINISHED WATER
NaOCl	SODIUM HYPOCHLORITE
RW	RAW WATER
RWC	RAW WATER COAGULATED
RWS	RAW WATER SETTLED
ST	SLUDGE TRANSFER
WS	WATER SURFACE
⧗	MAGNETIC FLOW METER
8	PROPELLER METER

NOTE:

1. ▨ SYMBOL REPRESENTS APPROXIMATE FINISH GRADE RELATIONSHIP.

2. BASED ON AN ULTIMATE PLANT FLOW OF 22 MGD.

CITY OF ROSEBURG, OR

Figure 4-2

WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

WINCHESTER WTP
HYDRAULIC PROFILE

JULY 2009

MSA Murray, Smith & Associates, Inc.
Engineers/Planners
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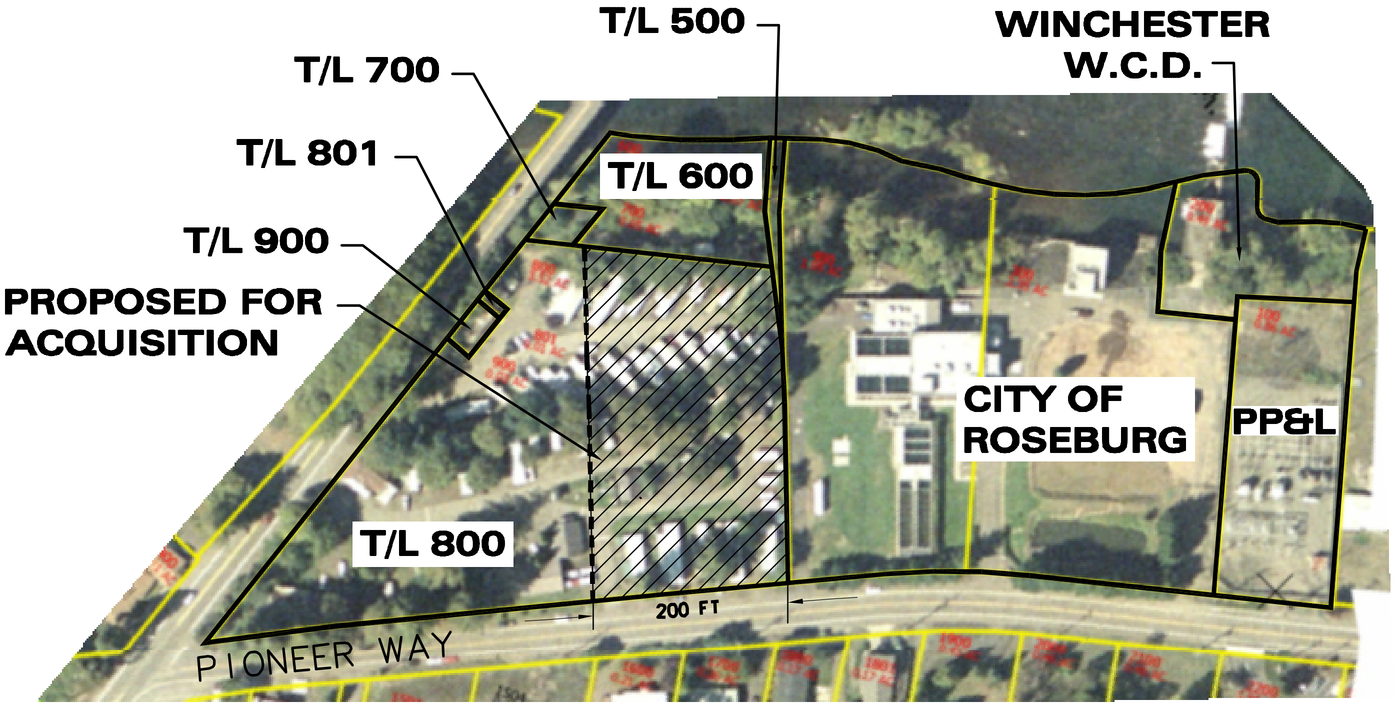
PHONE 503-225-9010 FAX 503-225-9022

G:\09\1015\401\CAD\09-1015-401-OR-FIG 4-X.dwg FIGURE 4-2 6/23/09 15:04 (DAK)



N

1 Inch = 200 Feet

CITY OF ROSEBURG, OR

Figure 4-3

WATER TREATMENT FACILITIES PRELIMINARY DESIGN REPORT

WINCHESTER WTP AND ADJOINING PROPERTIES

JULY 2009

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G:\09\1015\401\CAD\09-1015-401-OR-FIG 4-3.dwg 4-3 6/24/09 08:33 (DAK)

SECTION 5

General

Each of the Winchester Water Treatment Plant's major equipment, systems and structures were observed to determine their existing condition and to determine if replacement, upgrading or other improvements are required presently. This facilities condition review was conducted to assist the City in making decisions regarding maintenance and equipment replacement and upgrading requirements for all process and support facilities at the plant.

An estimate of the useful life of each item of major equipment, system or structure is made and the approximate estimated remaining useful life calculated based upon the year of installation or construction. Equipment, systems or structures which have a remaining estimated useful life of 20 years or less are noted. Recommendations are then made as to upgrading or replacement of equipment, system and structures which have less than 20 years of remaining useful life. Budget costs are provided for those items. This work constitutes a capital maintenance plan for the Winchester Water Treatment Plant. Detailed information on the hydraulic and treatment process capacity of the equipment, systems and structures is presented in Section 4.

Plant Equipment and Facilities Inventory

General

Table 5-1 contains an inventory of major plant equipment and facilities. This inventory consists of the following major categories of facilities as listed below:

- River intake
- Rapid mix basin
- Flocculation basin
- Sedimentation basin
- Filters
- High service pump station
- Flow meters
- Operations and control building
- Chemical feed systems
- Wastewater pump station
- Solids handling system
- Monitoring and control systems
- Site access control and security systems
- Yard piping and valving
- Site development

The inventory notes the year of construction or installation, the current age, the estimated useful life of the equipment or facility, and the approximate remaining useful life. The equipment or facility is then described in terms of type, capacity or size, manufacturer and manufacturer's data, condition, and performance. There may be specific comments related to each item. Recommendations are then made as to any current work that should be accomplished with respect to the inventoried equipment, systems or structures and an estimated project cost is provided. For items with a useful remaining life of less than 20 years, an estimated upgrading or replacement cost is provided.

Plant Equipment and Facilities Condition Assessment

General

The following is a general discussion of pertinent information on each major item of equipment, system or structure indicated in the above-described inventory and assessment. The discussion also includes observations and assessments made to determine remaining useful life associated with the plant equipment and facilities.

River Intake

The river intake was constructed in 1986 and 1987 as the first phase of the water treatment plant replacement program and replaced an older intake located directly upstream. The installed equipment is in good condition. The structure appears sound. The roof appears to be leaking and should be replaced immediately.

There have been some failures and repairs required on the vertical turbine intake pumps. Shafts have broken on two of the original pumps and the shaft bearings have been replaced on two of the pumps. Periodic maintenance of vertical turbine pumps can be expected although the shaft breakage is unusual. It is thought that silt buildup in the pump well collapsed and plugged the pumps, resulting in broken shafts. Small gravels may also pass through the gap between the traveling screen frames and the screens. The pumps are currently functioning well. Some plant capacity tests conducted in recent years indicate that possibly one or more of the pumps are not pumping to their design capacity. The pumps should be tested in a systematic manner for performance and those that are found to be pumping below their design capacity should be rebuilt.

The operators have had some difficulty obtaining spare parts for the two traveling screens, which are inherently a relatively high maintenance type of equipment.

There is shoaling, the accumulation of a gravel and sand bar, in front of the intake. As this process continues, there is the potential that the shoaling could restrict flow into the intake.

Rapid Mix, Flocculation and Sedimentation Basins

The rapid mix, flocculation and sedimentation basins, which are comprised of one structure, were built in 1989 during the Phase 3 Rapid Mix, Flocculation and Sedimentation Basins project. The facilities and installed equipment are in good condition.

There is some cracking in the vertical walls of the flocculation and sedimentation basins resulting in leakage through the wall. This is causing an unsightly appearance on the exterior basin walls and the exterior basin stucco finish. The cracks need to be repaired from the inside of the basin. Because there is only one flocculation and sedimentation basin, there has been no opportunity to dewater the basin for an extended period to conduct crack repair from the interior. With construction of the second flocculation/sedimentation basin, the existing basin can be dewatered and the work accomplished.

The tube settlers in the sedimentation basin have reached the end of their useful life. The supporting structures appear to be in good condition and should not need to be replaced. The wear surfaces on the sludge collectors should be replaced.

Filters

The four mixed-media gravity filters were built as part of the Phase 4 Filters and Operations Building project in 1992. The filter underdrain and support gravel system were rebuilt and the filter media was replaced in 1997. Based on discussions with plant staff and visual observations during plant visits, all of the filters are operating properly. A well-designed and properly maintained granular media filter system should have a useful life of 15 to 25 years before replacement is required. No special filter analyses were conducted as part of this plant evaluation.

The filter structures are in good condition. The existing backwash troughs show signs of deterioration due to wear and tear and exposure to sunlight. Personnel access from the filter level to the roof of the adjacent high service pump station needs to be upgraded.

The filter gallery piping is in good condition. As noted in Section 4, improvements to the existing filter-to-waste piping to provide for measurement and control of filter-to-waste flows and providing filter-to-waste capacity equal to that of the filters’ rated capacity are recommended.

The air scour system is in good condition. It is recommended that the air flow control valve be relocated to the air waste line to provide for improved air flow control.

The hydraulically-operated filter valve actuators are those originally installed in the plant and the cylinders appear to function well with the operators reporting no serious problems maintaining the actuators. The hydraulic actuator on the backwash control valve has been troublesome for years and has been replaced twice. Each time the cylinder was replaced, it was replaced with a similar cylinder to the one initially installed. Both the first and second cylinders functioned well for a period of time before developing problems. The third cylinder, recently installed, is working well thus far. The backwash control valve was recently replaced and is functioning well. The existing backwash flow meter is a propeller-type meter that has not been calibrated since it was installed.

High Service Pump Station

The high service pump station, finished water transmission main and finished water meter were built in 1988 during the Phase 2 High Service Pump Station project. The facility and equipment are in good condition. The roof is ready for replacement and this could be done when the roof on the intake structure is replaced.

Three of the four high service pump motors failed and were rewound between 1989 and 1996. The pump bearings on Pump No. 1 were repaired in 1997. While the motor failures are more frequent than would be expected, the pump bearing failure is not an unusual occurrence. The pump control valve for Pump No. 1 was rebuilt due to failed seals. Recently, Pump No. 3 has developed an erratic motor tripping condition.

The finished water flow meter is a propeller-type meter installed in 1992. As noted in Section 2, the meter readings have a negative bias of about 6% to 7% when compared with the newer and more accurate magnetic-type meter that measures raw water flow. The hydropneumatic surge control system's automatic air control system is not functioning properly. The clearwell vent screen shows signs of corrosion.

Operations and Control Building

The operations and control building was built in 1992 during the Phase 4 Filters and Operations Building project. The building and the associated equipment and facilities are in good condition. The roof was recently replaced. The elevator controls are apparently troublesome and should be replaced.

Electrical conduits and a junction box beneath the settled water pipe in south corner of dry chemical loading and storage room show signs of corrosion. This is likely due to a corrosive atmosphere emanating from the opening in the nearby clear well access hatch combined with insufficient air circulation in this corner of the room.

Chemical Feed Systems

The plant's five chemical feed systems are in good condition.

Wastewater Pump Station

The wastewater pump station was constructed in 1992 during the Phase 4 Filters and Operations Building project. The facility and associated equipment are in good condition. There are three 8-inch non-clog submersible pumps installed in a common pump sump. All three submersible non-clog pumps were rebuilt in 2002.

Solids Handling System

The solids handling system consists of two settling basins, two sludge drying beds and an equipment building. The system was constructed in 2006 to replace an older system and was put into service in the fall of 2006.

Monitoring and Control Systems

The plant has a supervisory control and data acquisition (SCADA) system. A Windows-based human-machine interface (HMI) using proprietary software provides the primary control, data display, and data logging via personal computer while select data is simultaneously displayed and recorded on LCD’s and chart recorders mounted on the east wall of the control room. The existing control systems were installed as part of the SCADA implementation in 1992 and should have significant remaining useful life.

In general, the plant’s monitoring and control systems are in good condition, although some components are ready for replacement with more current models. The chlorine residual analyzer was replaced in 2006 with a current model, the Hach CL 17. The turbidimeters have reached the end of their useful lives and newer models are available. The turbidimeters should be replaced soon and independent of the plant expansion program.

A technical memorandum prepared by the City’s systems integrator for the plant, S&B, Inc., is included in Appendix B. This report provides background on the existing instrumentation and control system, addresses the suitability of the existing systems for an expanded plant, considers advancements in control system technology, and provides recommendations for control system improvements, both to update the existing system and provide for expanded treatment capacity.

Site Access Control and Security System

The plant has site access control systems and a security system. The system appears to function well, although there are a few blind spots in the video surveillance system which could be eliminated with additional cameras. A comprehensive plant security assessment has previously been performed on the existing system.

Yard Piping and Valving

The yard piping and valving is understood to be in good condition.

Site Development

The site development features at the plant such as paving, landscaping, fencing, signage and other features appear to be in good condition.

Plant Equipment and Facilities Upgrading and Replacement Program

General

This section presents recommendations for capital maintenance improvements associated with the plant equipment and facilities. The recommended work is proposed to be performed either as part of the plant expansion project or separately as independent work.

River Intake

It is recommended that the City proceed with a project to evaluate the shoaling condition in front of the intake and, depending upon that evaluation, possibly proceed to remove this material. The evaluation should consist of an analysis of the river hydraulics in the area to determine whether the shoaling might increase and adversely impact the intake or whether it is a condition that does not need to be addressed. A hydrographic survey may be needed in order to perform this evaluation. The evaluation should identify the permitting requirements and develop a proposed project schedule and budget if the recommendation is to proceed with removal of the material. Permitting is required to conduct such operations within the river. The window during which removal operations may be conducted in the North Umpqua River is from July 1 through August 31. This evaluation should proceed independent of the plant expansion program.

As noted in Section 4, the City is currently undertaking the installation of a variable frequency drive (VFD) on one of the raw water pumps to provide for improved flow control and balancing through the plant. Installation of a second VFD is recommended to provide for reliability. This work should be included in the plant expansion program.

The existing firm capacity of the intake pumps has been demonstrated to be 14 to 15 mgd. As noted in Section 4, there are data indicating that at least one of the pumps is not functioning at capacity. All the pumps should be tested in a systematic manner for performance, and those that are found to be pumping below their design capacity should be rebuilt. This work should proceed independent of the plant expansion program.

Rapid Mix, Flocculation and Sedimentation Basins

The cracks in the sedimentation basin walls should be sealed from the inner wall surfaces with epoxy injection, a "strip and seal" system, or other appropriate means of repair once the new basin becomes operational. The exposed exterior stucco surface of the basin can be repaired after the cracks are sealed. Replacement of the wear surface on the sludge collectors can also be done after the 2nd flocculation and sedimentation basin is constructed. This work should be included in the plant expansion program.

The existing plastic tube settlers need to be replaced; however, they can continue to function for a few more years until the second basin is constructed. This work should be included in the plant expansion program.

Filters

With older filters and/or with filters that have apparent problems, it is common practice to perform detailed filter investigations. A special filter system analysis is recommended to be performed as described in Section 2. The City should perform these tests on the existing filters within the next 1 to 2 years to determine the remaining useful life of the filter media and to determine if improvements should be made to the backwash procedures.
This work should be included in the plant expansion program.

It is recommended that the existing backwash troughs be replaced. Personnel access from the filter level to the roof of the adjacent high service pump station should be upgraded. This work should be included in the plant expansion program.

Changes to the filter effluent and filter-to-waste piping are desirable. Options for this work vary depending upon the extent to which the City desires to monitor and control filter-to-waste. It may be possible to achieve more accurate effluent flow measurement by simply installing a different meter while using the existing piping configuration. In addition, relocating the filter effluent sample point would enable measurement of filter-to-waste turbidity. The lowest cost option would be to change the type of effluent meter and the location of the sample point without reconfiguring the filter effluent piping. This option would not provide for measurement and control of filter-to-waste flow as discussed in Section 4. To provide measurement and control of filter-to-waste flow, it will be necessary to reconfigure the filter effluent and the filter-to-waste piping; therefore, replacement of the filter effluent flow meters is recommended along with piping changes to integrate filter-to-waste flow measurement and control, all as described in Section 4. This work should be included in the plant expansion program.

It is recommended that the City investigate whether the backwash control valve cylinder, which has been replaced twice as noted above, is undersized for this service. The sizing of the operator is performed by the valve manufacturer. If it is undersized, then a larger cylinder should be installed before the newest cylinder develops problems. This work should proceed independent of the plant expansion program.

The existing propeller-type backwash flow meter should be replaced with a magnetic-type meter, consistent with the existing raw water meter and the proposed finished water meter. This work should be included in the plant expansion program.

The air scour system control valve should be relocated to more accurately control air flow. This work should be included in the plant expansion program.

High Service Pump Station

As discussed in Section 4, the City is currently undertaking installation of a VFD on one of the finished water pumps to provide for improved flow control and balancing through the plant. Installation of a second VFD is recommended to provide for reliability. This work should be included in the plant expansion program.

The hydropneumatic surge control system's automatic air control system needs to be upgraded or replaced. The corroded clearwell vent screen should be replaced. This work should be included in the plant expansion program.

Installation of a safer personnel access to the top of the high service pump station building is recommended for necessary building maintenance work. The alternatives for this access include installing a walkway from the top of the filters building or installing a permanent ladder. This work should be included in the plant expansion program.

The finished water propeller flow meter should be replaced with a magnetic-type flow meter. This work should be included in the plant expansion program.

It is recommended that roof replacement proceed as soon as possible and independent of the plant expansion project.

Operations and Control Building

Air circulation in the south corner of the dry chemical loading and storage room should be improved to reduce the extent to which the corrosive atmosphere emanating from the clearwell access hatch corrodes nearby electrical equipment. An exhaust fan should be installed in the wall, low and near the access hatch. The exhaust should be conveyed in a duct to the east, away from sedimentation basin structure. This work can proceed independent of the plant expansion program.

Monitoring and Control Systems

The existing turbidimeters appear to be nearing the end of their service life and have been superseded by newer versions. It is recommended that all of the existing turbidimeters be replaced using the same manufacturer and current model to simplify maintenance and reduce the amount of repair parts stocked. The plant's six turbidimeters and the streaming current monitor are recommended for replacement within the next year. This work should proceed independent of the plant expansion program.

The upgrading and expansion of the plant's existing instrumentation and control system as described in the S&B, Inc. technical memorandum in Appendix B should be included in the plant expansion program. When these changes are made, the data logging system should be modified to allow for direct printing of trending graphs for much of the data that are currently transferred only as data files. This would reduce the amount of data manipulation that operators must perform to prepare trending graphs.

As new systems and equipment are added to the plant, the SCADA system will need to be periodically modified and integrated accordingly. As technology evolves, the SCADA system at the plant will likely require additional upgrading; therefore, periodic hardware and software replacements will be needed to stay current with developing technology. These improvements and upgrades should be made using operating budget investments at the appropriate time.

Site Access Control and Security System

The video surveillance system could be improved by adding two new cameras, one on the high service pump station to look back at the main building and one on the low service pump station to look at the river and to provide an additional angle on the main building. A video data recorder would also be a useful addition to the system. This work should proceed independent of the plant expansion program.

As noted above, a comprehensive plant security assessment has been performed on the existing system. However, a new vulnerability assessment would be appropriate as part of the plant expansion project.

Summary and Recommendations

Based upon this facilities condition review, the following actions are recommended to be taken prior to and independent of the proposed plant expansion:

- Replace the roof of the high service pump station.
- Undertake and complete an evaluation of gravel bar accumulated in front of the intake and prepare a report including an action plan for removal of the material if that is deemed necessary.
- Test all four raw water pumps and rebuild those found to be pumping significantly below their design capacity.
- Diagnose and repair motor tripping condition on high service pump no. 3.
- Install a vent fan near the clearwell access hatch in the chemical storage room to eliminate the corrosive atmosphere that exists in the hatch area.
- Upgrade elevator control system.
- Replace the plant's six turbidimeters.

The cost of the above work to be completed independent of the proposed plant expansion project is included in the capital maintenance plan implementation plan that follows in this Section 5.

The following actions are recommended to be taken as part of the proposed plant expansion:

- Add a second VFD at the intake structure and at the high service pump station.
- Replace all wear surfaces on sludge collectors.
- Repair cracks in the sedimentation basin walls.
- Replace the existing plastic tube settlers in the existing sedimentation basin.
- Replace the existing filter backwash troughs.
- Perform detailed filter media evaluation testing program.

- Modify the filter-to-waste piping
- Upgrade personnel access from the filter level to the roof of the adjacent high service pump station.
- Replace the existing propeller-type finished water flow meter with a magnetic-type flow meter.
- Relocate the air flow control valve to the air waste line.
- Replace corroded clearwell vent screen in high service pump station room.
- Upgrade or replace the hydropneumatic surge control system’s automatic air control system.
- Replace the existing propeller-type backwash flow meter with a magnetic-type flow meter.
- Perform plant security assessment and upgrade systems as recommended.
- Replace existing filter effluent flow meters with magnetic flow meters.
- Upgrade the plant SCADA system.

The cost of the above work to be included in the proposed plant expansion project is included in the implementation plan and project cost estimates presented in Section 7.

Capital Maintenance Plan

Table 5-2 presents a recommended capital maintenance plan for the water treatment plant from the present through the year 2027. The information presented in the table can be used as a guide to budgeting for equipment and structure upgrading, improvements and replacements over the next 20 years.

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
A. River Intake												
A-1	Trash Rack No. 1 (East)	1987	19	50	31	Fabricated steel	10' X 10'		Good	Satisfactory		None
A-2	Trash Rack No.2 (West)	1987	19	50	31	Fabricated steel	10' X 10'		Good	Satisfactory		None
A-3	Sluice Gate No.1 (East)	1987	19	60	41	Cast iron	4' X 4'	Fidelity Environmental Equipment Co. (FEE Co.)	Good	Satisfactory		None
A-4	Sluice Gate No. 2	1987	19	60	41	Cast iron	4' X 4'	FEE Co.	Good	Satisfactory		None
A-5	Sluice Gate No. 3	1987	19	60	41	Cast iron	4' X 4'	FEE Co.	Good	Satisfactory		None
A-6	Sluice Gate No. 4 (West)	1987	19	60	41	Cast iron	4' X 4'	FEE Co.	Good	Satisfactory		None
A-7	Eductor No. 1 (East)	1987	19	60	41	Water jet sand and mud eductor	5-inch, 43ft. discharge head, 350 gpm min. discharge flow	AMETEK, Inc. / Model 224	Good	Operators report of plugging with sticks		
A-8	Eductor No. 2 (West)	1987	19	60	41	Water jet sand and mud eductor	5-inch, 43ft discharge head, 350 gpm min. discharge flow	AMETEK, Inc. / Model 224	Good	Operators report of plugging with sticks	Plugged up, thus not able to remove water to check. The main problem is with the downstream Eductor No. 2, where big sticks enter this eductor. Has plugged twice.	
A-9	Traveling Screen No. 1 (East)	1987	19	50	31	Traveling water screen, continuous basket	9 mgd @ 8ft water depth, 1.5 hp, 7' wide, 1/8-inch screen openings	Jeffrey Dresser, Dresser Industries, Inc.	Fair	Satisfactory	Problem with the solenoid activated water supply valve. Yearly failures with the valve (Magnatrol, type MOF44A39, 120VAC solenoid).	Difficulty in obtaining repair parts. Remove traveling screens in coordination with future installation of new fish screens on face of intake structure.
A-10	Traveling Screen No. 2 (West)	1987	19	50	31	Traveling water screen, continuous basket	10 mgd @ 8ft water depth, 1.5 hp, 7' wide, 1/8-inch screen openings	Jeffrey Dresser, Dresser Industries, Inc.	Fair	Satisfactory	Out of service at the time of site visit on 6/29/06 due to shear pins issue. Problem with the solenoid activated water supply valve. Yearly failures with the valve (Magnatrol, type MOF44A39, 120VAC solenoid).	Difficulty in obtaining repair parts. Remove traveling screens in coordination with future installation of new fish screens on face of intake structure.
A-11	Sluice Gate No. 5 (East)	1987	19	60	41	Cast iron	5' X 5'	FEE Co.	Good	Satisfactory		None
A-12	Sluice Gate No. 6 (West)	1987	19	60	41	Cast iron	5' X 5'	FEE Co.	Good	Satisfactory		None
A-13	Pump No. 1	1987	19	50	31	Vertical turbine, enclosed lineshaft	2,800 gpm @ 49 ft, 50 hp	Peerless Pump / Model 14HH-1 / U.S. Motor	Good	Satisfactory	Shaft bearings repaired in 1991.	Test pump performance. Rebuild pump if performance significantly below original design requirements.
A-14	Pump No. 2	1987	19	50	31	Vertical turbine, enclosed lineshaft	4,200 gpm @ 49 ft, 75 hp	Peerless Pump / Model 16HH-1 / U.S. Motor	Good	Satisfactory	Broken shaft repaired in 1992. City to install VFD on this pump soon.	Test pump performance. Rebuild pump if performance significantly below original design requirements.
A-15	Pump No. 3	1987	19	50	31	Vertical turbine, enclosed lineshaft	4,200 gpm @ 49 ft, 75 hp	Peerless Pump / Model 16HH-1 / U.S. Motor	Good	Satisfactory	Broken shaft repaired in 1990; shaft bearings repaired in 2002. Per pump testing in 2002, appears to be pumping substantially below its rated capacity.	Test pump performance. Rebuild pump if performance significantly below original design requirements.
A-16	Pump No. 4	2001	5	50	45	Vertical turbine, enclosed lineshaft	4,200 gpm @ 49 ft, 100 hp	Ingersoll-Dresser / Model 16F NH-1 / U.S. Motor	Good	Satisfactory		Test pump performance. Rebuild pump if performance significantly below original design requirements.
A-17	Pump Discharge Valves	1987 / 2001 (No. 4)	19 / 5	50	31	Swing check, butterfly & air release valves	16-inch (3-inch for air release)		Good	Satisfactory		None
A-18	Plant Water Pressure Reducing Valve	1987	19	50	31	Globe style	3-inch		Good	Satisfactory		None
A-19	Raw Water Flow Meter	2001	5	50	45	Magnetic meter	30-inch	Water Specialties / UltraMag	Good	Satisfactory	Pre-cast, concrete vault with sump pump	None
A-20	Structure	1987	19	75	56				Good	Satisfactory	Leakage in roof in SW corner.	Replace roof.

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
A-21	Heating and Ventilation Systems	1987	19	30	11				Good	Satisfactory		None
A-22	Electrical and Controls	1987	19	40	21				Good	Satisfactory		None
A-23	Miscellaneous Equipment	1987	19	Varies	Varies				Good	Satisfactory		None
A-24	River Deposition		19	20	1							Undertake evaluation of deposition in front of intake.
B. Rapid Mix Basin												
B-1	Bypass Stop Plate	1989	17	60	43	Aluminum	1/4" X 4'-9.5" X 6'-2"	Whipps	Good	Satisfactory		None
B-2	Slide Gate SG-1	1989	17	60	43	Aluminum	4' X 4'	Whipps	Good	Satisfactory		None
B-3	Slide Gate SG-2	1989	17	60	43	Aluminum	4' X 4'	Whipps	Good	Satisfactory		None
B-4	Rapid Mixer No. 1 (East)	1989	17	25	8	Vertical turbine type	84 rpm, 10 hp	Philadelphia Mixers / 3800 Series PTO	Good	Satisfactory	Motor replaced recently.	None
B-5	Rapid Mixer No. 2 (West)	1989	17	25	8	Vertical turbine type	84 rpm, 10 hp	Philadelphia Mixers / 3800 Series PTO	Good	Satisfactory	Motor replaced recently.	None
B-6	Slide Gate SG-3	1989	17	60	43	Aluminum	4'W X 5'H	Whipps	Good	Satisfactory		None
B-7	Slide Gate SG-4	1989	17	60	43	Aluminum	4'W X 5'H	Whipps	Good	Satisfactory		None
B-8	Slide Gate SG-5	1989	17	60	43	Aluminum	4'W X 5'H	Whipps	Good	Satisfactory		None
B-9	Flocculation Basin Inlet Stop Plates (6)	1989	17	60	43	Aluminum	1/4" X 1'-3.5" X 5'-2"	Whipps	Good	Satisfactory		None
B-10	Slide Gate SG-6	1989	17	60	43	Aluminum	3.5' X 3.5'	Whipps	Good	Satisfactory		None
B-11	Structure	1989	17	75	58				Good	Satisfactory		None
B-12	Electrical and Controls	1989	17	40	23				Good	Satisfactory		None
B-13	Miscellaneous Equipment	1989	17	Varies	Varies				Good	Satisfactory		None
B-14	Chemical Feed Lines	1989	17	75	58	Schedule 40 PVC	1"		Good	Satisfactory		None
C. Flocculation Basin												
C-1	Stage 1 Flocculator	1989	17	30	13	Horizontal paddle wheel	9'-9"DIA X 27.5'	GCR Manufacturing Co.	Good	Satisfactory		None
C-2	Flocculator Drive Unit No. 1 (North)	1989	17	25	8	Adjustable frequency drive	0.75 hp, 2 fps	Eurodrive, Inc.	Good	Satisfactory		None
C-3	Stage 1 Flocculator Baffle Wall	1989	17	30	13	Clear redwood			Good	Satisfactory		None
C-4	Stage 2 Flocculator	1989	17	30	13	Horizontal paddle wheel	9'-9"DIA X 27.5'	GCR Manufacturing Co.	Good	Satisfactory		None
C-5	Flocculator Drive Unit No. 2	1989	17	25	8	Adjustable frequency drive	0.75 hp, 2 fps	Eurodrive, Inc.	Good	Satisfactory		None
C-6	Stage 2 Flocculator Baffle Wall	1989	17	30	13	Clear redwood			Good	Satisfactory		None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
C-7	Stage 3 Flocculator (South)	1989	17	30	13	Horizontal paddle wheel	9'-9"DIA X 27.5'	GCR Manufacturing Co.	Good	Satisfactory		None
C-8	Flocculator Drive Unit No. 3	1989	17	25	8	Adjustable frequency drive	0.75 hp, 2 fps	Eurodrive, Inc.	Good	Satisfactory		None
C-9	Stage 3 Flocculator Baffle Wall	1989	17	30	13	Clear redwood			Good	Satisfactory		None
C-10	Structure	1989	17	75	58				Good	Satisfactory	Cracking in vertical walls is leaking and causing unsightly condition on exterior, visible walls.	Repair cracking with a suitable repair method such as epoxy injection or a strip and seal system. Repair exterior visible walls to improve structure appearance. Perform work after construction of second flocculation and sedimentation basin.
C-11	Heating and Ventilation	1989	17	30	13				Good	Satisfactory		None
C-12	Electrical and Controls	1989	17	40	23				Good	Satisfactory		None
C-13	Miscellaneous Equipment	1989	17	Varies	Varies				Good	Satisfactory		None
D. Sedimentation Basin												
D-1	Sludge Transfer Valve and Actuator	1989	17	25	8	Plug valve with motor operator	8-inch	DeZurik	Good	Satisfactory		None
D-2	Sludge Cross Collector	1989	17	30	13	Thermoplastic collector chains / fiberglass flights	4 fpm / 3'-11" X 6" flights	Dresser Industries	Good	Satisfactory		Replace wear surface on all collectors as part of plant expansion project.
D-3	Sludge Collector No. 1 (East)	1989	17	30	13	Thermoplastic collector chains / fiberglass flights	2 fpm / 13'-11.75" X 8" flights	Dresser Industries	Good	Satisfactory		Replace wear surface on all collectors as part of plant expansion project.
D-4	Sludge Collector No. 2 (West)	1989	17	30	13	Thermoplastic collector chains / fiberglass flights	2 fpm / 13'-11.75" X 8" flights	Dresser Industries	Good	Satisfactory		Replace wear surface on all collectors as part of plant expansion project.
D-5	Sludge Collector Drive Unit	1989	17	25	8	Helical gear	0.75 hp	Foote-Jones Dresser / Baldor Motor	Good	Satisfactory		None
D-6	Tube Settlers No. 1 (East)	1989	17	20	3	60^o	60 = 30" X 118.75" X 20.75" 12 = 16" X 118.75" X 20.75"	Microfloc	Poor	Satisfactory	Tube settler blocks have deteriorated significantly. Supporting structure is in good condition.	Replace tube settler blocks as part of plant expansion project.
D-7	Tube Settlers No. 2 (West)	1989	17	20	3	60^o	60 = 30" X 118.75" X 20.75" 12 = 16" X 118.75" X 20.75"	Microfloc	Poor	Satisfactory	Tube settler blocks have deteriorated significantly. Supporting structure is in good condition.	Replace tube settler blocks as part of plant expansion project.
D-8	Launders	1989	17	40	23	Fiberglass	12" W X 16.25" Deep X 13'-3" Long, 540 gpm	Leopold	Good	Satisfactory	9 troughs on each side of channel collector; each trough equipped with V-notch weir plates; both sides	None
D-9	Structure	1989	17	75	58				Good	Satisfactory	Cracks in walls leaking, causing plaster to delaminate. Condition is unsightly but the cracks do not appear to be structurally significant.	Repair cracks with a suitable repair method such as epoxy injection or a strip and seal system. Repair plaster on exterior walls to improve structure appearance. Perform work as part of plant expansion project.
D-10	Electrical and Controls	1989	17	40	23				Good	Satisfactory		None
D-11	Miscellaneous Equipment	1989	17	Varies	Varies				Good	Satisfactory		None
E. Filters												
E-1	Filter No. 1 (Southeast)	1992	14	50	36	Multi-media rapid sand gravity	418 ft2; 3 mgd @ 5 gpm/ft2	F.B. Leopold Co.	Good	Satisfactory	Filter media replaced and underdrain system repaired in 1997.	None
E-2	Filter No. 2 (Northeast)	1992	14	50	36	Multi-media rapid sand gravity	418 ft2; 3 mgd @ 5 gpm/ft2	F.B. Leopold Co.	Good	Satisfactory	Filter media replaced and underdrain system repaired in 1997.	None
E-3	Filter No. 3 (Southwest)	1992	14	50	36	Multi-media rapid sand gravity	418 ft2; 3 mgd @ 5 gpm/ft2	F.B. Leopold Co.	Good	Satisfactory	Filter media replaced and underdrain system repaired in 1997.	None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
E-4	Filter No. 4 (Northwest)	1992	14	50	36	Multi-media rapid sand gravity	418 ft2; 3 mgd @ 5 gpm/ft2	F.B. Leopold Co.	Good	Satisfactory	Filter media replaced and underdrain system repaired in 1997.	None
E-1 - E-4	Filter Media Replacement	1997	9	20	11							Replace media in Filter Nos. 1 through 4 when media is 20 years old in 2017.
E-5	Filter Flow Meter No. 1	1992	14	50	36	Propeller meter	18-inch	Sparling / Direct Drive Tube	Good	Satisfactory	Consider replacing with magmeter as part of overall upgrading of filter piping, valving and instrumentation and control system during proposed plant expansion project.	None
E-6	Filter Flow Meter No. 2	1992	14	50	36	Propeller meter	18-inch	Sparling / Direct Drive Tube	Good	Satisfactory	Consider replacing with magmeter as part of overall upgrading of filter piping, valving and instrumentation and control system during proposed plant expansion project.	None
E-7	Filter Flow Meter No. 3	1992	14	50	36	Propeller meter	18-inch	Sparling / Direct Drive Tube	Good	Satisfactory	Consider replacing with magmeter as part of overall upgrading of filter piping, valving and instrumentation and control system during proposed plant expansion project.	None
E-8	Filter Flow Meter No. 4	1992	14	50	36	Propeller meter	18-inch	Sparling / Direct Drive Tube	Good	Satisfactory	Consider replacing with magmeter as part of overall upgrading of filter piping, valving and instrumentation and control system during proposed plant expansion project.	None
E-9	Backwash Troughs	1992	14	40	26	Reinforced fiberglass with stainless steel stabilizers	1'-3" wide X 1.5' deep	F.B. Leopold Co.	Fair	Satisfactory	6 troughs per filter; troughs show signs of "wear and tear."	Replace the troughs during the construction of two new filters.
E-10	Filter Control Valves, Filters 1 through 4	1992	14	50	36	Butterfly valves, solenoid controlled, hydraulically actuated	Varies	Pratt Butterfly valves; Pratt Dura-Cyl actuators	Good	Satisfactory	When plant expands, may use same actuators or may install electric actuators on new valves and transition existing valves to electric at that time or at later date	
E-11	Structure	1992	14	75	61				Good	Satisfactory		Install improved personnel access facility to high service pump station roof. Current City project will replace roof.
E-12	Electrical and Controls	1992	14	40	26				Good	Satisfactory		None
E-13	Miscellaneous Equipment	1992	14	Varies	Varies				Good	Satisfactory		None
F. High Service Pump Station												
F-1	Pump No. 1	1988	18	50	32	Vertical turbine	1,400 gpm at 305 ft., 150 hp.	Peabody Floway / Model 14 DKL (4-stage) / GE Motor	Good	Satisfactory	Motor failed in 1995. Motor rewound. Pump bearings repaired in 1997.	None
F-2	Pump No. 2	1988	18	50	32	Vertical turbine	3,500 gpm (nomial) @ 305 ft., 300 hp actual rate: 3,300 gpm	Peabody Floway / Model 16 DKL (3-stage) / GE Motor	Good	Satisfactory	Motor failed in 1989, was rewound. City to install VFD on this pump soon.	None
F-3	Pump No. 3	1988	18	50	32	Vertical turbine	3,500 gpm (nomial) @ 305 ft., 300 hp actual rate: 3,300 gpm	Peabody Floway / Model 16 DKL (3-stage) / GE Motor	Good	Satisfactory	Starter trips out for no apparent reason. Problem started in early 2005.	Retain electrician to diagnose problem and repair.
F-4	Pump No. 4	1988	18	50	32	Vertical turbine	3,500 gpm (nomial) @ 305 ft., 300 hp	Peabody Floway / Model 16 DKL (3-	Good	Satisfactory	Motor failed in 1996, was rewound.	None
F-5	Pump Control Valves	1988	18	50	32	Ball valves, Class 300	8-inch for No. 1, 12-inch for Nos. 2, 3 and 4	Henry Pratt Co.	Good	Satisfactory	Rebuilt Ball Valve No. 1 to replace the shaft seals. Each pump discharge is also equipped with an air release valve and a 250-lb flanged butterfly valve.	None
F-6	Backwash Pump No. 1 (North)	1992	14	50	36	Vertical turbine	3,250 gpm @ 41 ft., 50 hp	Fairbanks Morse / Model 17H (single-phase) / U.S. Motors	Good	Satisfactory		None
F-7	Backwash Pump No. 2 (South)	1992	14	50	36	Vertical turbine	3,250 gpm @ 41 ft., 50 hp	Fairbanks Morse / Model 17H (single-phase) / U.S. Motors	Good	Satisfactory		None
F-8	Backwash Pump Discharge Valves	1992	14	50	36	Swing check & butterfly valves	16-inch		Good	Satisfactory	Each pump discharge is also equipped with an air release valve.	None
F-9	Plant Water Flow Meter	1992	14	50	36	Turbine meter	3-inch		Good	Satisfactory		None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
F-10	Finished Water Flow Meter	1992	14	50	36	Propeller meter	30-inch	Sparling / Direct Drive Tube	Unknown	Unsatisfactory	Pre-cast concrete vault with sump and drain.	Replace propellor meter with a new 30-inch magmeter.
F-11	Surge Tank	1992	14	50	36	Above-ground steel, hydropneumatic surge arrestor	Approx. 14,200 gallons, 9' Dia. X 33' Long	Fluid Kinetics	Good	Satisfactory	Problems with tank automatic air level control system. Not properly controlling air level in tank per original design. Level sensing system likely needs replacing.	Upgrade or replace tank automatic air level control system.
F-12	Structure	1988	14	75	61				Good	Satisfactory	Roof is ready for replacement.	Improve roof access per Item E-11. Replace roof.
F-13	Heating and Ventilation	1988	18	30	12				Good	Satisfactory		None
F-14	Electrical and Controls	1988	18	40	22				Good	Satisfactory		None
F-15	Miscellaneous Equipment	1988	18	Varies	Varies						Corrosion present on the clearwell vent screen.	Replace clearwell vent screen.
G. Operations and Control Building												
G-1	Backwash Flow Meter	1992	14	50	36	Propeller meter	16-inch	Sparling / Direct Drive Tube	Unknown	Unknown	Include in program to replace propeller meters with magmeters.	Replace with a magmeter.
G-2	Backwash Control Valve	1992	14	50	36	Butterfly valve	16-inch		Good	Unsatisfactory	Original cylinder developed problems after a few years so it was replaced. Second cilnder worked for a while then developed problems. A third cylinder was installed recently and appears to work fine.	Investigate whether current and previous cylinders were undersized. If so, replace existing with properly sized cylinder.
G-3	Air Blower	1992	14	40	26	Centrifugal	1,050 scfm, 60 hp	Lamson Corp. / Model 555 Series	Good	Satisfactory		Relocate air flow control system to blower waste vent.
G-4	Air Compressor	1992	14	25	11	Reciprocating	15 hp, 120-gallon tank	Quincy / Model F370	Good	Satisfactory	For air supply to hydropneumatic surge control system	None
G-5	Emergency Plant Water Pump	1992	14	20	6	Submersible vertical turbine, 9-stage	4" DIA, 1.5 hp, 25 gpm	Grundfos / Model 25515-9	Good	Satisfactory	Located in clearwell beneath the dry chemical loading and storage room	None
G-6	Raw Water (RW) Sample Pump	1992	14	20	6	Horizontal end suction magnetic centrifugal pump	2 hp	March / TE-5C-MD	Good	Satisfactory		None
G-7	Raw Water Coagulated (RWC) Sample Pump	1992	14	20	6	Horizontal end suction magnetic centrifugal pump	2 hp	March / TE-5C-MD	Good	Satisfactory		None
G-8	Surge Tank Control Panel	1992	14	40	26				Good	Satisfactory	Located in blower room	Coordinate with air control system upgrading under Item F-11.
G-9	Structure	1992	14	75	61				Good	Satisfactory	Roofing recently replaced. Efflorescence or corrosion apparent on CMU wall beneath 42" settled water pipe in dry chemical loading and storage room.	Possibly due to corrosive atmosphere emanating from the opening in the clear well access hatch. See HVAC system.
G-10	HVAC System	1992	14	30	16				Good	Satisfactory	Corrosive atmosphere in south corner of dry chemical loading & storage room emanating from the opening in the clear well access hatch.	Install exhaust fan to remove corrosive atmosphere. Locate fan low, near opening in clear well access hatch. Duct exhaust to the east, away from sed basin structure.
G-11	Building Plumbing Systems	1992	14	40	26				Good	Satisfactory		None
G-12	Building Electrical Systems	1992	14	40	26				Good	Satisfactory	Conduits and junction box beneath 42" settled water pipe in south corner of dry chemical loading & storage room show signs of corrosion.	Possibly due to corrosive atmosphere emanating from the opening in the clear well access hatch. See HVAC system.
G-13	Elevator	1992	14	40	26	Hydraulic		U.S. Elevator / Diplomat series	Good	Satisfactory		Upgrade elevator control system.
G-14	Miscellaneous Equipment	1992	14	Varies	Varies				Good	Satisfactory		None
H. Chemical Feed Systems												
H-1	ACH Storage Tank No. 1	1992	14	40	26	FRP	6,000 gallons, 12' DIA X 7' High		Good	Satisfactory		None
H-2	ACH Storage Tank No. 2	1992	14	40	26	FRP	6,000 gallons, 12' DIA X 7' High		Good	Satisfactory		None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
H-3	ACH Feed Pump No. 1	1992	14	25	11	Simplex chemical proportioning pump, positive displacement diaphragm type	20.8 gph, 0.25 hp	Wallace & Tiernan / 44-113	Good	Satisfactory	Equipped with a variable speed motor and an SCR controller.	None
H-4	ACH Feed Pump No. 2	1992	14	25	11	Simplex chemical proportioning pump, positive displacement diaphragm type	20.8 gph, 0.25 hp	Wallace & Tiernan / 44-113	Good	Satisfactory	Equipped with a variable speed motor and an SCR controller.	None
H-5	Polymer Mixing Unit	1992	14	25	11	Automatic aged polymer solution mixer	26 lbs. per day	Wallace & Tiernan / Series 35-300, Model 25	Good	Satisfactory	Equipped with a Wallace & Tiernan 32-055 volumetric feeder with an SCR controller.	None
H-6	Polymer Feed Pump No. 1	1992	14	25	11	Dual head double simplex chemical proportioning pump, positive displacement diaphragm type	69.1 gph @ 50 psig Max, 0.25 hp	Wallace & Tiernan / 44-124	Good	Satisfactory	Equipped with a variable speed motor and an SCR controller.	None
H-7	Polymer Feed Pump No. 2	1992	14	25	11	Dual head double simplex chemical proportioning pump, positive displacement diaphragm type	69.1 gph @ 50 psig Max, 0.25 hp	Wallace & Tiernan / 44-124	Good	Satisfactory	Equipped with a variable speed motor and an SCR controller.	None
H-8	PAC Mixing Unit	1992	14	25	11	Screw-type feeder with explosion-proof motor, 35-gallon SS tank with explosion-proof mixer	35-gallon SS tank, 0.60 cubic feet per hour	Wallace & Tiernan / Model 32-055 volumetric feeder	Good	Satisfactory	PAC slurry fed with TMG Services eductor.	Unit has not been used therefore customary replacement schedule at end of useful life can be deferred indefinitely.
H-9	Brine Tank	2001	5	40	35	Cross-linked polyethylene	8,225 gallons, 37 tons salt capacity, 10' DIA X 14' High	Core-Rosion Products / 11008050110	Good	Satisfactory		None
H-10	Brine Booster Pump A	2001	5	25	20	Centrifugal, seal-less magnetic drive	490 gph @ 3 ft.; 16' Max Head, 0.05 hp	March / LC-3CP-MD	Good	Satisfactory		None
H-11	Brine Booster Pump B	2001	5	25	20	Centrifugal, seal-less magnetic drive	490 gph @ 3 ft.; 16' Max Head, 0.05 hp	March / LC-3CP-MD	Good	Satisfactory		None
H-12	Oxidant Generating Unit No. 1	2001	5	25	20		100 lbs. per day	MIOX	Good	Satisfactory	System includes control & softening unit; located in former chlorine storage room.	None
H-13	Oxidant Generating Unit No. 2	2001	5	25	20		100 lbs. per day	MIOX	Good	Satisfactory	System includes control & softening unit; located in former chlorine storage room.	None
H-14	Oxidant Tank A	2004	2	40	38	HDLPE	525 gallons	Snyder California Container	Good	Satisfactory	Located in former chlorine storage room; replaced the original cross-linked polyethylene tank installed	None
H-15	Oxidant Tank B	2005	1	40	39	HDLPE	500 gallons	Snyder California Container	Good	Satisfactory	Located in former chlorine storage room; replaced the original cross-linked polyethylene tank installed	None
H-16	Oxidant Pump No. 1	2001	5	25	20	Diaphragm metering	300 gph @ 50 psi	LMI Milton Roy	Good	Satisfactory	Located in former chlorine storage room; pumps to rapid mix basin.	None
H-17	Oxidant Pump No. 2	2001	5	25	20	Diaphragm metering	300 gph @ 50 psi	LMI Milton Roy	Good	Satisfactory	Located in former chlorine storage room; standby pump	None
H-18	Oxidant Pump No. 3	2001	5	25	20	Diaphragm metering	147 gph @ 100 psi	LMI Milton Roy	Good	Satisfactory	Located in former chlorine storage room; pumps to clearwell.	None
H-19	Dry Chemical Feed System No. 1	1992	14	25	11	Volumetric type	3 cubic feet/hr. - 75 gallon solution tank	Wallace & Tiernan / Series 35-150	Good	Satisfactory	Not used since 1999	Unit has not been used extensively therefore customary replacement schedule at end of useful life can be deferred indefinitely.
H-20	Dry Chemical Feed System No. 2	1992	14	25	11	Volumetric type	3 cubic feet/hr. - 75 gallon solution tank	Wallace & Tiernan / Series 35-150	Good	Satisfactory	Not used since 1999	Unit has not been used extensively therefore customary replacement schedule at end of useful life can be deferred indefinitely.
I. Wastewater Pump Station												
I-1	Wastewater Pump No. 1	1992	14	50	36	Submersible, non-clog centrifugal	1,500 gpm, 35 hp, 8-inch	FLYGT / CP3201-638	Good	Satisfactory	Pump rebuilt in 2002	None
I-2	Wastewater Pump No. 2	1992	14	50	36	Submersible, non-clog centrifugal	1,500 gpm, 35 hp, 8-inch	FLYGT / CP3201-638	Good	Satisfactory	Pump rebuilt in 2002	None
I-3	Wastewater Pump No. 3	1992	14	50	36	Submersible, non-clog centrifugal	1,500 gpm, 35 hp, 8-inch	FLYGT / CP3201-638	Good	Satisfactory	Pump rebuilt in 2002	None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
I-4	Pump Discharge Valves	1992	14	50	36	Swing check and plug valves	10-inch	Swing check valves: GA Industries, Plug valves: Keystone	Good	Satisfactory		None
I-5	Structure	1992	14	75	61				Good	Satisfactory		None
I-6	Electrical and Controls	1992	14	40	26				Good	Satisfactory		None
I-7	Miscellaneous Equipment	1992	14	Varies	Varies				Good	Satisfactory		None
J. Solids Handling System (Put in service, fall 2006)												
J-1	Wastewater Settling Basin No.	2006	0	75	75		72' X 72' X 12' deep		New	Not known		None
J-2	Wastewater Settling Basin No.	2006	0	75	75		72' X 72' X 12' deep		New	Not known		None
J-3	Solids Removal Pipe Gate No. 1	2006	0	50	50	Knife gate	12-inch		New	Not known		None
J-4	Solids Removal Pipe Gate No. 2	2006	0	50	50	Knife gate	12-inch		New	Not known		None
J-5	Solids Removal Pump No. 1	2006	0	25	25	Submersible, non-clog		FLYGT / CP3068.180	New	Not known		None
J-6	Solids Removal Pump No. 2	2006	0	25	25	Submersible, non-clog		FLYGT / CP3068.180	New	Not known		None
J-7	Slide Gate No. 1	2006	0	50	50		2' X 1' Opening		New	Not known		None
J-8	Slide Gate No. 2	2006	0	50	50		2' X 1' Opening		New	Not known		None
J-9	Series Operation Slide Gate	2006	0	50	50		2' X 1' Opening		New	Not known		None
J-10	Sludge Drying Bed No. 1 (North)	2006	0	75	75		74' X 150' X 4.5' deep		New	Not known		None
J-11	Sludge Drying Bed No. 2 (South)	2006	0	75	75		74' X 150' X 4.5' deep		New	Not known		None
J-12	Equipment Bldg.	2006	0	75	75				New	Not known		None
J-13	Electrical and Controls	2006	0	40	40				New	Not known		None
J-14	Miscellaneous Equipment	2006	New	Varies	Varies				New	Not known		None
K. Monitoring and Control Systems												
K-1	Finished Water Turbidimeter	1992	14	15	1	Low range, continuously reading, on-line meter	100 NTU	Hach / 1720C	Good	Satisfactory		Replace existing unit. Unit has reached the end of its useful life and newer versions are available.
K-2	Raw Water Turbidimeter	1992	14	15	1	High range, continuously reading, on-line meter	9,999 NTU	Hach / Model 556E Surface Scatter Turbidimeter	Good	Satisfactory		Replace existing unit. Unit has reached the end of its useful life and newer versions are available.
K-3	Filter Water Turbidimeters (4)	1992	14	15	1	Low range, continuously reading, on-line meter	100 NTU	Hach / 1720C	Good	Satisfactory	One turbidimeter present on each filter effluent line.	Replace existing units. Units have reached the end of their useful lives and newer versions are available.
K-4	Pressure Differential Transmitters (5)	1992	14	20	6	Electric transmitter	25" - 150" Water, Max static pressure 2,500 psig	ABB Kent-Taylor / 505T	Good	Satisfactory	One differential pressure transmitter present on each filter; one pressure transmitter present on finished water line at HSPS. Lower housings on all transmitters show corrosion, but useful life does not appear to be compromised.	None
K-5	Chlorine Residual Analyzer	2006	1	15	14	DPD colorimetric type	5 mg/L	Hach / CL17	Good	Satisfactory		Existing unit was recently replaced and is not currently in need of replacement.
K-6	Raw Water Temperature Transmitter	1992	14	20	6	Resistance temperature detector (RTD)	100 degree F. max.		Good	Satisfactory		None

Table 5-1
Winchester Water Treatment Plant
Facility Inventory and Condition Review

Item No.	Equipment or Facility	Year of Constr. or Install.	Age, Years	Approx. Useful Life, Years	Approx. Remaining Useful Life, Years	Type	Capacity/Size	Manufacturer/Model	Condition	Performance	Comments	Recommendations
K-7	Raw Water Coagulated Streaming Current Monitor	1992	14	20	6			Chemtrac Systems, Inc. / 2000XR	Good	Satisfactory		Replace existing unit. Unit has reached the end of its useful life and newer versions are available.
K-8	Plant Supervisory Control and Data Acquisition (SCADA) System	1992	14	20	6			S&B, Inc.	Good	Satisfactory	See S&B, Inc. technical memorandum.	Upgrade with plant expansion project.
L. Site Access Control and Security Systems												
L-1	Security Systems Assessment											Perform plant security systems assessment.
L-2	Security Systems Upgrade											Upgrade security systems as recommended in assessment.
M. Yard Piping and Valving												
M-1	Yard Piping and Valving	1987 - 2006	0 - 19	Varies	Varies	Varies	Varies	Varies	Good	Satisfactory	Need input from plant operators.	None
N. Site Development												
N-1	Paving, Landscaping, and Misc.	1987 - 2006	0-19	Varies	Varies	Varies	Varies	Varies	Good	Satisfactory	Need input from plant operators.	None

Table 5-2
Winchester Water Treatment Plant
Capital Maintenance Program Summary

Location	Project Description	2007/2008	2008/2009	2009/2010	2010/2011	2011/2012	2012/2013	2013/2014	2014/2015	2015/2016	2016/2017	2017/2018	2018/2019	2019/2020	2020/2021	2021/2022	2022/2023	2023/2024	2024/2025	2025/2026	2026/2027	2027 +	Estimated Project Cost
River Intake	Replace roofing	$ 10,000																					$ 10,000
	River deposition study	$ 10,000																					$ 10,000
	Test & rebuild intake pumps	$ 55,000																					$ 55,000
	Replace heating and ventilation systems											$ 8,000											$ 8,000
	Sub-Total	$ 75,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 8,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 83,000
Rapid Mix Basin	Replace rapid mixers (2)								$ 33,000														$ 33,000
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 33,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 33,000
Flocculation Basin	Replace stage 1, 2 and 3 flocculators													$ 53,000									$ 53,000
	Replace flocculator drive units (3)								$ 28,000														$ 28,000
	Replace flocculator baffle walls (3)													$ 22,000									$ 22,000
	Replace heating and ventilation systems													$ 7,000									$ 7,000
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 28,000	$ -	$ -	$ -	$ -	$ 82,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 110,000
Sedimentation Basin	Replace sludge transfer valve and accuator								$ 10,000														$ 10,000
	Replace sludge collectors and cross collectors (2)													$ 70,000									$ 70,000
	Replace sludge collector drive								$ 9,000														$ 9,000
	Replace tube settlers		See Note 1																				$ -
	Repair cracking in basin walls and repair exterior basin surfaces		See Note 1																				$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 19,000	$ -	$ -	$ -	$ -	$ 70,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 89,000
Filters	Replace media in filter nos. 1 through 4											$ 141,000											$ 141,000
	Replace backwash troughs		See Note 1																				$ -
	Install improved personnel access to high service pump station roof		See Note 1																				$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 141,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 141,000
High Service Pump Station	Replace existing 30" finished water propeller meter with 30" magmeter		See Note 1																				$ -
	Upgrade or replace surge tank automatic air level control system		See Note 1																				$ -
	Replace roofing	$ 15,000																					$ 15,000
	Replace heating and ventilation systems												$ 37,000										$ 37,000
	Sub-Total	$ 15,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 37,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 52,000
Operations and Control Building	Replace the 16" backwash meter with a magmeter		See Note 1																				$ -
	Investigate problems with backwash flow control valve operator	$ 2,000																					$ 2,000
	Relocate blower air flow control valve to blower waste vent		See Note 1																				$ -
	Upgrade elevator control system	$ 5,000																					$ 5,000
	Replace air compressor											$ 18,000											$ 18,000
	Replace emergency plant water pump						$ 5,000																$ 5,000
	Replace raw and finished water sample pumps						$ 3,000																$ 3,000
	CT Compliance study	$ 8,000																					$ 8,000
	Install vent fan in chem storage room near clearwell access hatch	$ 5,000																					$ 5,000
	Upgrade and/or replace critical elements of heating and ventilation systems																$ 33,000						$ 33,000
	Sub-Total	$ 20,000	$ -	$ -	$ -	$ -	$ 8,000	$ -	$ -	$ -	$ -	$ 18,000	$ -	$ -	$ -	$ -	$ 33,000	$ -	$ -	$ -	$ -	$ -	$ 79,000

Table 5-2
Winchester Water Treatment Plant
Capital Maintenance Program Summary

Location	Project Description	2007/2008	2008/2009	2009/2010	2010/2011	2011/2012	2012/2013	2013/2014	2014/2015	2015/2016	2016/2017	2017/2018	2018/2019	2019/2020	2020/2021	2021/2022	2022/2023	2023/2024	2024/2025	2025/2026	2026/2027	2027 +	Estimated Project Cost
Chemical Feed Systems	Replace ACH feed pumps (2)											$ 12,000											$ 12,000
	Replace polymer mixing unit											$ 34,000											$ 34,000
	Replace polymer feed pumps (2)											$ 12,000											$ 12,000
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 58,000	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 58,000
Wastewater Pump Station	None																						$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Solids Handling System	None																						$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Instruments & Controls	Replace existing turbidimeters and streaming current monitor	$ 20,000																					$ 20,000
	Replace pressure differential transmitters (5)						$ 12,500																$ 12,500
	Replace raw water termperature transmitter						$ 2,000																$ 2,000
	Purchase benchtop UV Spec unit for TOC analysis	$ 7,000																					$ 7,000
	Upgrade plant supervisory control and data acquisition (SCADA) system		See Note 1																				$ -
	Sub-Total	$ 27,000	$ -	$ -	$ -	$ -	$ 14,500	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ 41,500
Site Security Systems	Perform site security assessment		See Note 1																				$ -
	Upgrade site security systems		See Note 1																				$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Yard Piping and Valving	None																						$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Site Development	None																						$ -
	Sub-Total	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
	Grand Totals	**$ 137,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 22,500**	**$ -**	**$ 80,000**	**$ -**	**$ -**	**$ 225,000**	**$ 37,000**	**$ 152,000**	**$ -**	**$ -**	**$ 33,000**	**$ -**	**$ -**	**$ -**	**$ -**	**$ -**	**$ 686,500**

NOTES:

1. These improvements are to be incorporated into the water treatment plant expansion project.
2. Cost estimates based upon ENR Construction Cost Index (Seattle) of 8685, June 2008.

SECTION 6

SECTION 6
ALTERNATIVES ANALYSIS FOR CRITICAL PROCESS ISSUES

General

This section presents and analyzes alternatives for several critical process issues at the Winchester WTP including alternatives for fish screening at the river intake, alternative clarification approaches in the sedimentation basins, membrane technology as an alternative to conventional granular media filtration, and alternative approaches to meeting CT requirements including use of alternative disinfectants.

Fish Screening at Intake

Although the intake and traveling fish screens complied with the regulations that existed at the time they were installed, the current regulations are more restrictive. Sections 3 and 4 describe the current regulations and the need to make improvements to the fish screening at the intake at flows above 16.5 mgd. Above that flow rate, the requirements of the current regulations will be violated because, due to the configuration of the intake structure, the maximum approach velocity to the structure will be exceeded. Fixed fish screens on the exterior of the structure will need to be installed at that time to meet the current fish screening requirements for intake structures. The traveling screens will not be useful or needed after the fixed screens are installed and they can then be removed.

The primary challenge with installing fixed screens on the existing structure will be the installation of a screen cleaning system. Intakes that draw more than 3 cfs through a fixed plate screen are required by regulation to have an automated screen cleaning system. The cleaning system may be activated by elapsed time, by head loss across the screen or by both.

Fixed screens can be cleaned with either water, air or mechanical brushes. For water cleaning, nozzles are typically located on the inside of the screens. The nozzles need to be located close the screens to ensure proper cleaning. In the existing structure, there is a concrete obstruction 2-feet wide between the 4-feet square openings that could pose a challenge to locating a nozzle cleaning system. It may be possible to install the fixed screens on the exterior face of the intake and locate the nozzle spray system in the 8-inch wide space currently occupied by the trash rack. Nozzles can be arranged in a fixed array or they can be aligned as a single row of nozzles. The latter arrangement has fewer nozzles so it requires a lower flow rate to clean the screens, but it requires that the nozzles travel up and down on a rail behind the screens to clean the entire screen area. This can introduce maintenance problems. Water supply for a water cleaning system can be supplied from a separate pumping system using raw water from within the intake structure.

For air cleaning, it is advisable to orient the fixed screens at an angle to the vertical. This encourages air introduced below the screen to scour the entire screen surface, reducing the tendency of air burst systems to clean only the upper portion of a screen. Air compressors and an air receiver tank are required for an air cleaning system.

Mechanical brush systems typically consist of a brush assembly mounted on a monorail track system that is parallel to the screen face. The track is mounted above the water surface, which provides access for maintenance.

Other methods of screen cleaning may be appropriate to the specific situation for the City's river intake. The type of algae that attaches to intake screens varies greatly among different intakes on different sources. Data on the type of algae that exists in the North Umpqua River at the intake site and that is expected to attach to the fixed screens should be obtained during design to assist in determining which screen cleaning system is likely to perform best for the conditions at the site.

The construction of the fish screen improvements could be implemented as part of the expansion of the plant to 18 mgd or could be deferred until the plant flow approaches 16 mgd.

Sedimentation Basin Settlers

The existing tube settlers in the existing sedimentation basin need to be replaced and a new sedimentation basin needs to be constructed to expand the plant capacity to 18 mgd. Clarification technologies for conventional treatment have advanced in recent years. As an alternative to installing new tube settlers, the installation of Lamella plate settlers was considered.

Although tube settlers have a shorter life expectancy than Lamella plates, tube settlers have a lower life-cycle cost. Tube settlers also have a shallower profile than Lamella plates, which makes them more suitable for the existing basin design. Finally the tube settlers have performed well in the basin at the Winchester WTP. Analysis of the historical plant performance data presented in Section 2 demonstrates that the existing pretreatment technology has consistently producing settled water with turbidity less than 2.0 NTU, even at the higher flow rates experience during the summer months. Given these facts, it is recommended that plastic tube settlers be installed to replace the old, existing tube settlers and that tube settlers be used in the new basin.

The existing tube settlers can continue to function until the second basin has been constructed. Replacement of these tube settlers can then be completed along with the other repairs to the existing sedimentation basin which are discussed in Section 5.

Filtration

Two options for expanding the treatment capacity of the filtration process at the Winchester WTP were considered.

- Option 1: Maintain the existing, conventional treatment process adding one new flocculation/sedimentation basin and two new granular media filters;

- Option 2: Replace the conventional media filters with a membrane filtration system.

In membrane filtration, hollow fiber membranes are used to separate particles from the water. The membranes have small pores, on the order of 0.1 micrometers (μm) or less. The pores allow water to pass through the membrane while retaining particles larger than about 1.0 μm. The membranes are formed into hollow fibers which are bundled together longitudinally and either encased into a pressure vessel or submerged in a basin. Pressure membranes operate with the unfiltered water pumped through the inside of the hollow fiber. Particles are retained on the inside of the hollow fiber while filtered water passes through the pores to the outside of the fiber. Submerged, or "vacuum," membranes operate with the unfiltered water on the outside of the fiber. Particles are retained on the outside of the fiber while filtered water passes through the pores to the inside of the hollow fiber under the pressure differential provided by a vacuum applied to the inside of the hollow fiber.

The existing plant could be converted to membrane technology as part of the program to expand the plant to 18 mgd. Alternatively, the initial expansion to 18 mgd could be done with the existing granular media design and expansion beyond 18 mgd could be accomplished with membranes. For retrofitting an existing rapid sand filtration plant for membrane filtration, it is common to use submerged membranes located inside the existing filter bays. This generally results in a lower capital cost than retrofitting with pressure membranes.

Membrane filtration has become an increasingly popular filtration alternative throughout the United States and in the Pacific Northwest. As the technology has matured, the costs for new construction are increasingly competitive with conventional filtration; however, the costs associated with converting existing conventional media filters to membrane filtration are still significantly higher than for other alternatives, particularly if no capacity expansion is desired.

For application at the Winchester WTP, membrane filtration would possibly be recommended in conjunction with pre-chlorination and coagulation. Membrane filters would provide an absolute barrier to *Giardia* and *Cryptosporidium*, thus ensuring continued compliance with future regulations; however, membranes are not capable of removing dissolved organic material, such as TOC, unless a coagulant is used to create a filterable floc.

It may be necessary, therefore, to coagulate the raw water prior to the membranes to remove total organic carbon (TOC) in order for the plant to continue to produce water with low disinfection by-products and to produce water with a consistently low and stable chlorine demand.

High-pressure membrane filtration is not considered a viable alternative for the Winchester WTP because the plant’s existing filters could be more readily retrofitted to accommodate the “submerged” technology, which would better match the plant’s existing hydraulic grade line and minimize additional pumping requirements. Membrane 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. There are several submerged membrane filtration systems on the market today which may be appropriate for the Winchester WTP, including those systems manufactured by General Electric/Zenon Environmental Inc. and Siemens/Memcor.

A pilot study to determine the design constraints for full-scale performance would be required if the City were to decide to implement this technology at the Winchester WTP, or if this process were to be considered for a new South Umpqua River supply. Significant engineering would be required to successfully integrate membrane technology into the existing Winchester WTP’s treatment process and to identify a location for all of 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 substantially less attractive compared to the alternative of expanded conventional treatment at the Winchester WTP.

Retrofitting the existing plant with submerged membrane technology to achieve an expanded capacity of 20 mgd is estimated to cost approximately $15 million. This is a conceptual planning-level estimate and includes engineering and contingencies. Further consideration of membrane filtration for expansion of the Winchester WTP is not warranted at this time based on this planning level cost estimate and on the following factors:

1) The existing plant and processes have been able to consistently produce high-quality water meeting all drinking water standards under a wide range of raw water quality;

2) The existing plant structures and processes have significant remaining useful life; and

3) There are no known regulatory “drivers”, such as high concentrations of *Cryptosporidium* in the raw water, to consider using a new technology.

The expansion of the Winchester WTP to 18 mgd should be accomplished by adding two new filters using the same media configuration as the existing four filters. If a subsequent plant expansion is undertaken to increase capacity beyond 18 mgd, replacing the existing granular media design with a deep bed design would expand capacity to about 22 mgd.

Based on this treatment capacity limitation, it is recommended that 22 mgd be considered the ultimate Winchester plant capacity using the plant's current treatment technologies.

Disinfection

Meeting CT in Existing Clearwell

As noted elsewhere, the most critical unit process that needs to be addressed immediately is the clearwell. The existing clearwell configuration coupled with current plant operating practices limit the plant's ability to meet the regulatory requirement for 0.5-log *Giardia* inactivation after filtration, even under current flow and water quality conditions.

The City should immediately plan to evaluate post-filtration CT compliance by conducting a tracer study to determine the clearwell contact time under existing conditions. A tracer study is underway and should be completed in the summer, 2009. The City should use the results of the tracer study to change the manner in which CT is presently calculated and reported to the State. The results should also be used to determine how much additional contact time will be needed to meet CT at the expanded capacity of 18 mgd. Improvements to the baffling in the clearwell that will increase the contact time should be included in the design of the plant expansion.

Based on the available data and on the assumptions noted in Table 4-4, it is likely that the existing clearwell can be modified to provide adequate CT when the new filters are constructed to treat up to 18 mgd. This can likely be accomplished using a slightly higher chlorine residual than the City has maintained in the recent past while significantly limiting clearwell operational storage volume by maintaining a clearwell level at or above 9.0 feet. Alternatively, adequate CT can be accomplished by maintaining a chlorine residual similar to that which the City has maintained in the recent past while completely eliminating clearwell operational storage volume by maintaining a full clearwell.

While significant increases in chlorine residual could be implemented for CT compliance, such increases are likely to generate customer complaints. The chlorine residual could be increased slowly over time to attempt to gain acceptance; but at some point there is a threshold which customers will not accept. Increasing chlorine residual needs to be approached with a great deal of caution.

Based on the available data, it will not be feasible to achieve adequate disinfection at flows of greater than 18 mgd with the available treated water contact volume modified as described in Section 4 and at current chlorine residual levels. For disinfection at plant capacities greater than 18 mgd, additional contact volume will be required or alternative disinfection processes must be introduced.

Meeting CT Using Off-Site Transmission System Volume

A portion of the finished water transmission system directly downstream of the plant could be used to provide additional contact volume to meet CT requirements beyond that provided by the clearwell. This approach could avoid the need for baffling the existing clearwell and potentially the need to provide more clearwell storage volume.

In order to accomplish this approach, the required length of the parallel finished water transmission pipelines providing the desired volume would need to be isolated from the distribution system. A sampling station would be installed at end of the contact length to provide for daily grab samples to measure residual chlorine concentration at that point. Existing service connections and distribution system connections within the contact length would need to be removed and reconnected to a new, parallel distribution system which would be supplied from the transmission system downstream of the sampling station. A preliminary review of the transmission and distribution system indicates that a new parallel distribution main pipe, perhaps 12-inch diameter, would be needed for the full contact length.

Currently there are two parallel transmission mains carrying finished water, a 20-inch outside diameter steel main with a 3/16-inch thick wall and a 30-inch diameter Class 50 ductile iron main. The steel pipe is very old and has a thin wall. It may be advisable to replace that portion of the 20-inch main that will be used to add contact time with a new, perhaps 36-inch, main. This would provide a combined total of 73 gallons of contact volume per foot of length in the new 36-inch pipe and the existing parallel 30-inch pipe. A substantial length of transmission system, perhaps several thousands of feet, would be needed to provide a significant contact volume.

The potential need for additional contact volume will depend upon the existing baffle efficiency determined by the tracer study and the potential maximum contact time that can be achieved in a practical, cost-effective manner when the two new filters are added and full clearwell baffling is accomplished. Once the maximum contact time that can be achieved through the recommended clearwell modifications has been determined, it will be possible to determine the maximum flow that can be treated with chlorine. If the study confirms the preliminary conclusions of the analysis conducted for preparation of Table 4-4, which determined that modifications to the clearwell can achieve 0.5-log *Giardia* inactivation using free chlorine at flows up to 18 mgd, then the need for additional contact volume outside of the clearwell will be obviated for the initial plant capacity expansion. If additional contact volume is determined to still be required, then use of the transmission system for contact volume may be considered.

Although there are other water treatment facilities that rely on their transmission pipelines for disinfection contact time, most plant owners prefer to retain all treatment unit operations at the water treatment plant site and deliver only completely treated water that meets all regulatory requirements from the site. This approach allows for better control of the treatment process, provides greater ease of operation and provides the greatest assurance of full regulatory compliance.

It is recommended that the City provide for both disinfectant contact volume and treated water storage volume on the existing or expanded plant site, this approach being preferable to using the transmission system for contact volume for the above-stated reasons.

Meeting CT with Alternative Disinfection Processes

Rather than increasing the contact time to ensure proper disinfection with free chlorine, the City could transition to an alternative disinfection process. Hypochlorite would continue to be used, but solely to maintain a residual in the distribution system. Alternative disinfection processes to consider would include ozone, ultraviolet light (UV) and chlorine dioxide.

If installation of alternative disinfection is to be considered in lieu of increased contact time during the design for the expansion to 18 mgd, it would be desirable to have the final results from the testing for *Cryptosporidium* in the source water. The City is currently conducting these tests under the requirements of the LT2ESWTR, as discussed in Section 3. Currently, it is believed that ozone may offer the best alternative treatment method for *Cryptosporidium* inactivation; however, EPA is required to develop support material and guidance manuals for the use of UV disinfection for *Cryptosporidium* inactivation.

Ozone would likely be the most expensive alternative disinfectant in terms of both capital and operating costs. Introducing ozone to the WTP has more occupational safety and health implications than does UV. In addition to *Cryptosporidium* inactivation, ozone does provide some taste and odor benefits; however, since taste and odor are not significant problems at the Winchester WTP, this latter benefit is negligible. A planning level capital construction cost estimate for installation of ozone at the Winchester WTP is $2.0 million. This assumes a dose of 2.0 mg/L for 18 mgd.

UV disinfection is a relatively new technology. Although it is listed as one of the “best available technologies”, there have been questions raised about the ability of many organisms to repair ultraviolet light-induced DNA damage. This could limit the utility of UV disinfection for *Cryptosporidium* inactivation. As part of the previously mentioned EPA mandate to develop materials to assist water providers regarding the use of UV, studies are being conducted to determine the infectivity of *Cryptosporidium* after irradiation with UV light.

Installing UV at the plant would be challenging due to space constraints. The UV system would be installed on a pipe between the filters and the clearwell, thus significant changes to the filter bay piping would be required. The UV reactor would be installed in a concrete structure below grade to the west of the filters and the water would be piped back to the east to the clearwell. A planning level capital construction cost estimate for installation of UV at the Winchester WTP is $1.8 to $2.0 million for a system to treat 18 mgd.

To the extent that *Cryptosporidium* inactivation is not a concern at the Winchester WTP, the additional capital and operating costs for converting to ozone or UV may not be justified. One alternative disinfectant that would have lower capital costs than both ozone and UV is chlorine dioxide. As the CT tables in Appendix C demonstrate, the CT requirements for chlorine dioxide are significantly less than those for free chlorine; thus, the existing clearwell could provide sufficient contact time for flows up to 18 mgd without significant modification. Alternatively, the clearwell could be modified as necessary to continue to use free chlorine up to 18 mgd and the transition to chlorine dioxide in the future would allow the clearwell to treat the ultimate plant capacity of 22 mgd.

Conversion to chlorine dioxide would significantly increase the complexity of plant operation. Chlorine dioxide is generated on-site and there are a number of methods for accomplishing this, however all of them use either chlorine gas or concentrated hypochlorite solution. The existing system for on-site hypochlorite generation would likely be abandoned and the chlorine room would be used for generation of the chlorine dioxide. Although chlorine dioxide can be used for maintaining a residual in the system (secondary disinfection), some utilities have reported customer complaints of chlorinous odors when chlorine dioxide is used for this purpose. To reduce complaints, some utilities that use chlorine dioxide for primary disinfection have converted to chloramines for secondary disinfection, further complicating plant operation.

Another chemical alternative to converting to ozone or UV while also avoiding the need to provide additional contact volume would be to increase the chlorine dose through the clearwell then dechlorinate with either ascorbic acid or sodium bisulfite to reduce the residual at the high service pump station before the water enters the system. Ascorbic acid would probably be the better choice for dechlorination, although it costs much more than sodium bisulfite. This approach would increase operating cost and increase disinfection complexity. It would also have an impact on the analysis in Section 4 regarding on-site hypochlorite generating capacity.

Recommendations

The City should immediately conduct a tracer study, using a methodology approved by DHS, and begin using the results of the tracer study to change the manner in which CT is presently calculated and reported. The design for expansion to 18 mgd should include improvements to the baffling in the clearwell to increase the contact time to the greatest extent possible. A second tracer study should be conducted after the expansion to 18 mgd to determine the contact volume with the modifications. Hypochlorite generated on-site should continue to serve as the disinfectant until the plant is expanded from 18 mgd to its ultimate capacity of 22 mgd. Determination of the preferred disinfection alternative for the expansion to 22 mgd can be made after tracer tests are done on the modified clearwell and after testing for *Cryptosporidium* in the source water, in line with the requirements of the LT2ESWTR, has been completed.

SECTION 7

General Conclusions from the Plant Evaluation

The plant has performed well as it approaches its current design capacity of 12 mgd. Expansion of the plant using the current treatment technology can increase the plant capacity to 18 mgd. With appropriate modification of the filter media, the treatment capacity of the plant can be expanded to 22 mgd with six filters in operation. Based on this filter treatment capacity, it is recommended that 22 mgd be considered the ultimate capacity for the Winchester WTP using rapid sand filtration.

Significant Regulatory Compliance Issues and Recommended Actions

The three most significant regulatory issues of concern regarding the existing plant and the plant expansion are:

1. Ability to consistently meet 0.5-log *Giardia* inactivation following filtration under all current and future plant flows and under a wide range of plant operating conditions.

2. Bin classification per the LT2ESWTR depending on raw water *Cryptosporidium* concentrations.

3. Compliance with maximum approach velocity to the raw water intake for protection of salmonid fish species at flows above 16.5 mgd.

The following actions are recommended to address these compliance issues:

1. The City should immediately conduct a tracer study to evaluate post-filtration CT compliance. The tracer study will determine the existing hydraulic efficiency in the clearwell, data which can then be used for updating the City’s calculation of CT. The data will also provide the basis of design for modifications to the clearwell to be implemented during the plant expansion to 18 mgd. Clearwell modifications are required to comply with disinfection regulations that were promulgated after the plant was designed. The City should discuss the CT calculation methodology further with DHS, obtain DHS approval for the methodology to be used for the tracer study, and conduct operations to assure that the disinfection contact time needed for 0.5-log *Giardia* inactivation in the clearwell is being achieved under current conditions. The City will complete a tracer study by the summer of 2009.

2. The City should continue its 2-year monitoring program for *Cryptosporidium* which is scheduled for completion in the summer of 2009.

3. Incorporate into the plant expansion improvements the installation of fixed screens in the river intake to replace the traveling screens or defer these improvements until the

required plant production capacity approaches 16.5 mgd which is estimated to be in the year 2022. The fixed screens will address compliance for maximum approach velocity for fish protection when peak plant flows exceed 16.5 mgd.

Other Recommended Immediate Actions

Additional actions are recommended for immediate accomplishment. These are described as follows:

1. Undertake and complete the work that is recommended in Section 2, Historical Plant Performance to be accomplished independent of the proposed plant expansion. This work is described as follows:

    a. Acquire a bench-top UV spectrophotometer for measurement of total organic carbon.

2. Undertake and complete the work as outlined in Section 5, Facilities Condition Review that is recommended to be accomplished independent of the proposed plant expansion. This work is described as follows:

    a. Complete an evaluation of the shoaling condition in front of the river intake and develop a plan of action, if removal of material is recommended.
    b. Replace the roof of the high service pump station.
    c. Test the river intake pumps and rebuild pumps as necessary.
    d. Replace the plant’s outdated turbidimeters.
    e. Complete the other recommended miscellaneous work as noted in Section 5.

3. Complete the recommended administrative actions with the Oregon Water Resources Department to secure the City’s existing water rights on the North Umpqua River at Winchester. (See discussions and recommendations in Long-Range Water Supply Plan.)

4. Undertake the recommended actions to seek to acquire additional water rights in the North Umpqua River Basin for use at the Winchester WTP to provide at least up to 22 mgd capacity. (See discussions and recommendations in Long-Range Water Supply Plan.)

5. Evaluate other potential sources of supply such as groundwater augmentation to the North Umpqua River to provide additional water supply to the plant. (See discussions and recommendations in Long-Range Water Supply Plan.)

6. Based upon the City’s success in securing its existing water rights at Winchester, acquiring additional water rights at the plant site, and developing additional water supply at Winchester, consider proceeding with acquisition of property adjacent to the

plant (portion or all of Tax Lot 800) to provide for expansion of the plant to beyond 18 mgd capacity and potentially to 22 mgd or more.

7. Adopt the capital maintenance plan and budget as developed in Section 5.

Plant Improvements to Achieve 18 mgd Capacity

It is recommended that the Winchester Water Treatment Plant be expanded soon to 18 mgd capacity. The general scope of the proposed project and the unit processes that need modification to expand the plant to 18 mgd are as follows:

1. Fish screening at river intake: Install fixed screens with cleaning system so that the approach velocity to the existing screens will not exceed regulatory requirements at flows above 16.5 mgd. Remove the existing traveling screens.

2. Raw water pumping: Replace Pump No. 1, a 4 mgd pump, with a 6 mgd pump to provide a firm capacity of 18 mgd. Install a variable frequency drive on new Pump No. 1.

3. Flocculation/sedimentation basin: Construct a second flocculation and sedimentation basin in parallel to the existing basin to achieve up to 24 mgd of pretreatment capacity.

4. Settled water transmission pipeline: Construct a second settled water transmission pipeline from the new flocculation and sedimentation basin. Connect the new pipeline to the existing settled water line to the existing filter influent channel.

5. Filtration: Construct two additional filters with the same filter media configuration as the existing to provide treatment capacity of 18 mgd. Provide filter-to-waste piping capable of handling 4 mgd, install metering and flow control facilities on the filter-to-waste, and install magnetic-type filter effluent flow meters. Modify the filter-to-waste piping, metering and flow control facilities and filter effluent metering in the existing four filters to be consistent with the new filters. Improve instrumentation on new and existing filters including the installation of particle counters on each filter.

6. Clearwell baffling: Install additional baffling and flow routing facilities within the existing clearwell and the expanded clearwell beneath the new filters to maximize to the extent practical and achievable the hydraulic efficiency of the clearwell for CT compliance.

7. Finished water pumping: Install a new 6 mgd pump (Pump No. 5) with a VFD and install a new 4 mgd pump (Pump No. 6) to provide a firm capacity of 18 mgd.

8. Hydropneumatic surge control: Replace the existing hydropneumatic surge tank with a larger tank to accommodate flows up to 18 mgd.

9. On-site sodium hypochlorite generation system: Install a fourth 100 pound per day generation unit to provide 300 ppd firm capacity for 18 mgd plant capacity. Expand hypochlorite solution storage volume.

10. Improvements recommended to be accomplished as part of the plant expansion project as described in Section 2, Historical Plant Performance, including installation of settled water turbidimeters, installation of particle counters on each filter, and detailed filter investigations.

11. Improvements, repairs, replacements and upgrading recommended to be accomplished as part of the plant expansion project as described and listed in Section 5, Facilities Condition Review.

Plant Improvements to Achieve up to 22 mgd Capacity

Based on the Long-Range Water Supply Plan, the Winchester WTP will need to be expanded beyond 18 mgd by the year 2025. The City’s current water rights total 20 mgd. The general scope of work required and the unit processes that need modification to expand the plant up to 22 mgd are as follows:

1. Fish screening at river intake: No modifications required if new fixed screens are designed to accommodate flows up to 22 mgd.

2. Raw water pumping: Replace three 6 mgd pumps with 8 mgd pumps to provide a firm capacity of 22 mgd. Provide VFDs on two new pumps. Replace all pump discharge piping and valving and raw water pipeline between intake and rapid mix basins with larger piping.

3. Filtration: Remove and replace the underdrains in Filter Nos. 1 through 4. Remove and replace the existing media in all six filters with a deep bed media configuration to increase treatment capacity to 22 mgd. Modify the backwash pumps if necessary to accommodate higher backwash rates. The backwash waste piping and system appear capable of handling higher backwash flows depending upon selection of the media size and the higher flows necessary to clean the media.

4. Clearwell contact time for disinfection: Existing clearwell with baffling improvements will not be sufficient for flows above 18 mgd. It will be necessary to:

    A. Increase the contact volume for disinfection with free chlorine by expanding the clearwell volume, or

    B. Change to an alternative disinfection technology, such as UV, ozone or chlorine dioxide, or

    C. Use a portion of the transmission main system off-site for contact volume, or

D. Increase the free chlorine in the clearwell substantially above current levels, then dechlorinate to the lower system residual level prior to pumping into the system.

The existing WTP site is seriously space-constrained and was intended for treatment of only 18 mgd. Alternative A will require additional land. The UV and ozone options under Alternative B will require additional land. The chlorine dioxide alternative will not require additional land; however, this alternative is highly complex to operate and control and is not recommended for implementation. Alternative C can be achieved without additional land but a portion of the treatment process is being accomplished off of the plant site. Alternative D is a complicated and relatively costly approach and is not recommended. For the purposes of cost estimating, it is assumed that the disinfectant will continue to be sodium hypochlorite and the clearwell volume will be expanded.

5. Clearwell storage volume: The volume of treated water storage should be increased to meet industry standards of at least 1 hour of detention time at the plant’s peak flow rate. This volume is in addition to that required for disinfectant contact time. Regardless of the disinfection method, additional clearwell volume beyond that presently existing will be required. The site cannot accommodate any additional clearwell volume and additional land will be required. For the purposes of cost estimating, it is assumed that the clearwell volume will be expanded by 1.25 million gallons to provide for 1 hour of storage volume and to provide for additional contact time for disinfection.

6. On-site hypochlorite generation system: A fifth 100 ppd unit will be needed if the plant is expanded to 22 mgd. Additional hypochlorite solution storage volume will be needed also.

7. Finished water pumping: Replace Pump No. 1 (2 mgd) with a 4 mgd pump and replace Pump No. 6 (4 mgd) with a 6 mgd pump to provide a firm capacity of 22 mgd.

8. Hydropneumatic surge control: Increase the capacity of the existing finished water transmission system to accommodate flows up to 22 mgd. Evaluate the existing hydropneumatic surge tank under the new hydraulic conditions. Replace the existing hydropneumatic surge tank with a larger tank if determined to be necessary.

9. Backwash waste and solids handling system: Increase the capacity of the system to accommodate flows up to 22 mgd. It is anticipated that an additional drying bed will be required. Due to the space constraints on the existing site, additional land will be required for this facility.

10. Electrical system upgrade: The plant's electrical supply and power distribution system will require substantial upgrades as the installed horsepower for the raw water and finished water pumps will exceed the existing system's capacity.

Recommended Implementation Schedule

General

As demonstrated in this plan, water demands are approaching the current 12 mgd capacity of the Winchester Water Treatment Plant. The City's Long-Range Water Supply Plan has projected near-term and long-term water demands and recommends proceeding immediately with expansion of the Winchester plant to 18 mgd. This capacity is estimated to meet the City's water demands until the year 2025. Figure 7-1 at the end of this section illustrates the existing plant site with the major recommended improvements shown.

The Long-Range plan further recommends expansion of the plant at that time to its maximum capacity of up to 22 mgd assuming the continued use of the present conventional treatment technologies at the plant and the availability of water rights and water supply in that amount at the Winchester site. The following are descriptions of the recommended implementation schedule for the recommended work described above.

Phase 1 - Plant Improvements and Expansion to 18 MGD – 2009 through 2012

- CT compliance
    - Conduct tracer study of existing clearwell to determine hydraulic efficiency
    - Update CT calculation methodology
    - Update operational protocols to achieve CT compliance under current flow conditions
    - Consult with Department of Human Services, Drinking Water Program regarding CT compliance, both current and in the future
    - Complete by October, 2009

- Monitoring program for *Cryptosporidium*
    - Continue with 2-year monitoring program to completion (test results to date indicate a bin classification #1 under LT2ESWTR regulations).

- Bench-top UV spectrophotometer
    - Utilize for total organic carbon measurement through treatment process
    - Acquire in 2009

- Shoaling condition in river at intake
    - Perform hydrographic survey
    - Complete river hydraulics analysis
    - Determine potential adverse impact to intake and operations

  - Develop action plan for recommended work including schedule, permitting and cost estimates
  - Prepare evaluation report
  - Complete all above tasks by end of 2009

- High service pump station roof
  - Replace roof
  - Complete by end of summer, 2009

- River intake pump testing
  - Test using City staff or private pump company
  - Rebuild pumps as necessary
  - Complete by end of summer, 2009

- Turbidimeters
  - Replace all existing units with new models
  - Complete in 2009

- Miscellaneous work as identified in Section 5, Facilities Condition Review
  - Complete by end of 2009

- Acquisition of additional property
  - Evaluate actions to secure City's existing water rights
  - Evaluate progress in obtaining additional water rights at Winchester
  - Evaluate progress in planning for development of other sources of supply at Winchester, specifically groundwater augmentation to the North Umpqua River
  - Establish future need for additional property at west boundary of plant by end of 2009
  - If decision positive, proceed with property acquisition program and complete in 2010

- Plant expansion program to 18 mgd
  - Preliminary work (public education, funding program, rate review and adoption, budget adoption) - July 2009 – June 2010
  - Commence final design – July 2010
  - Complete final design – January 2011
  - Advertise for bids – March 2011
  - Award construction contract – May 2011
  - Notice to proceed with construction – June 2011
  - Construction complete, facilities operational– December 2012

Phase 2 - Plant Improvements and Expansion up to 22 MGD – 2022 to 2025

- Plant expansion program from 18 mgd up to 22 mgd
    - Final design and construction completion by 2025 to expand plant capacity up to 22 mgd

Cost Estimates

Estimates of cost have been developed for the recommended work. For construction work, the estimated project costs are based upon recent experience with construction costs for similar work in the region and an evaluation and updating of the costs of construction of the original plant. It is assumed that construction work will be completed by private contractors. Construction cost estimates represent opinions of cost only, acknowledging that final costs of projects will vary depending on actual labor and material costs, market conditions for construction, regulatory factors, final project scope, project schedules, and other factors.

The estimated project costs for construction presented in this report include provisions for estimated construction costs plus allowances for construction contingencies, engineering, administration, permitting and approvals, and other project-related costs. An indexing method to adjust present estimates into the future is useful. The Engineering News Record (ENR) Construction Cost Index (CCI) is a commonly used index for this purpose. For purposes of cost estimate updating, the April 2009 ENR CCI for Seattle, Washington, the closest construction market index, is 8704.50.

For recommended engineering studies and related work, budget estimates are developed based upon the anticipated scope of work, preliminary budget estimates from service and materials suppliers, and upon general experience with similar work. Final costs will be governed by the final scopes of work and schedules.

Table 7-1 presents the estimated costs for the recommended regulatory compliance actions and other recommended immediate actions that are not included in the overall proposed Winchester plant improvement and expansion project.

TABLE 7-1

BUDGET ESTIMATES - REGULATORY COMPLIANCE AND OTHER IMMEDIATE RECOMMENDED ACTIONS (YEAR 2009)

Item	Estimated Budget, Current $
1. CT compliance review including tracer study & operator & DHS consultations	$8,000
2. Purchase UV spectrophotometer	$7,000
3. Property acquisition (portion of Tax Lot 800)	$350,000
4. Evaluation study of shoaling at river intake	$12,000
5. Replace roof of high service pump station	$25,000
6. Test and rebuild river intake pumps	$55,000
7. Replace turbidimeters	$20,000
8. Miscellaneous improvements per Section 5	$10,000
Total Estimated Budget	**$487,000**

Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

TABLE 7-2

PROJECT COST ESTIMATE PHASE 1 – PLANT IMPROVEMENTS AND EXPANSION TO 18 MGD (YEARS 2009-2012)

Item	Estimated Cost, Current $
Estimated Construction Costs	
Fish screening at river intake	$690,000
Raw water pumping improvements	$102,000
Flocculation & sedimentation basin no. 2	$1,407,000
Filters 5 & 6	$1,833,000
Additional clearwell baffling	$300,000
Finished water pumping improvements	$429,000
Hydropneumatic surge system upgrading	$186,000
Total Estimated Direct Construction Cost	$4,947,000
Construction Contingency (15%)	$742,000
Total Estimated Construction Cost	$5,689,000
Allowance for Inflation (2 years - 3%/yr. – 6% total)	$341,000
Total Estimated Construction Cost With Inflation Allowance	$6,030,000
Estimated Indirect Costs	
Design Engineering (15%)	$904,000
Construction Engineering (10%)	$603,000
Administration, Legal, Permits & Approvals (1%)	$60,000
Total Estimated Indirect Costs	$1,567,000
Total Estimated Project Cost	**$7,597,000**

Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

Table 7-2 presents the estimated project costs for the initial Winchester plant improvement and expansion phase work to be accomplished from 2009 through 2012. This phase will expand the plant to 18 mgd capacity. The estimate in this table includes an inflation allowance.

Table 7-3 presents the estimated project costs for the second plant improvement and expansion phase work to be accomplished from 2022 through 2025. This phase expands the plant from 18 mgd to as much as 22 mgd. Since the scope of the work required for this expansion is only generally defined, the costs presented are conceptual level cost estimates. The costs presented do not include an inflation allowance.

TABLE 7-3
PROJECT COST ESTIMATE
PHASE 2 – PLANT IMPROVEMENTS AND EXPANSION UP TO 22 MGD (YEARS 2022 - 2025)

Item	Estimated Cost, Current $
Estimated Construction Costs	
Raw water pumping improvements	$425,000
Remove & replace underdrains on 4 filters & remove & replace media in 6 filters	$540,000
Construct 1.25 million gallon clearwell addition	$2,500,000
Expand on-site hypochlorite generation system	$125,000
Finished water pumping improvements	$290,000
Backwash waste and solids handling system	$675,000
Electrical power supply & distribution system upgrade	$350,000
Total Estimated Direct Construction Cost	$4,905,000
Construction Contingency (25%)	$1,226,000
Total Estimated Construction Cost	$6,131,000
Estimated Indirect Costs	
Design Engineering (15%)	$920,000
Construction Engineering (10%)	$613,000
Administration, Legal, Permits & Approvals (1%)	$61,000
Total Estimated Indirect Costs	$1,594,000
Total Estimated Project Cost	**$7,725,000**

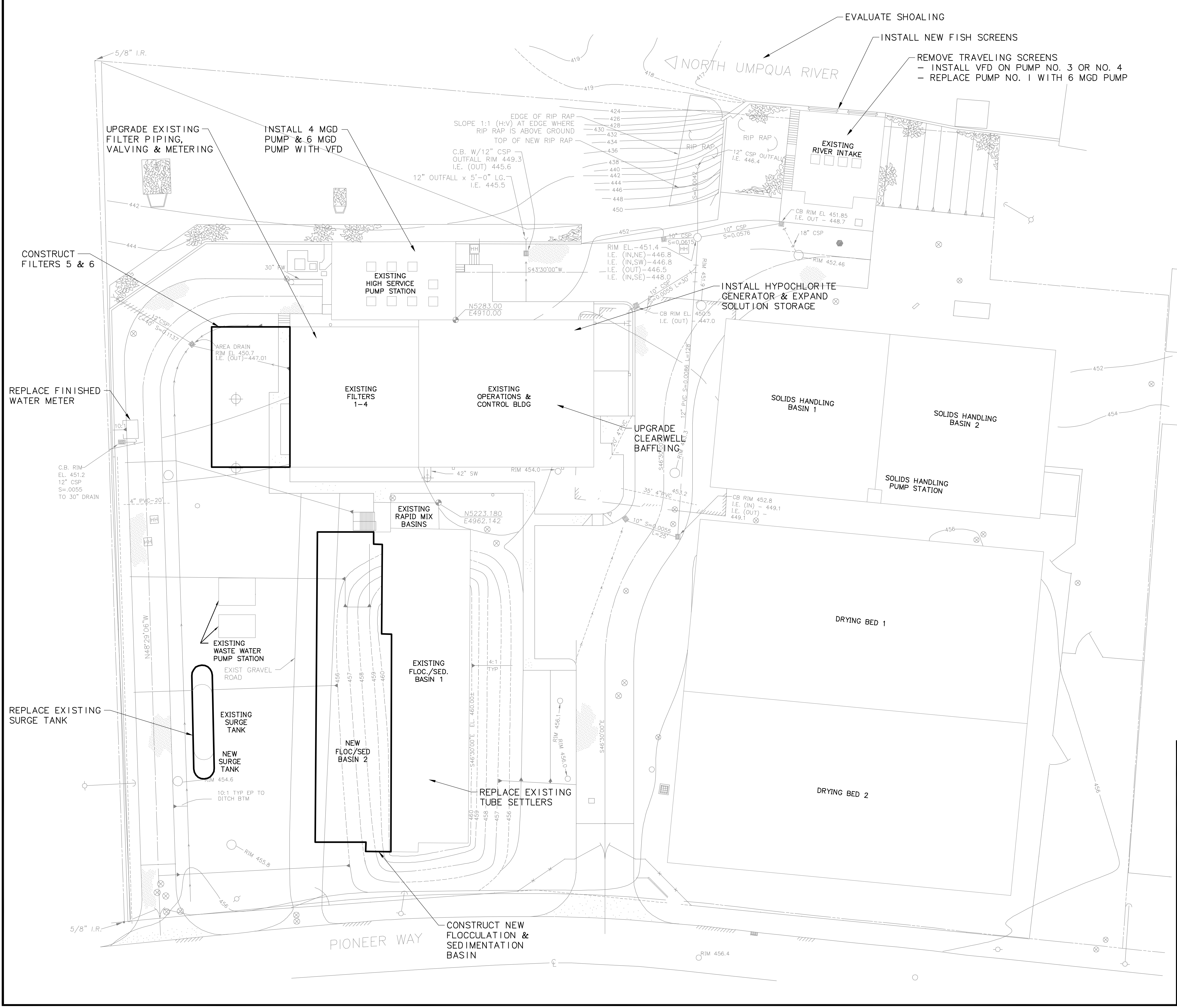
Note: Cost estimates based upon ENR Construction Cost Index (Seattle) of 8704.50, April 2009.

Conclusions

This preliminary design report evaluated the historical performance of the City's existing Winchester Water Treatment Plant. The report also reviewed the current and anticipated regulations governing water treatment, performed a hydraulic and treatment capacity review, evaluated the condition of the existing plant and developed a capital maintenance plan, and analyzed alternatives for critical processes. Recommendations for plant upgrading, improvements and expansion are then made and a plan to implement the recommendations is proposed. It is recommended that immediate actions consisting of plant repairs, improvements, further evaluations, and property acquisition be undertaken in 2009 and 2010. It is further recommended that the program to expand the plant to 18 mgd be undertaken in 2009 with the expansion completed by the end of 2012. It is recommended that, by approximately the year 2025, the plant be further expanded up to 22 mgd which is its approximate maximum ultimate capacity using rapid sand filtration technology.

Plan Adoption

It is recommended that the City of Roseburg adopt this preliminary design report for the City's Winchester Water Treatment Plant to guide improvements to and expansion of the plant.



1 Inch = 250 Feet

OTHER MAJOR WORK

- UPGRADE SCADA SYSTEM
- UPGRADE SECURITY SYSTEM
- UPGRADE WATER QUALITY MONITORING & CONTROL SYSTEMS

Figure 7-1

CITY OF ROSEBURG, OR

WATER TREATMENT FACILITIES
PRELIMINARY DESIGN REPORT

SITE PLAN -
PHASE 1 PLANT EXPANSION TO 18 MGD

JULY 2009

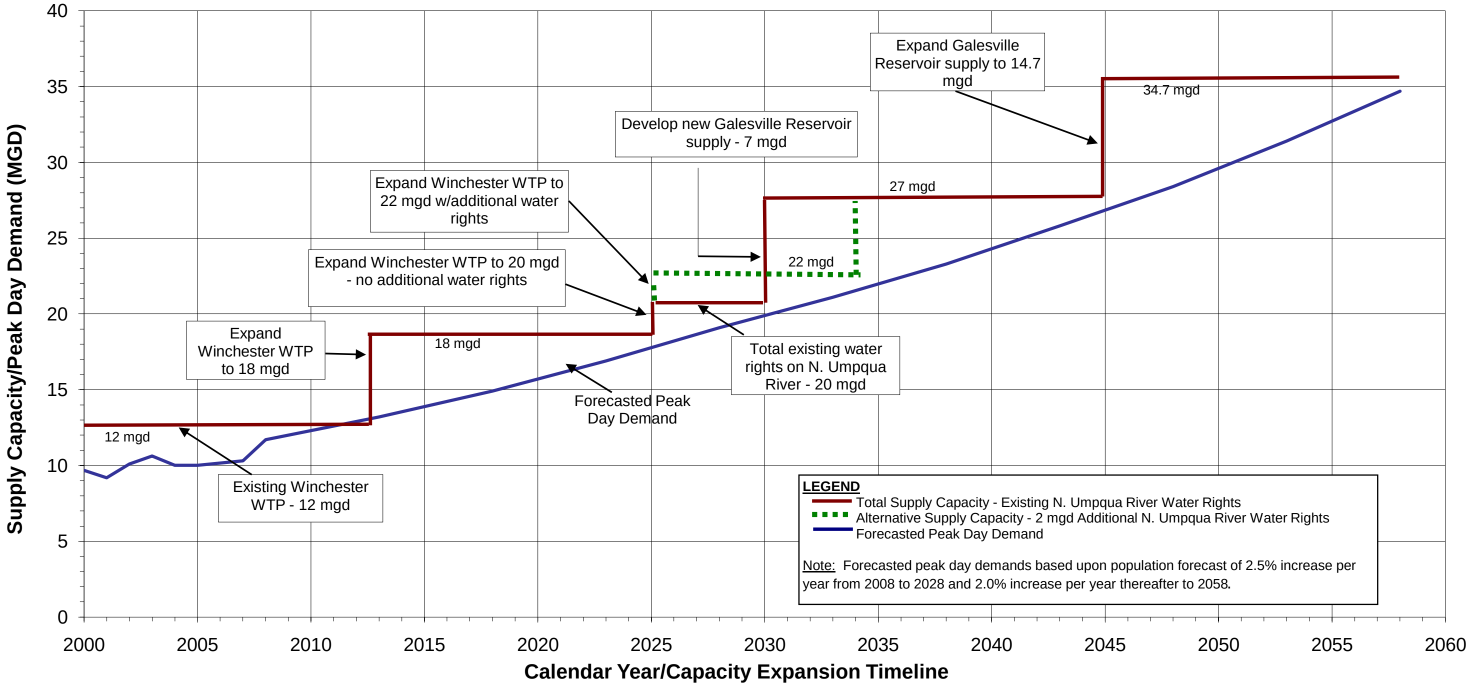
MSA Murray, Smith & Associates, Inc.
Engineers/Planners
121 S.W. Salmon, Suite 900 Portland, Oregon 97204
PHONE 503-225-9010 FAX 503-225-9022

G:\09\1015\401\CAD\09-1015-401-OR-FIG 4-X.dwg FIGURE 7-1 6/23/09 15:04 (DAK)

APPENDIX

APPENDIX A

Figure 7-1
City of Roseburg
Water Demand and Water Supply Schedule



Note: Forecasted peak day demands based upon population forecast of 2.5% increase per year from 2008 to 2028 and 2.0% increase per year thereafter to 2058.

\\Archive1\archives\2009\1015\401\Report Updates\Long-Range Water Supply Plan Update 6-2009\FINAL REPORT JULY 2009\Demand-Supply Capacity Figure 7-1.xls

APPENDIX B

S&B inc. **13200 SE 30th St., Bellevue, Washington 98005 (425) 644-1700 FAX (425) 746-9312**

TECHNICAL MEMORANDUM

City of Roseburg Water Treatment Plant Instrumentation and Control Systems

PREPARED FOR: Murray Smith & Associates

PREPARED BY: Randy Stead, P.E.

DATE: October 20, 2006

PROJECT: Water Treatment Plant Expansion

Background

The Instrumentation & Control (I&C) System at the City's Winchester Water Treatment Plant provides control for the pumping and treatment systems within the plant. The system was designed and installed between 1991 and 1992 as part of the City's WTP project. The design approach used for the I&C system complied with the P&ID design drawings published by Murray Smith & Associates for this project.

The I&C System discussed as part of this Tech Memo includes a programmable logic controller (PLC) based automation system with a SCADA graphic computer front end that provides for operator interaction in the auto control setpoints and for visualization of the main process areas. The control system is centralized at this time, with the plant's master control panel (MCP) having wire terminations for almost every device in the plant. The MCP is located in the control room and situated immediately next to the City's telemetry system panel. The MCP is 108" wide and the telemetry panel is 72" wide, for an overall width of fifteen feet. Presentation Drawings showing these panels are attached as a reference.

The automation portion of the control system is based on a Siemens 545 PLC unit and 505 series input / output modules. This unit has the necessary software control features required for both in plant control and SCADA functions required for telemetry systems. The PLC was installed using about 70% of its available programming capacity and after the 14 years of expansion and changes, the unit is at 91% capacity. The input / output capacity of the system is limited to a maximum 2000 digital and 1000 analog points. Current configuration has the system using 648 digital points and 64 analog points of data.

The graphic computer system was upgraded twice since the 1992 plant commissioning, first moving off a DOS based system as part of a Year 2000 computer upgrade and then again as a minor upgrade this past year to replace aging computer equipment. The SCADA software is current technology and provides what the process industry calls "visualization" for each of the major process areas in the plant and in off-site reservoir and pump stations. A second computer acts as a data historian, summarizing and storing data in hourly and daily formats. The computer software has not been changed since the plant installation, though the computer hardware has been upgraded. Plant data is stored a minimum of five years in the data historian, with older data only available on tape backup.

Focus

This Tech Memo addresses the automation system and its usability into the future with the planned WTP expansion. Typical life-span for automation and control systems are 15 to 20 years. As the existing plant undergoes changes required for additional capacity, a plan is necessary for the control system too, both in transition and as for the completed project. The result for the Winchester WTP must include changes to the I&C that will provide an extended life expectancy commiserate with the technology available in 2006.

This Tech Memo does not address equipment within each of the process areas in the plant. For instance, the I&C system includes flow paced automatic control signals for the chemical feed systems, but this document does not address suitability of the existing chemical feed equipment to meet the new demands of the expanded plant.

Recommendations related to upgrades and expansion in the I&C system herein are based on keeping all I&C systems functional during the transition. Equipment changeovers, where necessary are designed for implementation in a four hour plant shutdown.

Process Control Changes

Initial information provided by MSA indicate the following changes to the process control:

- Minimum upsize of plant capacity from 12MGD to 18MGD
- Revision of Raw Water pumping system to include up to two adjustable speed drives
- Addition of a second Flocculation/Sedimentation Basin
- Addition of two filters
- Changes to the Chemical feed related to MIOX and hypo-chlorination
- Addition of two High Service pumps with adjustable speed drives.
- Improved Water Quality Instrumentation

The current I&C system provides automatic control for each of these process areas. Modifications will require careful integration and coordination to keep the operation ‘look and feel’ compatible between new and existing systems.

Suitability of Existing Systems in the Expanded Plant Control System

With a plant expansion, the operator will desire to have a uniform look and feel for the control system such that the same features are available for new components and older components. In most cases for the Winchester plant, the ‘look and feel’ can be identical and the new process control features seamlessly added in the control room.

A primary concern for designing an addition to the control system is the viability of the existing system to meet the new demands and the remaining service life available. The WTP's existing PLC system is a mature product line of Siemens Energy and Automation. This line was moved to "mature status" in February 2005 after twenty years of production. Siemens guarantees full support for this product line with spare parts until at least 2015. Siemens cites that while the 505 product line was very successful in US industry, it did not have a place in their world market. Control Technology Inc is an automation company based in Tennessee now designing and building new parts for the 505 line, including input/output (I/O) modules used in the Winchester WTP. The modules are completely interchangeable with the Siemens parts such that the City may select from either Siemens or CTI for replacement parts. The 505 line is a current production platform for CTI and they have no plans at this time to move their line to mature status.

The existing I/O monitors key process information needed in the expanded plant. The mean time before failure (MTBF) rating of the I/O modules range from 34 years to as high as 90 years for the 24Vdc units used throughout the WTP system. Based on the remaining lifespan of a minimum of twenty years and the availability of parts during this period, there is not a compelling reason to replace the I/O because of equipment durability.

The SCADA computer system is a combination of a Siemens WinCC software package and a S&B Data Management System (DMS), each configured as a "standalone" system. The WinCC graphic system provides basic control setpoint entry for the operator within each process area of the plant. The computer operation is necessary for the operator to make changes in plant flow, chemical feed and pump operation while in automatic. The plant does not require computer operation for manual control. The WinCC package is current, but the development is only basic. The package may be expanded to include better trending, report writing, and alarm logging features. Current these features are provided by the DMS, but they will not be available in the future on this platform. The power of the SCADA software packages has been sufficient for seven years to eliminate the DMS functionality, and the DMS has been a mature product for six years. As part of the expansion project, the DMS functionality must be transferred to the WinCC software.

The panel control switches were as manufactured by Honeywell Microswitch as their CMC series units. These units provide both switch and lamp indication on a single device. The switch line was sold five years ago to Senasys and are still in production. Lifespan for switches is based on use and age, and we estimate the WTP switches' service life at thirty years. The switch is unique in the industry in the configuration options for switch position and in conservation of panel space, but this always commands a premium in price. With two additional filters planned and up to two additional High Service Pumps that will be added to the MCP, it makes sense for continuity to continue with the CMC switches. Lamps used to indicate status conditions are a high maintenance item for the control panel, with a bulb typically lasting only six to nine months. LED replacement bulbs are now available for the CMC type modules that will last five years before the intensity of the lamp is no longer acceptable. LED lamps cost $8 to $10 each, about 10 times more than incandescent lamps and are a cost neutral decision. Dependability and lower power demand on the 24Vdc bus is the primary reason we recommend updating lamp assemblies.

Circular chart recorders are in place on the MCP with a seven day trend capability. In 1992 chart recorders were a familiar face and friend to operators, a carryover from past control rooms and necessary for operation in many old WTP control systems. The chart recorders have a typical lifespan of fifteen years and require periodic maintenance every five years. ABB still offers a chart recorder in the same size and feature configuration. The City replaced at least two recorders in recent years and it should be expected to replace all remaining units as part of the WTP expansion project in order to sustain the lifespan of the new control system. During the design phase of this project, a decision should be made whether to move the trending and reporting functions currently provided by the chart recorders into a SCADA report. Chart recorders with three pens cost approximately $3,500 each installed and configured.

Digital indicators used on the MCP are no longer available and replacement units are slightly larger (3/8”) than the size in place on the existing panel. The physical appearance of the panel will not be negatively affected by use of newer indicators. Depending on how the SCADA computer system is expanded, the need for digital indicators may be reduced or eliminated.

Digital and analog connections to the control system for indication and monitoring are typically 24Vdc to reduce electrical noise and interference problems. The 24Vdc power supplies in use at the MCP should be replaced as part of the expansion since they are critical for operation, at the end of their expected lifespan and are relatively inexpensive.   Wiring is terminated in the MCP, organized by process area with moderate to low amounts of space available for expansion.

Advancements in Control System Technology

Additional features found in new processors include a wide range of communications for data exchange with process instruments, motor controllers and distributed I/O devices. The communication options will be an important feature for the plant expansion.

There are several open communication standards for process control available for deployment in the automation industry. The network communication is known as fieldbus technology and there have been as many as twelve different forms of this available. Profibus is the world’s largest and most widely recognized protocol with offerings in cable, fiber and Cat6 wire infrastructure. Typical communication rates are between 187k baud and 100M baud. Using an open communication standard allows equipment from different electrical, instrumentation and automation manufacturers to work together under a single network and process controller.

While Profibus is useful for in plant operation, other serial and half-duplex communication standards must be used for telemetry functions between the WTP and remote reservoir and pump station facilities. Typical data communication rates for telemetry using Modbus communication are 1200 to 9600 baud, depending on the leased line or radio system performance.

The selected PLC processor for the plant expansion should have multiple communication protocols available for interconnection and also provide linkage to legacy I/O devices within the MCP. Where more than four logic connections are required to a single piece of equipment, it is typically more cost effective to use a communication link instead of discrete wiring. Valve controllers, motor starters, VFD units and process instruments may need to be evaluated based on communication options available in addition to conventional performance specifications. Some devices include a control/monitoring communication network feature as part of the base price and others provide for this as an optional accessory. The network connection provides a more complete set of data and diagnostic tools for operator and maintenance personnel to use in understanding device behavior and in resolving problems.

New processors also include the expected faster and smaller features associated with computer technology over the past fourteen years. The selection of the processor is made by evaluating immediate processing needs and anticipated growth over the next seven to ten years. Siemens S7-300 or S7-400 series units are the most compatible with the existing technology and the best candidate for replacement of the 545 processor since they are a certified and supported upgrade path for 505 I/O integration and software adaption. Other manufacturers have capable replacement units as well and should be considered if the cost, performance or risk areas of evaluation find a better solution. Brands that can provide this alternative path for the existing infrastructure include CTI and VIPA.

Recommendations for Control System Improvements

The plant expansion requirements will mandate approximately 20% more capacity in the plant processing unit. The existing unit is not large enough to provide this functionality and lacks many important communication options required for new generation equipment. This together with the technology changes in processors over the past fourteen years, will drive a selection of a new plant CPU.

At this same time of replacing the central processing unit (CPU) to the control system, there is not a compelling reason to replace existing control system infrastructure in switches, lamps and I/O wiring. The bulk of the existing plant process will remain intact during the expansion project and no significant benefit will likely be found in replacing these components. The limitations of existing conduits and wiring within the plant will make remote I/O panels, smart motor controllers and intelligent instruments a cost effective solution since they will reduce or eliminate “home run” wiring paths and overcrowding in the termination area of the existing MCP. The data cabling is compatible with the existing low voltage conduit system such that there will be little or no requirements for conduit additions in the control room.

A distribution I/O panel can serve the control system expansion needs for several areas within the plant. These panels save installation costs by regionalizing wire/conduit runs within each room for low wire density devices or equipment that does not have a higher level communication provision. We anticipate this needed for the new filters, new sedimentation basin and possibly for the water quality analyzers. The MCP should be modified to match existing switch and lamp implementation for operator familiarity, but all interaction between the MCP and the field device would be over the digital network.

Motor starter and/or VFD additions in the Raw Water Pump Station and High Service Pump Station may be direct connected to the plant PLC, requiring only that the data cable connection be extended to the device. This technology is already in place at the newer pumping stations the City has placed in service including Winchester Creek and Eagle View. The data from the starter provides on-line energy data that when combined with pressure and flow information will provide real time efficiency values for each pump system.

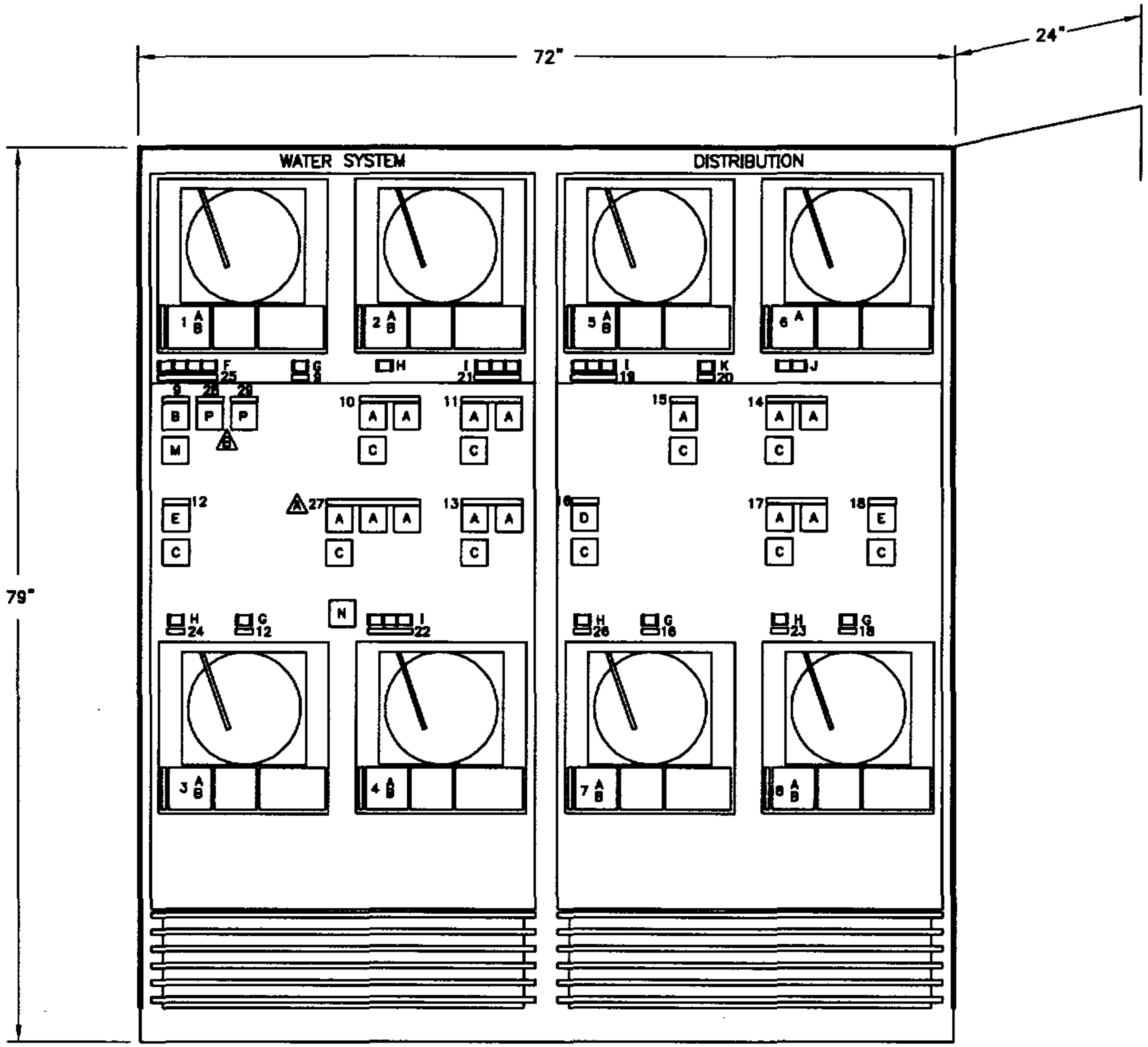
The SCADA computer system is a critical link in the automatic operation of the plant. The SCADA computer is the most likely major component to fail at the WTP. Desktop computers have a three to five year life cycle and are not robust.   As a minimum, a second computer system should be set up as a redundancy. In addition to the second computer, different configurations are available that provide the ability to link many client machines to the SCADA network, both locally and remotely using secure VPN connections. The WinCC software is compatible with all options and can be adapted as necessary to meet the levels of redundancy and features desired by the City.

Trending using the chart recorders can be transferred to the SCADA computer. Trending can be provided in either digital file format Adobe pdf format or directly to color laserjet for about the price of a single circular chart recorder. This change is an operator comfort decision, and we suspect that most if not all the trending could be better served with SCADA.

The DMS computer system requires replacement as part of this expansion project. The SCADA computer system can assume the report functions and historical archiving as an add-in module. The WinCC historian uses Microsoft's SQL database for record archiving such that it is on an open platform for data exchange. All historical data in the DMS may be exported in common csv file format for future reference. This csv file format is fully compatible with Excel, Access and several other databases.

Further Action

As this project moves into design issues, a clear understanding of the role of the control system should be defined early. Our firm is available to answer questions and assist the Engineer during the design phase with conceptual drawings and operating descriptions to help define the role of the control system in the expanded plant.



LEGEND:

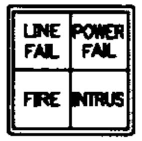
1. (A) W. MILITARY AVE LEVEL
   (B) BROCCOLI ST. PRESSURE
2. (A) KLINE ST LEVEL
   (B) FAIRHILL DR. LEVEL
3. (A) STACEY CT. LEVEL
   (B) STACEY CT. HP PRESSURE
4. △ (A) QUARRY ST. LEVEL
   (B) MT. ROSE LEVEL
5. (A) RESERVOIR #1 & #2 LEVEL
   (B) RESERVOIR #7 LEVEL
6. (A) RESERVOIR AVE LEVEL
7. (A) TERRACE DR. RES
   (B) TERRACE DR. HP PRESSURE
8. (A) NORA LANE LEVEL
   (B) GIBBY HP PRESSURE
9. BROCCOLI ST. VALVE
10. KLINE ST. PS
11. FAIRGROUNDS PS
12. STACEY CT. HP
13. △ GARDEN VALLEY PS
14. RESERVOIR AVE PS
15. HAWTHORNE PS
16. TERRACE DR. HP
17. DENN NORA PS
18. GIBBY HP
19. RESERVOIR #1 & #2
20. RESERVOIR #7
21. FAIRHILL DR. RESERVOIR
22. △ MT. ROSE RESERVOIR
23. NORA LANE / RESERVOIR
24. STACEY CT. / RESERVOIR
25. WEST MILITARY RD. RESERVOIR
26. TERRACE DR. / RESERVOIR
27. △ QUARRY ST. RESERVOIR & P.S.
28. △ WINCHESTER
29. △ EAGLE VIEW

NOTES:

1. MATERIAL: CABINET 14 GA. CRS, PANEL AND DOOR 12 GA. CRS. ALL WELDED CONSTRUCTION WITH EDGES AND SEAMS GROUND SMOOTH.
2. FINISH: CABINET, PANEL AND DOOR NITRO BLUE #5210109 BAKED ENAMEL. ALL CABINET EXTERIOR SURFACES SMOOTH AND FREE OF IMPERFECTIONS.
3. GRILL AND TRIM ANODIZED BRUSHED ALUMINUM WITH REPLACEABLE DUST FILTER BEHIND GRILL.
4. CABINET DOORS ARE REAR ACCESS.

DETAIL A

DETAIL B



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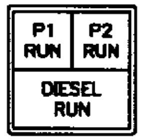

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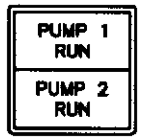

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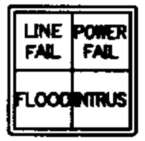

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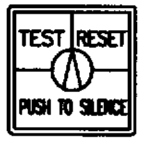

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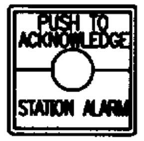
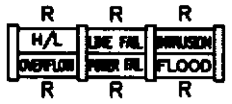


DETAIL P △

DETAIL F

DETAIL G

DETAIL H



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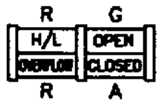

DETAIL J

DETAIL K

DETAIL L

DETAIL Q △

REV.	REVISION DESCRIPTION	APP	DATE
△	ADDED WINCHESTER & EAGLE VIEW PER 20241,20385	JES	5/06
△	PROPOSAL MODIFICATIONS FOR NEW RESERVOIR/P.S.	RTS	7/96

DRWN JRB 9/10/91 ASMB ENGR REB 5/2/92 SCALE NTS

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LAST MODIFIED: 05/15/06 2:32 PM

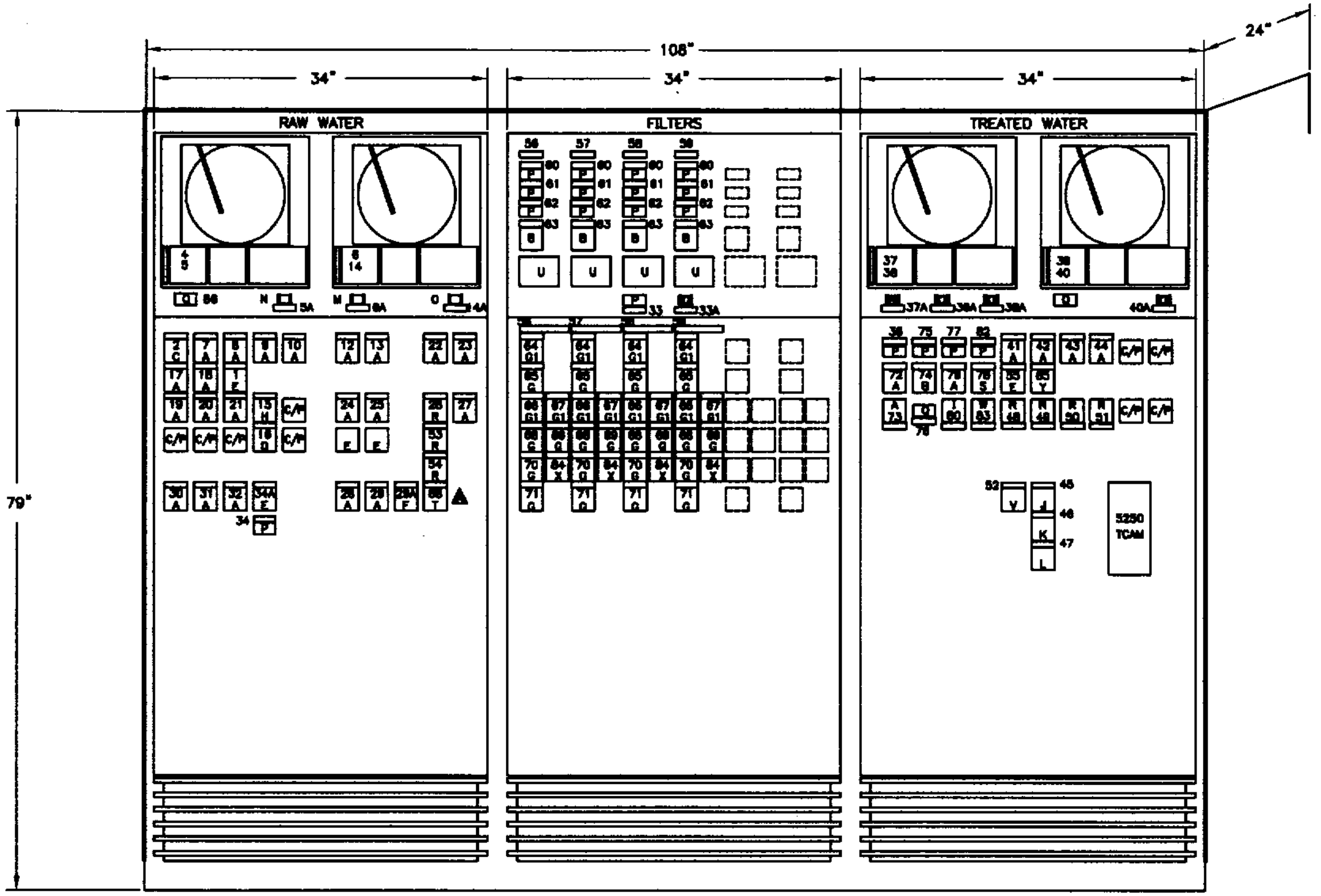
S&B Systems Specialists
S&B Inc.

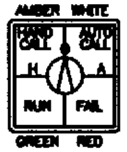
PROJECT
CITY OF ROSEBURG, OR

TITLE
PRESENTATION
TELEMETRY MASTER PANEL

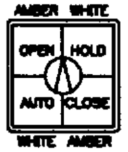
DRAWING NUMBER

SIZE	JOB NUMBER	KEY	SHEET	REV
D	13533	100	1 OF 2	B





DETAIL A



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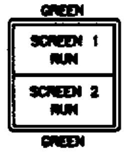

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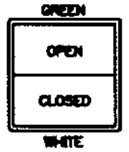
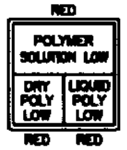


DETAIL D

DETAIL E

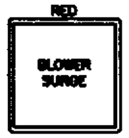


DETAIL F

DETAIL G

DETAIL G1

DETAIL H



DETAIL I

DETAIL J

DETAIL K

DETAIL L

DETAIL M

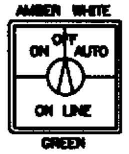
DETAIL N

DEYAIL Y

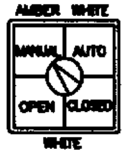
DETAIL O

DETAIL P

DETAIL Q



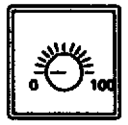
DETAIL R



DETAIL S

DETAIL T

DETAIL V

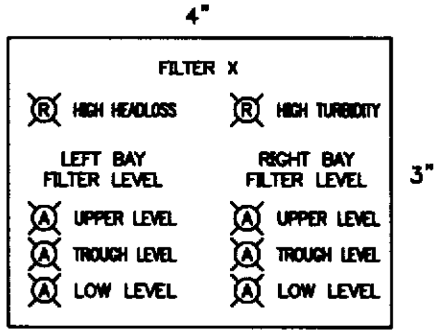


DETAIL W

DETAIL X

LEGENDS:

1. RIVER INTAKE/WATER LEVEL/ALARMS
2. TRAVELING SCREENS
4. RAW WATER FLOW / MGD (RED)
5. RAW WATER / TURBIDITY – NTU (GREEN)
5A. TURBIDITY HIGH ALARM
6. RAW WATER TEMPERATURE (RED)
6A. TEMPERATURE ALARM
7. RAW WATER / PUMP NO. 1
8. RAW WATER / PUMP NO. 2
9. RAW WATER / PUMP NO. 3
10. RAW WATER / PUMP NO. 4
11.
12. RAW WATER / SAMPLE PUMP
13. COAGULATED WATER SAMPLE PUMP
14. STREAMING CURRENT (GREEN)
14A. STREAMING CURRENT ALARM
15. SLUDGE / COLLECTOR
16. SLUDGE / TRANSFER / VALVE
17. RAPID / MIXER NO. 1
18. RAPID . MIXER NO. 2
19. FLOCCULATOR / NO. 1
20. FLOCCULATOR / NO. 2
21. FLOCCULATOR / NO. 3
22. ALUM FEED / PUMP NO.1
23. ALUM FEED / PUMP NO.2
24. DRY CHEMICAL / FEEDER NO. 1
25. DRY CHEMICAL / FEEDER NO. 2
25A.
26. CHLORINE / FEED
27. CARBON / FEEDER
28. CL2 PUMP NO. 1
29. POLYMER / FEED PUMP NO. 2
29A. POLYMER MIXING UNIT ALARMS
30. WASTEWATER / PUMP NO. 1
31. WASTEWATER / PUMP NO. 2
32. WASTEWATER / PUMP NO. 3
33. FILTER INFLUENT / CHANNEL LEVEL
33A. INFLUENT CHANNEL ALARM
34. WASTEWATER / PUMP STATION / WET WELL LEVEL
34A. WASTEWATER WET WELL ALARM
36. CLEARWELL LEVEL / FEET
36A. CLEARWELL LEVEL ALARM
37. FINISHED WATER / TURBIDITY / NTU (RED)
37A. TURBIDITY ALARM HIGH
38. CHLORINE RESIDUAL / PPM (GREEN)
38A. CHLORINE RESIDUAL ALARM
39. FINISHED WATER FLOW / MGD (RED)
40. HIGH SERVICE PUMP STATION / DISCHARGE PRESSURE – PSI (GREEN)
40A. DISCHARGE PRESSURE ALARM
41. FINISHED WATER / PUMP NO. 1
42. FINISHED WATER / PUMP NO. 2
43. FINISHED WATER / PUMP NO. 3
44. FINISHED WATER / PUMP NO. 4
45. ALARM ACKNOWLEDGE
46. OUTSIDE ALARM HORN
47. ALARM DIALER
48. FILTER NO. 1 / CHLORINE FEED
49. FILTER NO. 2 / CHLORINE FEED
50. FILTER NO. 3 / CHLORINE FEED
51. FILTER NO. 4 / CHLORINE FEED
52. PLANT ALARM
53. CL2 PUMP NO. 2
54. CL2 PUMP NO. 3
55. SURGE TANK WATER LEVEL ALARMS
56. FILTER NO. 1
57. FILTER NO. 2
58. FILTER NO. 3
59. FILTER NO. 4
60. LOSS OF HEAD / FEET
61. TURBIDITY / NTU
62. FLOW / MGD
63. EFFLUENT VALVE
64. INLET VALVE
65. BACKWASH ISOLATION VALVE
66. BACKWASH LEFT BAY VALVE
67. BACKWASH RIGHT BAY VALVE
68. AIR WASH LEFT BAY VALVE
69. AIR WASH RIGHT VALVE
70. BACKWASH WASTE VALVE
71. FILTER TO WASTE VALVE
72. BACKWASH PUMP NO. 1
73. BACKWASH PUMP NO. 2
74. BACKWASH CONTROL VALVE
75. BACKWASH FLOW / MGD
76. BACKWASH TOTAL
77. AIR FLOW / CFM
78. AIR VALVE
79. BACKWASH AIR BLOWER
80. BLOWER ALARM
81. CLOCK
82. AIR VALVE POSITION
83. MANUAL CONTROL / AIR WASH VALVE
84. AUTOMATIC BACKWASH / SELECTOR
85. SURGE TANK AIR COMPRESSOR
86. RAW FLOW TOTAL X1000
87. FINISHED FLOW TOTAL X1000
▲88. MIOX SYSTEM



DETAIL U

NOTES:

1. MATERIAL: CABINET 14 GA. CRS, PANEL AND DOOR 12 GA. CRS. ALL WELDED CONSTRUCTION WITH EDGES AND SEAMS GROUND SMOOTH.

2. FINISH: CABINET, PANEL AND DOOR NITRO BLUE #5210109 BAKED ENAMEL. ALL CABINET EXTERIOR SURFACES SMOOTH AND FREE OF IMPERFECTIONS.

3. GRILL AND TRIM ANODIZED BRUSHED ALUMINUM WITH REPLACEABLE DUST FILTER BEHIND GRILL.

4. CABINET DOORS ARE REAR ACCESS.

REV.	REVISION DESCRIPTION	APP	DATE
▲	ADDED MIOX SYSTEM	JES	5/06

DRWN JEK 9/9/91 ASMB ENGR SCALE NTS

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PROJECT
CITY OF ROSEBURG, OR

TITLE
TREATMENT MASTER PANEL

DRAWING NUMBER

SIZE	JOB NUMBER	KEY	SHEET	REV
D	13533	101	1 OF 2	A

APPENDIX C

APPENDIX C. CT VALUES FOR INACTIVATIONS ACHIEVED BY VARIOUS DISINFECTANTS

This appendix provides a reprint of the CT tables for determining inactivations achieved by various disinfectants. These tables were originally provided in EPA's *Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Sources* (AWWA, 1991).

Table C-1. CT Values for Inactivation of *Giardia* Cysts by Free Chlorine at 0.5°C or Lower

CHLORINE CONCENTRATION	pH<=6 Log Inactivation						pH=6.5 Log Inactivation						pH=7.0 Log Inactivation						pH=7.5 Log Inactivation					
(mg/L)	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	169	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	155	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	169	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	pH=8.0 Log Inactivation						pH=8.5 Log Inactivation						pH=9.0 Log Inactivation					
(mg/L)	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	113	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	55	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	369	460	552

Source: AWWA, 1991.

Table C-2. CT Values for Inactivation of Giardia Cysts by Free Chlorine at 5°C

CHLORINE CONCENTRATION	pH<=6 Log Inactivation						pH=6.5 Log Inactivation						pH=7.0 Log Inactivation						pH=7.5 Log Inactivation					
(mg/L)	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	49	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	97	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	69	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	pH=8.0 Log Inactivation						pH=8.5 Log Inactivation						pH=9.0 Log Inactivation					
(mg/L)	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	197	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	189	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

Source: AWWA, 1991.

Table C-3. CT Values for Inactivation of Giardia Cysts by Free Chlorine at 10°C

CHLORINE CONCENTRATION (mg/L)	pH<=6						pH=6.5						pH=7.0						pH=7.5					
	Log Inactivation						Log Inactivation						Log Inactivation						Log Inactivation					
	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	12	24	37	49	61	73	15	29	44	59	73	88	17	35	52	69	87	104	21	42	63	83	104	125
0.6	13	25	38	50	63	75	15	30	45	60	75	90	18	36	54	71	89	107	21	43	64	85	107	128
0.8	13	26	39	52	65	78	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131
1	13	26	40	53	66	79	16	31	47	63	78	94	19	37	56	75	93	112	22	45	67	89	112	134
1.2	13	27	40	53	67	80	16	32	48	63	79	95	19	38	57	76	95	114	23	46	69	91	114	137
1.4	14	27	41	55	68	82	16	33	49	65	82	98	19	39	58	77	97	116	23	47	70	93	117	140
1.6	14	28	42	55	69	83	17	33	50	66	83	99	20	40	60	79	99	119	24	48	72	96	120	144
1.8	14	29	43	57	72	86	17	34	51	67	84	101	20	41	61	81	102	122	25	49	74	98	123	147
2	15	29	44	58	73	87	17	35	52	69	87	104	21	41	62	83	103	124	25	50	75	100	125	150
2.2	15	30	45	59	74	89	18	35	53	70	88	105	21	42	64	85	106	127	26	51	77	102	128	153
2.4	15	30	45	60	75	90	18	36	54	71	89	107	22	43	65	86	108	129	26	52	79	105	131	157
2.6	15	31	46	61	77	92	18	37	55	73	92	110	22	44	66	87	109	131	27	53	80	107	133	160
2.8	16	31	47	62	78	93	19	37	56	74	93	111	22	45	67	89	112	134	27	54	82	109	136	163
3	16	32	48	63	79	95	19	38	57	75	94	113	23	46	69	91	114	137	28	55	83	111	138	166

CHLORINE CONCENTRATION (mg/L)	pH=8.0						pH=8.5						pH=9.0					
	Log Inactivation						Log Inactivation						Log Inactivation					
	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	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

Source: AWWA, 1991.

Table C-4. CT Values for Inactivation of Giardia Cysts by Free Chlorine at 15°C

CHLORINE CONCENTRATION (mg/L)	pH<=6 Log Inactivation						pH=6.5 Log Inactivation						pH=7.0 Log Inactivation						pH=7.5 Log Inactivation					
	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 Inactivation						pH=8.5 Log Inactivation						pH=9.0 Log Inactivation					
	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	59	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	109	135	162	33	65	98	130	163	195

Source: AWWA, 1991.

Table C-5. CT Values for Inactivation of Giardia Cysts by Free Chlorine at 20°C

CHLORINE CONCENTRATION (mg/L)	pH<=6						pH=6.5						pH=7.0						pH=7.5					
	Log Inactivation						Log Inactivation						Log Inactivation						Log Inactivation					
	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	9	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						pH=8.5						pH=9.0					
	Log Inactivation						Log Inactivation						Log Inactivation					
	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	19	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	139
2.6	16	32	49	65	81	97	20	39	59	78	98	117	24	47	71	94	117	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

Source: AWWA, 1991.

Table C-6. CT Values for Inactivation of Giardia Cysts by Free Chlorine at 25°C

CHLORINE CONCENTRATION (mg/L)	pH<=6						pH=6.5						pH=7.0						pH=7.5					
	Log Inactivation						Log Inactivation						Log Inactivation						Log Inactivation					
	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	13	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	19	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						pH=8.5						pH=9.0					
	Log Inactivation						Log Inactivation						Log Inactivation					
	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	19	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	89
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

Source: AWWA, 1991.

Table C-7. CT Values for Inactivation of Viruses by Free Chlorine, pH 6.0-9.0

	Temperature (°C)																									
Inactivation (log)	**0.5**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
2	6.0	5.8	5.3	4.9	4.4	4.0	3.8	3.6	3.4	3.2	3.0	2.8	2.6	2.4	2.2	2.0	1.8	1.6	1.4	1.2	1.0	1.0	1.0	1.0	1.0	1.0
3	9.0	8.7	8.0	7.3	6.7	6.0	5.6	5.2	4.8	4.4	4.0	3.8	3.6	3.4	3.2	3.0	2.8	2.6	2.4	2.2	2.0	1.8	1.6	1.4	1.2	1.0
4	12.0	11.6	10.7	9.8	8.9	8.0	7.6	7.2	6.8	6.4	6.0	5.6	5.2	4.8	4.4	4.0	3.8	3.6	3.4	3.2	3.0	2.8	2.6	2.4	2.2	2.0

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-8. CT Values for Inactivation of Giardia Cysts by Chlorine Dioxide, pH 6.0-9.0

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
0.5	10.0	8.6	7.2	5.7	4.3	4.2	4.2	4.1	4.1	4.0	3.8	3.7	3.5	3.4	3.2	3.1	2.9	2.8	2.6	2.5	2.4	2.3	2.2	2.1	2.0
1	21.0	17.9	14.9	11.8	8.7	8.5	8.3	8.1	7.9	7.7	7.4	7.1	6.9	6.6	6.3	6.0	5.8	5.5	5.3	5.0	4.7	4.5	4.2	4.0	3.7
1.5	32.0	27.3	22.5	17.8	13.0	12.8	12.6	12.4	12.2	12.0	11.6	11.2	10.8	10.4	10.0	9.5	9.0	8.5	8.0	7.5	7.1	6.7	6.3	5.9	5.5
2	42.0	35.8	29.5	23.3	17.0	16.6	16.2	15.8	15.4	15.0	14.6	14.2	13.8	13.4	13.0	12.4	11.8	11.2	10.6	10.0	9.5	8.9	8.4	7.8	7.3
2.5	52.0	44.5	37.0	29.5	22.0	21.4	20.8	20.2	19.6	19.0	18.4	17.8	17.2	16.6	16.0	15.4	14.8	14.2	13.6	13.0	12.2	11.4	10.6	9.8	9.0
3	63.0	53.8	44.5	35.3	26.0	25.4	24.8	24.2	23.6	23.0	22.2	21.4	20.6	19.8	19.0	18.2	17.4	16.6	15.8	15.0	14.2	13.4	12.6	11.8	11.0

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-9. CT Values for Inactivation of Viruses by Chlorine Dioxide, pH 6.0-9.0

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
2	8.4	7.7	7.0	6.3	5.6	5.3	5.0	4.8	4.5	4.2	3.9	3.6	3.4	3.1	2.8	2.7	2.5	2.4	2.2	2.1	2.0	1.8	1.7	1.5	1.4
3	25.6	23.5	21.4	19.2	17.1	16.2	15.4	14.5	13.7	12.8	12.0	11.1	10.3	9.4	8.6	8.2	7.7	7.3	6.8	6.4	6.0	5.6	5.1	4.7	4.3
4	50.1	45.9	41.8	37.6	33.4	31.7	30.1	28.4	26.8	25.1	23.4	21.7	20.1	18.4	16.7	15.9	15.0	14.2	13.3	12.5	11.7	10.9	10.0	9.2	8.4

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-10. CT Values for Inactivation of Giardia Cysts by Chloramine, pH 6.0-9.0

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
0.5	635	568	500	433	365	354	343	332	321	310	298	286	274	262	250	237	224	211	198	185	173	161	149	137	125
1	1,270	1,136	1,003	869	735	711	687	663	639	615	592	569	546	523	500	474	448	422	396	370	346	322	298	274	250
1.5	1,900	1,700	1,500	1,300	1,100	1,066	1,032	998	964	930	894	858	822	786	750	710	670	630	590	550	515	480	445	410	375
2	2,535	2,269	2,003	1,736	1,470	1,422	1,374	1,326	1,278	1,230	1,184	1,138	1,092	1,046	1,000	947	894	841	788	735	688	641	594	547	500
2.5	3,170	2,835	2,500	2,165	1,830	1,772	1,714	1,656	1,598	1,540	1,482	1,424	1,366	1,308	1,250	1,183	1,116	1,049	982	915	857	799	741	683	625
3	3,800	3,400	3,000	2,600	2,200	2,130	2,060	1,990	1,920	1,850	1,780	1,710	1,640	1,570	1,500	1,420	1,340	1,260	1,180	1,100	1,030	960	890	820	750

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-11. CT Values for Inactivation of Viruses by Chloramine

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
2	1,243	1,147	1,050	954	857	814	771	729	686	643	600	557	514	471	428	407	385	364	342	321	300	278	257	235	214
3	2,063	1,903	1,743	1,583	1,423	1,352	1,281	1,209	1,138	1,067	996	925	854	783	712	676	641	605	570	534	498	463	427	392	356
4	2,883	2,659	2,436	2,212	1,988	1,889	1,789	1,690	1,590	1,491	1,392	1,292	1,193	1,093	994	944	895	845	796	746	696	646	597	547	497

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-12. CT Values for Inactivation of Giardia Cysts by Ozone

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
0.5	0.48	0.44	0.40	0.36	0.32	0.30	0.28	0.27	0.25	0.23	0.22	0.20	0.19	0.17	0.16	0.15	0.14	0.14	0.13	0.12	0.11	0.10	0.10	0.09	0.08
1.0	0.97	0.89	0.80	0.72	0.63	0.60	0.57	0.54	0.51	0.48	0.45	0.42	0.38	0.35	0.32	0.30	0.29	0.27	0.26	0.24	0.22	0.21	0.19	0.18	0.16
1.5	1.50	1.36	1.23	1.09	0.95	0.90	0.86	0.81	0.77	0.72	0.67	0.62	0.58	0.53	0.48	0.46	0.43	0.41	0.38	0.36	0.34	0.31	0.29	0.26	0.24
2.0	1.90	1.75	1.60	1.45	1.30	1.23	1.16	1.09	1.02	0.95	0.89	0.82	0.76	0.69	0.63	0.60	0.57	0.54	0.51	0.48	0.45	0.42	0.38	0.35	0.32
2.5	2.40	2.20	2.00	1.80	1.60	1.52	1.44	1.36	1.28	1.20	1.12	1.04	0.95	0.87	0.79	0.75	0.71	0.68	0.64	0.60	0.56	0.52	0.48	0.44	0.40
3.0	2.90	2.65	2.40	2.15	1.90	1.81	1.71	1.62	1.52	1.43	1.33	1.24	1.14	1.05	0.95	0.90	0.86	0.81	0.77	0.72	0.67	0.62	0.58	0.53	0.48

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments.

Table C-13. CT Values for Inactivation of Viruses by Ozone

	Temperature (°C)																								
Inactivation (log)	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**
2	0.90	0.83	0.75	0.68	0.60	0.58	0.56	0.54	0.52	0.50	0.46	0.42	0.38	0.34	0.30	0.29	0.28	0.27	0.26	0.25	0.23	0.21	0.19	0.17	0.15
3	1.40	1.28	1.15	1.03	0.90	0.88	0.86	0.84	0.82	0.80	0.74	0.68	0.62	0.56	0.50	0.48	0.46	0.44	0.42	0.40	0.37	0.34	0.31	0.28	0.25
4	1.80	1.65	1.50	1.35	1.20	1.16	1.12	1.08	1.04	1.00	0.92	0.84	0.76	0.68	0.60	0.58	0.56	0.54	0.52	0.50	0.46	0.42	0.38	0.34	0.30

Source: AWWA, 1991. Modified by linear interpolation between 5°C increments

APPENDIX D

Oregon
Theodore R. Kulongoski, Governor

Department of Human Services
Drinking Water Program
2860 State Street
Medford, OR 97504
(541) 776-6229 ext. 284
Fax (541) 776-6013

August 29, 2006

Tim Brady
City of Roseburg
900 SE Douglas Ave.
Roseburg, OR 97470

RE: Sanitary Survey of Water System

Dear Tim:

Thank you for meeting with me on 8/224/06 as I conducted a **sanitary survey of the City of Roseburg's water system.** I appreciate the time and information you provided.

The purpose of the sanitary survey is to evaluate the water system as a whole and to identify any deficiencies that might interfere with the production and delivery of safe drinking water. A copy of the survey is enclosed for your records.

Overall, I found the water system to be well operated and maintained, and no significant deficiencies were identified. Please note that page 16 of the survey report lists upcoming due dates for various water quality tests.

Thank you again for your assistance, and please contact me if you have any questions.

Sincerely,

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

"Assisting People to Become Independent, Healthy and Safe"

Sanitary Survey Deficiency Summary

OHD Drinking Water Program Sanitary Survey

Page 1 of 16

System: Roseburg PWS ID: 41 00720

Operator/Contact: Tim Brady County: Douglas

Yes No		Date to be Corrected	Date Corrected
☐ ☒	Surface Source Deficiencies		
N A	Well Construction Deficiencies		
N A	Spring / Other Deficiencies		
☐ ☒	Disinfection Deficiencies		
☐ ☒	Treatment Deficiencies		
☐ ☒	Storage / Pressure Tank Deficiencies		
☐ ☒	Distribution Deficiencies		
☐ ☒	Monitoring Deficiencies		
☐ ☒	Management Deficiencies		

Comments: No deficiencies identified-

Surveyor: Scott Curry Date of Survey: 8/24/06

Inventory & Narrative

OHD Drinking Water Program Sanitary Survey

Page 2 of 16

System: Roseburg

PWS ID: 41 00720

Inventory

Date of Survey: 8/24/06

County: Douglas

Type

☒ Community (C)

☐ Non-Transient non-community (P)

☐ Transient non-community (N)

☐ State-regulated (S)

Size

Population: 28,000

Connections: 10,755

Service Chars: R 1

Ownership: 4

License

☐ Not Lic. ☐ HD ☐ Ag.

Season

☒ All year ☐ Seasonal

Begins: Ends:

Coliform Sampling

Period: ☒ Monthly ☐ Quarterly

Samples Required: 30

Responsible Agency

☒ State ☐ County ☐ Dept. of Ag.

Mailing Address:

Contact Name / Phone #: Tim Brady (541) 673-0397

Street Address: 900 SE Douglas Ave.

City, State, Zip: Roseburg, OR

Legal/Owner Address:

Contact Name / Phone #: same

Street Address:

City, State, Zip:

Supply Address:

Contact Name / Phone #: same

Street Address:

City, State, Zip:

Emergency Systems available:

Name: Umpqua Basin W.A. PWS ID#: 4100719

Name: Roberts Creek W.D. PWS ID#: 4100717

Name: PWS ID#:

Narrative: The City of Roseburg obtains its drinking water from the N. Umpqua River below the Winchester Dam, and treats it through a conventional filtration process. The water is distributed through a network that includes 7 pump stations and 10 reservoirs. The City is in compliance with current drinking water regulations, and staff are certified at the required levels. Maximum peak summertime demand is approximately 10 MGD.

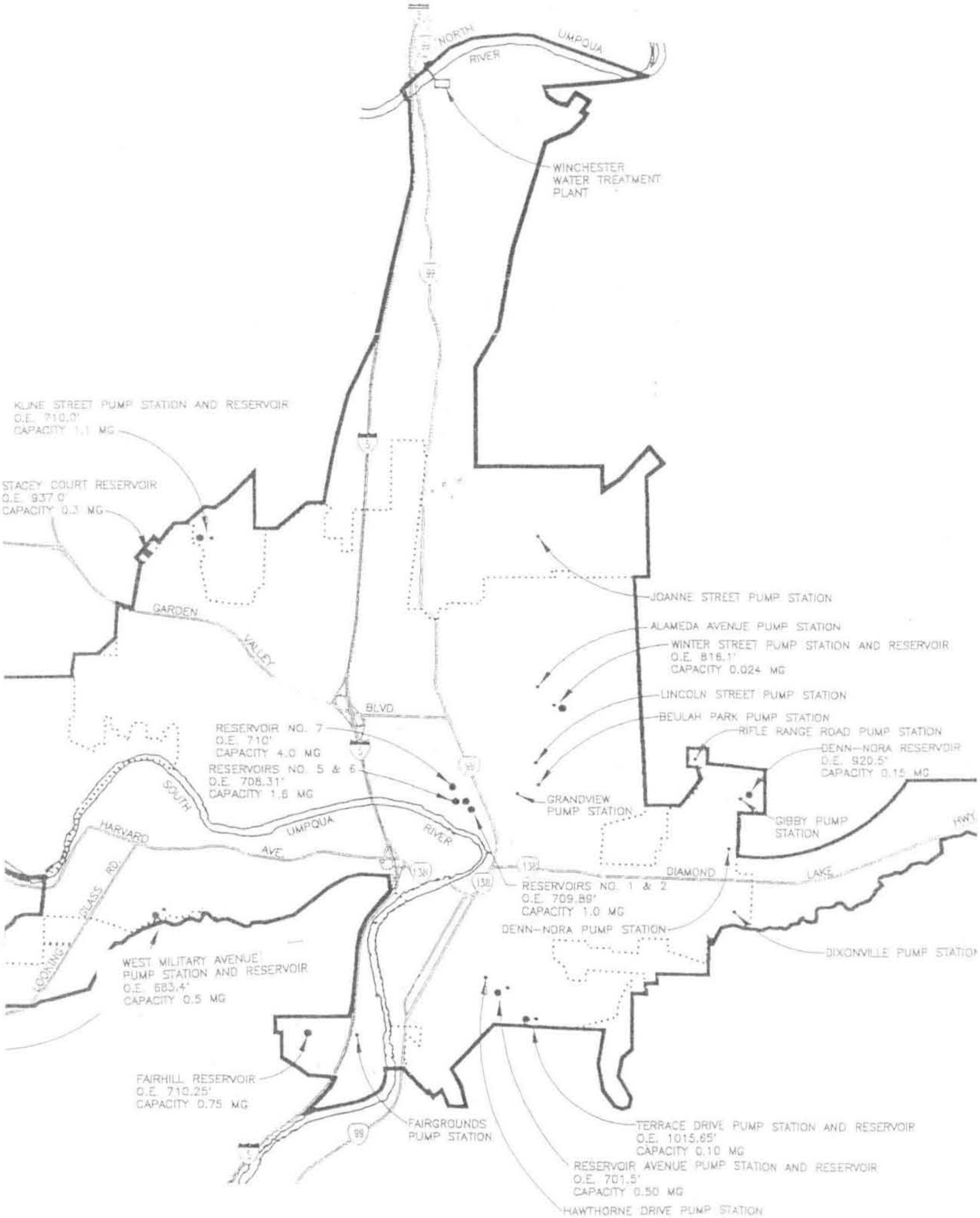
Schematic Drawing

OHD Drinking Water Program Sanitary Survey

Page 3 of 16

System: Roseburg

PWS ID: 41 0 0 7 2 0



Source Information

OHD Drinking Water Program Sanitary Survey

Page 4 of 16

System: Roseburg

PWS ID: 41 00720

Entry Points: (Location where water enters distribution and is sampled)

ID	Name	Source Type: Ground	Surface	GWUDI	Pur. Ground	Pur. Surface	Availability: Permanent	Seasonal	Begins	Ends	Emergency	None	Treatment Codes
A	N. Umpqua R.	☐	☒	☐	☐	☐	☒	☐	/	/	☐	☐	See below
		☐	☐	☐	☐	☐	☐	☐	/	/	☐	☐	
		☐	☐	☐	☐	☐	☐	☐	/	/	☐	☐	
		☐	☐	☐	☐	☐	☐	☐	/	/	☐	☐	
		☐	☐	☐	☐	☐	☐	☐	/	/	☐	☐	
		☐	☐	☐	☐	☐	☐	☐	/	/	☐	☐	

Individual Sources contributing at Entry Point:

ID		Name	Land Use*	Capacity (GPM)	Source Type: Ground	Surface	GWUDI	Pur. Ground	Pur. Surface	Availability: Permanent	Seasonal	Emergency	None	Treatment Codes
A	A	N. Umpqua R.	G	12 MGD	☐	☒	☐	☐	☐	☒	☐	☐	☐	P600
					☐	☐	☐	☐	☐	☐	☐	☐	☐	P240
					☐	☐	☐	☐	☐	☐	☐	☐	☐	P360
					☐	☐	☐	☐	☐	☐	☐	☐	☐	P660
					☐	☐	☐	☐	☐	☐	☐	☐	☐	P345
					☐	☐	☐	☐	☐	☐	☐	☐	☐	D800

*Land Use Codes: (A) Pristine Forest (B) Irrigated Crops (C) Non-Irrigated Crops (D) Pasture (E) Light Industry (F) Heavy Industry (G) Urban-Sewered Area (H) Rural On-Site Sewage Disposal (I) Urban On-Site Sewage Disposal (J) Rangeland (K) Managed Forest (L) Commercial (M) Recreational Use

Comments: (How and when sources are used, etc.) Single source used year-round.

Yes	No	
☒	☐	Does the water system have water rights for all sources? (☐ Not required)
☐	☐	USGS Location Map (name and number) attached?
☐	☒	Has a Source Water Assessment been completed by OHD or DEQ?

Yes	No	
☐	☐	Delineation attached/on file?
☐	☐	Hydrogeologic/sensitivity analysis on file?
☐	☐	• Highly sensitive aquifer or watershed? Explain:

☐☐ Have there been any changes since the original Source Water Assessment? Explain:

Comments on Source Water Assessment:

Conventional & Direct Water Treatment Plant Inspection

Basic Plant Information Page

DHS Drinking Water Program

System Name: Roseburg PWS ID# 4100720

☒ WTP inspection done with Sanitary Survey (Contact Code 1 and 1D)

WTP inspection only (Contact Code 1D)

Date of Inspection / Evaluation: 8-24-06

Plant Operator: Tim Brady Inspected By: Scott Curry

Total Points given (see 3rd page): 8

Points (to be determined)	Visit Frequency	Check One:
Low range	Every 3 years	☑
Mid range	Annually	☐
High range	Every 6 months	☐

Comments:

Source:

Describe Intake: Concrete tower w/ Trash racks + Travelling screen

Describe pumping facilities: 4 pumps (1@ 100 HP, 2@ 75 HP, 1@ 50 HP)

Watershed control information (protection plan, security measures, etc): Open watershed

Factors affecting water quality (algal blooms, logging...): algae in summer

General:

Processes: ☒Coagulation – chemical added PAC; ☒Flocculation; ☒Sedimentation Basin; ☒Tube Settlers; ☐Adsorption Clarifier; ☐Solids Contact Clarifier; ☐pH adjustment; ☐Corrosion Control; ☐Other

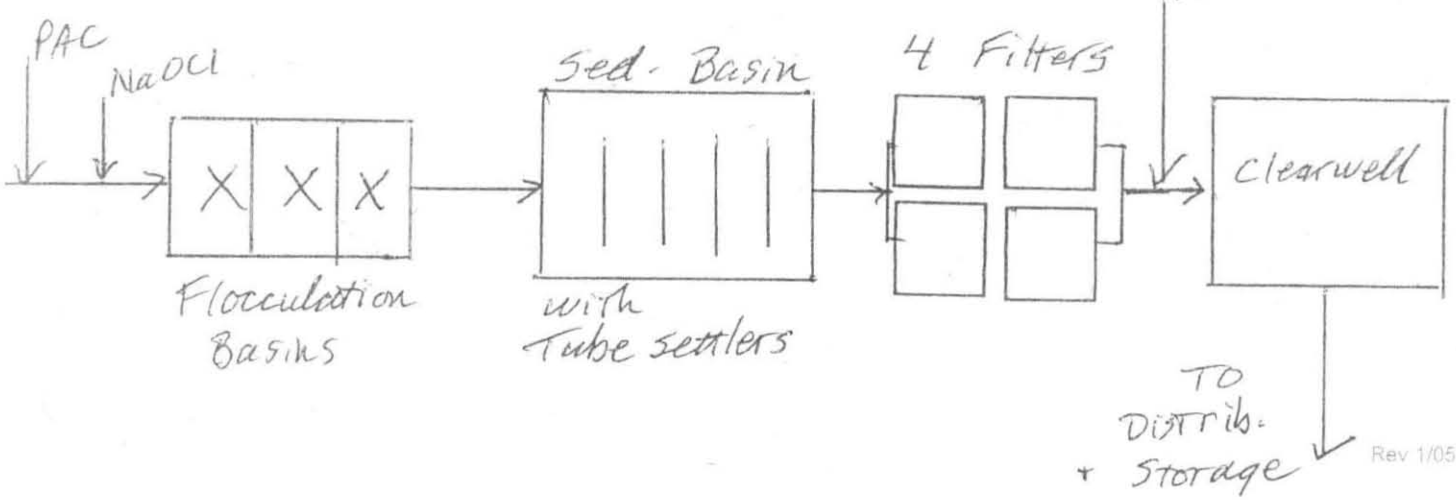
☒ ☐ •Plan review approved? Any outstanding issues:

Log removal credit given: 2.5 Giardia; 2.0 Crypto Date: 5/93

Based on: ☒CPE; ☐ Plan Review; ☐ WTP evaluation / rating form

---

Treatment Plant Schematic:



Name of System: Roseburg PWS ID# 4100720

Treatment Plant: (If no, Circle)

Yes/No #Points

☒ ☐ Is raw water turbidity data collected at least daily? ☒ On-line ☐ Bench-top 3
Average raw water 2.0 NTU Peak 300 NTU

☒ ☐ •For 2.5-log plants only: Is settled water turbidity measured at least daily? 5
☒ ☐ When raw water is < 10 NTU, settled <= 1 NTU? 2
☒ ☐ When raw water is > 10 NTU, settled <= 2 NTU? 2

☒ ☐ •Are turbidity compliance standards met? (<0.3 NTU 95% of time; all < 1 NTU) 10
☒ ☐ CFE Optimization goals met? (<0.1 NTU 95% of time; always < 0.3 NTU) 4
☐ ☒ •Is CFE monitoring location acceptable (prior to any storage)? [✓ 5]
☒ ☐ Are IFE turbidity always below triggers?
☐ ☐ Turbidity > 1.0 NTU in 2 consecutive 15-min readings
☐ ☐ > 10,000 only: Turbidity > 0.5 NTU in 2 cons. readings 1st 4 hrs after startup?
☐ ☐ Turbidity > 1.0 NTU in 2 consecutive 15-min readings for 3 months in a row
☐ ☐ Turbidity > 2.0 NTU in 2 consecutive 15-min readings for 2 months in a row
☐ ☒ Can chart recorder document turbidity > 1.5 NTU?

☒ ☐ Are chemical dosages adjusted with water quality changes (jar test or equivalent)? Process identified STreaming Current monitor 3

N A If using alum, is raw water alkalinity collected at least weekly? 3

☒ ☐ Does the operator know all chemical dosages applied in mg/L? 3
☒ ☐ Chemical strengths in %?
☐ ☒ Chemical feed rates in ml/min?
Could calculate ml/min.

☒ ☐ Are feed pumps calibrated at least annually? 3

How is backwash initiated?
☐ turbidity level ☐ headloss ☒ time

☒ ☐ Does the plant have filter to waste piping? 3
☐ ☒ If yes, is duration of filter-to-waste cycle based on turbidity profile results? [✓ 3]
Criteria for putting filter back on-line? based on time

Yes No

☒ ☐ • Are filter profiles conducted after backwash at least quarterly? 5

☒ ☐ Are optimization goals after backwash met? 4

☒ Max spike < 0.3 NTU ☒ ≤0.1 NTU after 15 minutes

NA • If recycling filter backwash water, is return location prior to chemical addition? 5

☒ ☐ • Are turbidimeters calibrated according to factory specifications or at least quarterly? 5

☒ ☐ Are calibration standards valid (not expired)?

☒ ☐ Is flow through turbidimeter within manufacturer's range?

☒ ☐ • Are CT's calculated correctly? 10

☒ ☐ • Is contact time based on tracer study or adequate alternative? estimated

☒ ☐ • pH, temperature, and chlorine residual measured at 1st user?

☒ ☐ • Is there a flow meter on effluent side of clearwell?

☐ ☒ Is corrosion control practiced?

NA • Is it operated within parameters set by DWS? 5

Method of corrosion control used ________________

☒ ☐ • Do all under-certified operators follow a written decision-making protocol as established by DRC? 5

☒ ☐ • Are standard plant operating procedures written and followed? 5

☒ ☐ Are operators on site during all hours of plant operation?

☐ ☐ • If no, is there an alarm for low chlorine and high turbidity? 5

☐ Low chlorine ☐ High turbidity ☐ Plant shutdown ☐ Auto-dial

Total Points = 8

☒ ☐ Is the water system / plant operators interested in participating in AWOP?

Comments:

Treatment

OHD Drinking Water Program Sanitary Survey

Page 8 of 16

System: Roseburg

PWS ID: 41 0 0 7 2 0

Process Used*	Chemical Added**	Purpose	Location in System	Code***
	PAC	Coagulation	pre rapid mix	
	Miox	pre-disinfection	"	
	Polymer	filter aid	pre-filter	
	Miox	disinfection	post-filter	

*See "Surface Info." page for details on filtration. **See "Disinfection" page for details on disinfection equipment. ***See Reverse.

Yes No

☒ ☐ Is equipment maintained properly?

☒ ☐ Is redundant equipment available?

What lab equipment is available and used? (jar testing, turbidimeter, pH meter, etc.)

Jar Tester, pH meter, Turbidimeter, hardness test Kit.
Streaming current monitor.

☒ ☐ • Are chemicals NSF-approved?

☒ ☐ Is operator aware of OSHA requirements for storage, handling, and spill containment?

Comments:

Yes No

☐ ☒ Does the system practice corrosion control?

N A Is it operated within parameters set by OHD?

Comments:

Records Kept:

Yes No

☒ ☐ • Dosages

☒ ☐ Raw pH

☒ ☐ Raw Temperature

☒ ☐ Raw Turbidity and/or particle counts

Yes No

☒ ☐ Flowrate

☒ ☐ Treated pH

☒ ☐ Treated Temperature

☒ ☐ Treated Turbidity

Comments:

Disinfection

OHD Drinking Water Program Sanitary Survey

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System: Roseburg PWS ID: 41 00720

#	Disinfection Method*	Location	Disinfection Source Water	Residual Maintenance	Proportional to flow	Dosage Recorded
1	Miox (Sodium Hypochlorite)	Pre-filtration	☒	☐	☒	☒
2	" "	Post-filtration	☐	☒	☒	☒
			☐	☐	☐	☐

*Chlorine Gas, Sodium Hypochlorite, On-Site Generated Sodium Hypochlorite, Calcium Hypochlorite, Chloramines, Ozone, UV, Mixed-Oxidants, Other

Yes	No	
☒	☐	• Is a DPD type test kit used?
☒	☐	• Is free chlorine residual maintained?
☒	☐	• Are residuals recorded at least daily? ☐ Daily ☒ Continuous ☐ Other
☒	☐	Is protective equipment available?

Type of protective equipment: goggles, gloves, face mask

N/A ☒ **Chlorine Gas:**

	#		#		#	
	Yes	No	Yes	No	Yes	No
• Separate room for gas storage and feeder?	☐	☐	☐	☐	☐	☐
• Fan with on/off switch outside?	☐	☐	☐	☐	☐	☐
• Vent located next to the floor?	☐	☐	☐	☐	☐	☐
• Door with a window?	☐	☐	☐	☐	☐	☐
• Gas cylinders properly secured?	☐	☐	☐	☐	☐	☐
• Door that opens out?	☐	☐	☐	☐	☐	☐
• Self-contained breathing apparatus?	☐	☐	☐	☐	☐	☐
An air scrubber system?	☐	☐	☐	☐	☐	☐

N/A ☒ **Disinfection with UV light**

Yes	No		Yes	No	
☐	☐	• Plan Review Approval (P.R. # ____)	☐	☐	• Is lamp replaced annually
☐	☐	• Does all water contact UV (no bypass)	☐	☐	• Alarm or shut off
☐	☐	• Is lamp sleeve cleaned			

Source Water Classification: ☐ Ground ☒ Filtered Surface ☐ Unfiltered Surface ☐ GWUDI
(check all that apply)

CT Evaluation

Disinfection requirement: 0.5 log removal for ☒ giardia ☐ viruses OR ____ minutes

Maximum demand flow: 8350 gpm, Minimum contact time: 61 minutes

Contact time determined by tracer study or estimated? estimated

Range of chlorine residuals at first user: 0.5 - 0.9

Yes	No	
☒	☐	• Are CT values met at all times?
☐	☒	Has disinfection benchmark been established? (☒ Not required)

Comments:

Distribution System Information

OHD Drinking Water Program Sanitary Survey

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System: Roseburg PWS ID: 41 00720

Service Area and Facility Map

Yes No

☒ ☐ • Does the system have a Service Area and Facility Map with the following features:

multiple maps

☒ Booster Pumps
☒ Pressure Reducing Valves
☒ Pressure Zones
☒ Sampling Points
☒ Sources-wells & withdrawal points
☒ Storage Facilities (reservoirs)
☒ Treatment Facilities
☒ Water Lines (including size and material)

Distribution Data

Yes	No		Comments
☒	☐	• System pressure >20 psi?	
☒	☐	System metered? (what %?)	100 %
☒	☐	Water system leakage <10%?	
☒	☐	Waterline depth >30"?	
☒	☐	Piping looped?	
☒	☐	• Hydrants or adequate blowoffs on all dead ends?	
☒	☐	• Routine flushing? (how often?)	
☒	☐	Adequate valving?	
☒	☐	Routine valve turning? (how often?)	

Comments:

Cross Connection Program (Community systems only)

Yes	No		Comments
☒	☐	• Ordinance or enabling authority?	
☒	☐	• Testing records current?	
☒	☐	Approved devices installed?	
☒	☐	• Annual summary report sent?	
		If more than 300 Connections:	
☒	☐	• Certified inspector?	
☒	☐	• Complete written program plan?	

Comments:

Booster Pumps

Number	Name (location)	Deficiencies Noted or Comments	HP	GPM	Aux. Power
					Y☐ N☐
	See next page				Y☐ N☐
					Y☐ N☐
					Y☐ N☐
					Y☐ N☐
					Y☐ N☐

Comments:

WTP Address
- 186 [illegible] Way
Winchester

City of Roseburg

4100720

BOOSTER STATION DATA SHEET

DEC. '83
REH

GENERAL			DISTRIBUTION						PUMP STATION										STORAGE			DESIGN	
NO.	NAME	FUNCTION	NO. EXISTING SERVICES	ELEVATION OF EXIST. SERVICES		LINES			PUMP Q (GPM)			GROUND ELEVATION	CYCLE PRESSURE		SUCTION PRES.		DISCHARGE PRES. AUG '83		TYPE		SIZE	SERVICE ELEV. @ 30 PSI	NO. SERVICES
				LOW	HIGH	SMALLEST	LARGEST	TOTAL LENGTH	#1	#2	#3		ON	OFF	STATIC	DYNAMIC	STATIC	DYNAMIC	HYDRO TANK	RES.	O.F. GRD.		
1	NEWTON CREEK BOOSTER STATION	DOMESTIC & LIMITED FIRE SERVICE TO RESIDENTIAL AREA	49	614'	704'	4"	8"	4060'	75 @ 110'	228 @ 80'	-	641.29'	CONTINUOUS		30	20	-	30-70	NONE			716'	60
2	CLOVERDALE BOOSTER STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	160	590'	756'	2"	8"	7800'	150 @ 165'	150 @ 165'	-	600.33'	FLOAT SWITCH ON RES.		42	39	92	101		✓	24,000 816.10' 783.04	715'	160±
3	WINTER ST. HYDRO-PNEUMATIC STATION	DOMESTIC SERVICE TO RESIDENCES	2	776'	786'	2"	2"	150'	40 @ 80'	-	-	770'	40	60					✓		120	793'	5
4	HOLLIS STREET BOOSTER STATION	DOMESTIC SERVICE TO RESIDENCES	10	594'	674'	1"	2"	1150'	15 @ 60'	-	-	560'	ON TIMER 6 AM-MN						NONE			680'	< 10
~~5~~	~~[illegible] ST. HYDRO-PNEUMATIC STATION~~	~~DOMESTIC SERVICE TO RESIDENCES~~	~~23~~	~~626~~	~~795'~~	~~4"~~	~~6"~~	~~1030'~~	~~35 @ 90'~~	~~60 @ 110'~~	~~-~~	~~610~~	~~90 80~~	~~110~~	~~98~~	~~30~~	~~TANK~~		~~✓~~		~~325~~	~~745'~~	~~23~~
6	BEULAH ST. HYDRO-PNEUMATIC STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	33	700'	804'	6"	8"	2300'	100 @ 212'	1000 @ 150'	-	570'	130 120	150 140	64	50	TANK		✓		2000	800'	39
7	Ventura BOOSTER STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	6	630'	814'	2"	8"	2700'	150 @ 260'	150 @ 260'	-	621.62'	FLOAT SWITCH ON RES.				130			✓	150,000 920.5' 897.0'	830'	180
8	HAWTHORNE ST. BOOSTER STATION	SUMMER BOOST TO RESERVOIRS #3 & #4	205	PUMPS TO RES. EL. 692.0-710.5		8"	8"	600'	850 @ 50'	-	-	614'	ON TIMER							✓	500,000 701.5' 692.0'	630'	N/A
9	HIGH LEVEL BOOSTER STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	203	600'	920'	1"	6"	19,730'	100 @ 335'	275 @ 350'	-	698.73'	FLOAT SWITCH ON RES.		1	1	138	146		✓	100,000 1015.65 992.01	920'	
10	TERRACE HYDRO-PNEUMATIC STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	23	942'	1060'	6"	8"	4780'	100 @ 190'	300 @ 190'	[illegible] @ 180' STBY	992.01'	70 60	90 80	8	5	TANK		✓		900	1080'	35
11	MILLER'S ADDITION BOOSTER STATION	MAINTAINS PRES. IN SUMMER TO RESIDENCES	8	605'	644'	2"	6"	330'	38 @ 60'	-	-	560'	ON TIMER						NONE			630'	<8
12	FAIRGROUNDS BOOSTER STATION	COMMERCIAL & FIRE SERVICE TO FAIRGROUNDS	14 FIRE BYPASS	460'	666'	8"	12"	5200'	750 @ 150'	-	-	496.01'	FLOAT SWITCH ON RES.		96	60	100	104		✓	750,000 710.25 670.75	630'	N/A
13	KLINE ST. BOOSTER STATION	DOMESTIC & FIRE SERVICE TO RESIDENTIAL AREA	2	630'	710'	6"	8"	1050'	62 @ 242'	62 @ 242'	750 @ 177'	678.5'	90 85 75	110 105 100	8	6	TANK		✓		1000	830'	30
14	WEST SIDE BOOSTER STATION	DOMESTIC TO RESIDENTIAL AREA				4"	8"	3590'	45 @ 95'	90 @ 95'	-	592'	-	-	-	-	-	-	NONE			690'	30
15	TRANSMISSION BOOSTER STATION	NO LONGER USED	-	-	-	-	-	-	6900 @ 180'	6500 @ 142'	-	540'	-	-	-	-	-	-	MAIN LEVEL			630	N/A
16	RIFLE RANGE ROAD	DOMESTIC SERVICE TO RESIDENCES	3	640'	700'	8"	8"	650'	33 @ 100'	33 @ 100'	-	600'	90 80	110 100	42				✓		330	700	3

REV 9/25/87

Roseburg

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Storage & Pressure Tanks

OHD Drinking Water Program Sanitary Survey

Page 12 of 16

System: Roseburg PWS ID: 41 00720

Number	Name	Tank Type*	Tank Material	Year Built	Volume (gal.)
1	Reservoir #1		Concrete	1900±	500,000
2	Reservoir #2		"	1900±	500,000
3	Reservoir #5		Steel	1949	800,000
4	Reservoir #6		"	1949	800,000
5	Reservoir #7		"	1980	4,000,000

*(G) Ground (E) Elevated (P) Pressure Total Volume= ______

Reservoir Features

Reservoir Number	1		2		3		4		5	
	Yes	No	Yes	No	Yes	No	Yes	No	Yes	No
Hatch: • Locked	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Watertight	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Shoebox type lid (curbing)	☐	☒	☐	☒	☐	☒	☐	☒	☒	☐
Features:										
• Drain to Daylight	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Overflow	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Flap Valve (on drain and/or overflow)	☒	☐	☒	☐	unk.		unk.		☐	☒
• Screened Vent	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Water Level Gauge	☐	☒	☐	☒	☒	☐	☒	☐	☒	☐
Bypass Piping	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Fence/Gate	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Cathodic Plates Watertight	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Alarm for high/low levels	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Maintenance:										
Exterior in Good Condition	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Approved Interior Coating	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Annual Inspection	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Cleaning Schedule	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Continuously disinfected	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Plumbing Configuration:										
Separate Inlet/Outlet	☐	☒	☐	☒	☒	☐	☒	☐	☐	☒
Baffling	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒
Used for Contact Time	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒

NA

Hydropneumatic Tank Number	☐		☐	
Used for Contact Time	☐	☐	☐	☐
• Accessible for Maintenance	☐	☐	☐	☐
Separate Inlet/Outlet	☐	☐	☐	☐
Bypass Piping	☐	☐	☐	☐
Access Port	☐	☐	☐	☐
• Drain	☐	☐	☐	☐
Pressure Relief Device w/Gauge	☐	☐	☐	☐
Air Blow Off Valve	☐	☐	☐	☐
Air Bladder/Diaphram	☐	☐	☐	☐
Valve for Adding Air	☐	☐	☐	☐
Water Level Sight Glass	☐	☐	☐	☐

Comments:

Reservoirs #1 + #2

out of service 2005.

Storage & Pressure Tanks

OHD Drinking Water Program Sanitary Survey

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System: Roseburg

PWS ID: 41 0 0 7 2 0

Number	Name	Tank Type*	Tank Material	Year Built	Volume (gal.)
6	Frontier Lane	G	Steel	1980	150,000
7	Fairhill Dr. (Fairgrounds)	G	Steel	1968	750,000
8	Kline St.	G	Steel	1975	1,000,000
9	Rocky Ridge	G	Steel	2002	1,000,000
10	Stacey Court	G	Steel	1991	300,000

*(G) Ground (E) Elevated (P) Pressure

Total Volume=

Reservoir Features

Reservoir Number	6		7		8		9		10	
	Yes	No	Yes	No	Yes	No	Yes	No	Yes	No
Hatch: • Locked	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Watertight	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Shoebox type lid (curbing)	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Features:										
• Drain to Daylight	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Overflow	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Flap Valve (on drain and/or overflow)	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒
• Screened Vent	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Water Level Gauge	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Bypass Piping	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Fence/Gate	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Cathodic Plates Watertight	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Alarm for high/low levels	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Maintenance:										
Exterior in Good Condition	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Approved Interior Coating	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Annual Inspection	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Cleaning Schedule	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
• Continuously disinfected	☒	☐	☒	☐	☒	☐	☒	☐	☒	☐
Plumbing Configuration:										
Separate Inlet/Outlet	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒
Baffling	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒
Used for Contact Time	☐	☒	☐	☒	☐	☒	☐	☒	☐	☒

NA

Hydropneumatic Tank Number				
Used for Contact Time	☐	☐	☐	☐
• Accessible for Maintenance	☐	☐	☐	☐
Separate Inlet/Outlet	☐	☐	☐	☐
Bypass Piping	☐	☐	☐	☐
Access Port	☐	☐	☐	☐
• Drain	☐	☐	☐	☐
Pressure Relief Device w/Gauge	☐	☐	☐	☐
Air Blow Off Valve	☐	☐	☐	☐
Air Bladder/Diaphram	☐	☐	☐	☐
Valve for Adding Air	☐	☐	☐	☐
Water Level Sight Glass	☐	☐	☐	☐

Comments:

Storage & Pressure Tanks

OHD Drinking Water Program Sanitary Survey

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System: Roseburg

PWS ID: 41 00720

Number	Name	Tank Type*	Tank Material	Year Built	Volume (gal.)
11	Terrace Drive	6	Steel	2000	800,000
12	W. Military Ave.	6	Steel	1956	500,000
~~13~~	~~Winter Street~~	~~6~~	~~Steel~~	~~1948~~	~~24,000~~

*(G) Ground (E) Elevated (P) Pressure

Total Volume= 12.8 MG

Reservoir Features

Reservoir Number	11		12		13					
	Yes	No	Yes	No	Yes	No	Yes	No	Yes	No
Hatch: • Locked	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Watertight	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Shoebox type lid (curbing)	☐	☒	☒	☐	☒	☐	☐	☐	☐	☐
Features:										
• Drain to Daylight	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Overflow	☒	☐	☐	☐	☒	☐	☐	☐	☐	☐
• Flap Valve (on drain and/or overflow)	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐
• Screened Vent	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Water Level Gauge	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Bypass Piping	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Fence/Gate	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Cathodic Plates Watertight	☐	☐	☐	☐	☐	☐	☐	☐	☐	☐
Alarm for high/low levels	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Maintenance:										
Exterior in Good Condition	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Approved Interior Coating	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Annual Inspection	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Cleaning Schedule	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
• Continuously disinfected	☒	☐	☒	☐	☒	☐	☐	☐	☐	☐
Plumbing Configuration:										
Separate Inlet/Outlet	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐
Baffling	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐
Used for Contact Time	☐	☒	☐	☒	☐	☒	☐	☐	☐	☐

Hydropneumatic Tank Number				
Used for Contact Time	☐	☐	☐	☐
• Accessible for Maintenance	☐	☐	☐	☐
Separate Inlet/Outlet	☐	☐	☐	☐
Bypass Piping	☐	☐	☐	☐
Access Port	☐	☐	☐	☐
• Drain	☐	☐	☐	☐
Pressure Relief Device w/Gauge	☐	☐	☐	☐
Air Blow Off Valve	☐	☐	☐	☐
Air Bladder/Diaphram	☐	☐	☐	☐
Valve for Adding Air	☐	☐	☐	☐
Water Level Sight Glass	☐	☐	☐	☐

NA

Comments:

① Winter St. Reservoir out of service 2006.

② Drain/Overflow lines that do not terminate in manholes must have flap valves.

Management & Operations

OHD Drinking Water Program Sanitary Survey

System: Roseburg PWS ID: 41 00720

Management Operations

Identify management structure of water system staff: Pub. Works Director → Civil Engineer → Chief Operator

Yes	No	
☒	☐	Do water system revenues pay for operation, maintenance and staffing?
☒	☐	Do water system revenues go into a reserve fund for capital improvement projects?
☒	☐	• Does system have an operation and maintenance manual?
☒	☐	• Does system have an emergency response plan?

Operator/Cross Connection Certification

Requirements for system: WT 3 WD 3

Name	Certification Number	Water Treatment Level	Water Distribution Level	Filtration Endorsement	Cross Connection Inspection Number	Backflow Assembly Tester #
DRC:* Tim Brady (Treatment)	2057	3		☒		
Bill O'Byrne (Distribution)	1397		3	☐		
				☐		
				☐		
				☐		

DRC = direct responsible charge. Attach additional sheets if necessary to list all certified personnel.

Yes	No	
☒	☐	• Is there an operator at required certification level?
☒	☐	• Are CEUs being maintained?

Plan Review/Master Plan

☒	☐	• Have all major modifications (since 8/21/81) been approved by OHD?
☒	☐	Does system have a current plan review exemption for water main extensions?
☒	☐	• Does the system have a current (<20 yr. old) master plan? (☐ Not required if <300 connections)

1992 What year was the plan completed?

Yes	No	
☐	☒	Does the master plan include a water conservation plan?

Compliance Status

☒	☐	• Is water system in compliance (all Administrative Orders and Notices of Violation resolved)?
1		How many violations has the system had in the past two years?
☒	☐	Does the system issue Public Notice for Violations as required?

Other

☐	☒	Has a capacity assessment been completed by OHD?

If yes, list deficiencies noted:

☒	☐	Are consumer confidence reports sent to users each year?

Comments:

Water Quality Monitoring

OHD Drinking Water Program Sanitary Survey

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System: Roseburg PWS ID: 41 00720

Contaminant	n/a	Frequency	Next Tests Due
Coliform Bacteria		Monthly	
Nitrate		Annually	2007
Arsenic		Annually	2007
Inorganic Chemicals (sw)	☐	Every 9 years	2015
Inorganic Chemicals (gw)	☒		
SOC's	☐	Every 3 years	2007
VOC's (sw)	☐	Annually	2006
VOC's (gw)	☒		
Radiologicals	☐	Every 9 years	2016
Asbestos	☐	Every 9 years	
TTHM's and HAA5's	☐	Quarterly	
TOC	☐	Quarterly	
Lead and Copper, # sites: (30)	☐	Every 3 years	2008
Turbidity	☐	Continuously	
Source Water Coliform	☒		
Other (specify)	☒		

For CT calculations (see CPE).

Yes No

☒ ☐ • Is all required monitoring current?

Comments:

☐ ☒ Has the system experienced chemical (last 5 years) or bacteriological (last 2 years) detections?

If yes, what contaminant and when?

N A • Have all MCL violations been addressed?

☐ ☐ Does the system have any monitoring reductions granted? Explain: IOC's every 9 years. Lead/Copper every 3 yrs.

☒ ☐ • Does the system have a written coliform sampling plan?

Does the plan include:

Yes No

☒ ☐ Brief narrative?

☒ ☐ Distribution map?

☒ ☐ Sample site locations?

☒ ☐ Rotation schedule?

☒ ☐ Repeat locations?

Where in the system are the monitoring sites for TTHM and HAA5: (☐Not required)

☒ ☐ Are TTHM and HAA5 samples taken at location of maximum residence time?

Comments:

APPENDIX E

Treatment Plant Expansion

Funding Analysis and Implementation Plan

City of Roseburg
May 2009 Update

Prepared By: Integrated Consulting Services, Inc.

Treatment Plant Expansion

Funding Analysis and Implementation Plan

Prepared For:

City of Roseburg
May 2009 Update

Prepared by:

Integrated Consulting Services, Inc.
400 SE Jackson Street
Roseburg, Oregon 97470

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SECTION 6: WATER RATE ANALYSIS

SECTION 1

INTRODUCTION

SECTION 1

INTRODUCTION

1.1 BACKGROUND

The water treatment facilities for Roseburg have a maximum capacity of 12 million gallons per day (mgd). Peak water production was measured at 10.6 mgd. Maximum water demands are approaching the treatment capacity of the existing facilities. The first phase of staged plant construction was finished in 1987. Components of the facility are nearing the expected design life of 20 years.

The City has initiated the development of a detailed plan for upgrading and expanding the capacity of the water treatment facilities. Major public works projects often take 2 to 3 years to complete, in order to provide time for public education, securing funding, engineering design, regulatory approvals, and construction.

The City retained Murray, Smith, and Associates, Inc. (MSA) in June 2006 to prepare two reports, titled “City of Roseburg Water Treatment Facilities Preliminary Design Report” and “City of Roseburg Long-Range Water Supply Plan”. A draft report was submitted to Roseburg for the treatment facilities preliminary design was submitted in late 2007. Recommendations included expanding the capacity of the treatment plant to 18 mgd at an estimated cost of $7.8 million. The City subsequently requested that MSA update the reports to include revised population projections. The final reports are were prepared by MSA in May 2009, with a revised estimated total of $8.1 million.

Integrated Consulting Services, Inc. prepared a draft “Treatment Plant Expansion, Funding Analysis and Implementation Report” in December 2007, based on findings in the draft MSA report. Integrated Consulting Services, Inc. updated the funding analysis and implementation plan so that information can be incorporated as an addendum of the “City of Roseburg Water Treatment Facilities Preliminary Design Report” completed by MSA.

The implementation plan has been revised to reflect the updated milestones for completing the expansion of the water treatment facilities. The funding analysis has not been changed from the December 2007 draft report, and is based on a loan amount of $8 million to finance the improvements to the treatment facilities.

Opinions of probable cost for the construction project are currently based on preliminary planning reports. The estimates of construction costs for expanding the water treatment facilities will continue to evolve throughout the design process as more detailed information becomes available. The final water rate adjustment required to fund bonds or loans is dependent on a combination of several factors, including interest terms available at the time of construction and actual construction bids received.

The options available for funding the project are not likely to change significantly as cost estimates become more refined. The City felt at this time it is most important for the funding analysis and implementation plan to focus on: a) options for funding, b) preliminary estimates of water rate adjustment, and c) plan and tentative schedule for implementing the project. The City will update the rate projections as the project moves closer to construction.

1.2 AUTHORIZATION

On November 1, 2007, the City of Roseburg authorized Integrated Consulting Services, Inc. (ICS) to develop an implementation plan for expanding the treatment facilities and to evaluate funding alternatives. The City of Roseburg authorized ICS to revise the plan on January 26, 2008.

1.3 SCOPE OF STUDY

The scope of this study, prepared by ICS, includes two major tasks:

1. Develop an implementation plan for expanding the treatment facilities, identifying major milestones and tentative schedule.

2. Evaluate funding alternatives and prepare a preliminary rate analysis, including financial projections for the next 5 years.

SECTION 2

SUMMARY AND IMPLEMENTATION PLAN

SECTION 2

SUMMARY AND IMPLEMENTATION PLAN

2.1 TEN-YEAR CAPITAL IMPROVEMENT PROGRAM

The funding evaluation for the treatment facilities upgrade must also address financing of the overall capital improvement program (CIP) for the water system. The 10-year CIP for the water system is summarized in Table 2-1 and was developed by City staff. A more detailed description of plan is included in Section 3.

Table 2-1. Ten-Year Water CIP

Program Description	Estimated Cost
Treatment	$8,100,000
Transmission	$4,350,000
Pump Stations	$600,000
Reservoirs	$2,350,000
Distribution	$5,000,000
Land	$500,000
Miscellaneous	$2,450,000
Contingency	$2,000,000
Total	$25,350,000
Total minus Treatment	$17,250,000

The average annual expenditure for capital improvements during the next 10 years, not including the treatment plant upgrade, is projected at $1.7 million (2007 dollars). Historical expenditures during the previous 10 years averaged $1.49 million per year (total of $14.9 million in expenditures). Considering inflation, the projection of $1.7 million per year for future expenditures is similar to historical levels of capital spending.

2.2 FINANCE OPTIONS

An overview and preliminary assessment of finance options is included in Section 4. The proposed upgrade of the water treatment facilities does not initially appear to qualify for most of the state and federal programs that provide grants and low interest loans for public works projects. Funding programs tied to economic development may be the best potential source of grants. It is recommended that Roseburg schedule a “One Stop” meeting through the regional coordinator of the Oregon Economic and Community Development Department (OECDD) to discuss funding eligibility in more detail.

The City Finance Director will work closely with Bond Counsel, City staff, and the City Council when determining the type of bonds to issue for project financing. For purposes of the preliminary rate analysis for funding the treatment plant upgrade, it has been assumed that bonds will be issued in the amount of $8 million at an annual interest rate of 5 percent and for a period of 20 years, and that the bonds will be repaid solely with revenue from the Water Enterprise Fund.

2.3 SYSTEM DEVELOPMENT CHARGES

Two categories of system development charges (SDCs) can be assessed: 1) *improvement fee*, assessed for costs associated with improvements to be constructed in the future, and 2) *reimbursement fee*, a "buy-in" fee to reimburse costs associated with capital improvements already constructed or under construction. The City of Roseburg charges a reimbursement SDC for water.

All public works projects must be designed with reserve capacity to accommodate future growth, otherwise the facilities would be at capacity the day they are constructed. The intent of the reimbursement SDCs is to recover a portion of the costs associated with excess capacity over the design life of the facility, which typically is a 20-year period.

The City's methodology for water SDCs includes the subtotals listed in Table 2.2 for the reimbursement SDC computation (updated for 2007 costs). Based on the methodology, Roseburg could charge a maximum of $3,072 per equivalent residential unit (ERU). The City currently charges $1,800 per ERU.

Table 2.2. Breakdown of Reimbursement SDC Computation, December 2007 ENR Construction Cost Index of 8,089.

System Component	Reimbursement SDC Fee Per ERU
Treatment	$1,085
Transmission	$542
Distribution	$134
Storage	$1,311
Combined Total	$3,072

The reimbursement component for treatment is $1,085, as computed in the SDC methodology for the existing treatment plant. The treatment component is based on a treatment capacity of 12 mgd. The plant is near capacity, and this component of the SDC charge will drop off after the expansion of the treatment facilities is complete.

Section 5 of this report includes a preliminary computation of the water SDC component for the expanded treatment facilities. The upgraded facilities will have a capacity of 18 mgd. The SDC for the *expanded* facilities has been computed at $1,098 in Section 5. Therefore, no significant increase to the water SDC (methodology) is anticipated due to the plant expansion. New development, within the Roseburg water service area, will pay for their share of the upgraded facilities as they connect to the system.

The City may wish to consider adjustments to the water SDC as part of overall funding for the 10-year CIP. Roseburg currently charges a water SDC of $1,800 per ERU while the existing methodology allows a charge of up to $3,070.

Very few communities in Oregon assess the maximum SDC that the methodology allows. The amount of SDC assessed is generally limited by market forces (potential impacts on development and affordable housing). A survey of SDCs charged in 14 communities, conducted July 2006, is included in Section 5.

Roseburg recently implemented a series of staged increases to the water SDC, increasing the water SDC from $710 in 2004 to $1,800 in July 2006. The water SDC was set at $710 in 1996, and no adjustments were made during the 8-year period between 1996 and 2004.

Water SDCs for Roseburg are budgeted to generate $225,000 in revenue during FY 2007/08. Annual adjustments (as allowed by Oregon Revised Statutes) are recommended to keep pace with inflation. Development has slowed in Roseburg, and the community will need to weigh the estimated increase in revenue versus potential impacts to new development when considering annual increases greater than the consumer price index (CPI). The development community should also recognize that the treatment plant expansion is required to support new development.

Preliminary funding projections to finance the water treatment facilities upgrade assume annual adjustments tied to the CPI only.

2.4 WATER RATE ANALYSIS

Historical expenditures and future projections for the next 5-years are evaluated in Chapter 6. Findings are summarized in the following paragraphs.

2.4.1 Rate History

The City of Roseburg adopted a surcharge in 1985 to generate up to $700,000 per year in revenue for phased treatment plant construction. The actual surcharge assessed was less than authorized, generating a maximum of $400,000 annually. The surcharge was dropped in 1992 after the plant construction was complete.

The City completed a Water Master Plan in 1993. Recommendations in the Master Plan included average annual expenditures of $2.1 million to fund the 10-year Capital Improvement Program (CIP). Combined rate increases of 21 percent were implemented in 1994 and 1995 to fund the recommended CIP.

No further rate adjustments occurred for nine years. An increase of 10 percent was adopted in 2005 to fund projected needs for short-term capital improvements, and subsequent annual increases of 2.5 percent were implemented to offset cost increases due to inflation. The rate history from 1993 through 2008 is summarized in Table 2.3.

Table 2.3. Rate History, 1993 to 2008

Year	Resolution	Effective	Base Rate	Commodity Rate (100 cf)	Percent Increase
FY 1993/94			$5.85	$0.93	----
FY 1994/95	94-12	Jul-94	$6.69	$1.07	14.4%
FY 1995/96	95-11	Jul-95	$7.16	$1.14	7.0%
FY 1996/97			$7.16	$1.14	0.0%
FY 1997/98			$7.16	$1.14	0.0%
FY 1998/99			$7.16	$1.14	0.0%
FY 1999/00			$7.16	$1.14	0.0%
FY 2000/01			$7.16	$1.14	0.0%
FY 2001/02			$7.16	$1.14	0.0%
FY 2002/03			$7.16	$1.14	0.0%
FY 2003/04			$7.16	$1.14	0.0%
FY 2004/05	2004-15	Jan-05	$7.88	$1.25	10.1%
FY 2005/06	2005-26	Jan-06	$8.08	$1.28	2.5%
FY 2006/07		Jan-07	$8.28	$1.31	2.5%
FY 2007/08		Jan-08	$8.49	$1.35	2.5%

2.4.2 Ten-Year History for Water Enterprise Fund

During the last 10 years, the increase in water sales due to growth (independent of water rate adjustments) increased at an average of approximately 2 percent per year, while operation and maintenance (O&M) expenses increased at an average of approximately 6 percent per year. O&M expenses reflect not only annual increases in personnel costs, but also increases in utilities, chemicals, materials, fuel, contracted labor, etc. Periodic rate increases are necessary to keep pace with inflation.

Currently, the Water Enterprise Fund generates $4.5 million in annual revenue from user fees, and is budgeted to receive $225,000 in revenue from SDCs. There is $1.4 million currently available on an annual basis, after O&M expenses, to fund capital improvements, increased cash reserves, and contingencies.

2.4.3 Five-Year Projections for Water Enterprise Fund

Past Projections

Projections presented to the City Council in 2004, estimated $1.66 million would be available for capital improvements in FY 2007/08, after outstanding bonds were paid off. The updated estimate is $1.4 million. The difference is within 5 percent of the total budget for the Water Enterprise Fund. Based on past projections, future projections can made with sufficient accuracy for planning and budgeting purposes. Projections should be updated on an annual basis to reflect actual costs and new information.

Annual CPI Increases Only

The five-year forecast indicates that annual CPI increases plus growth (increase in sales due to new development) will be adequate to fund projected O&M expenses plus an average annual expenditure of $1.5 million (2008 dollars) for capital improvements. This is an important finding from a budgeting standpoint since this demonstrates that without the plant expansion, annual CPI (inflation) adjustments appear adequate to fund projected operating costs plus an annual expenditure of $1.5 million for capital projects.

Surcharge to Finance Treatment Facilities Upgrade

Circumstances are different today than when the treatment plant was originally constructed in phases between 1987 and 1992. When the existing facilities were constructed, components of the old 1935 plant could be used for treatment, providing the flexibility to build the replacement plant in stages. Today, the proposed plant upgrade is an addition to the existing facility, and the project does not lend itself as well to staging. The upgrade will likely need to be constructed as a single project.

To pay cash for the upgrade, instead of issuing bonds or securing loans, will require a sinking fund to generate adequate cash reserves prior to commencing with construction. A surcharge of $2 million per year (approximately 45 percent of the current revenue) for 4 years would generate a cash reserve of $8 million. The surcharge would be the equivalent of a $10 per month increase for an average residential customer.

Interest rates were relatively high in the 1980's when the design of the plant was initiated, and significant savings were realized by paying cash and building the project in phases. Today interest rates are relatively low, and potential savings in interest by paying with cash may be offset by increased costs over time from inflation.

If the City elects to pursue this option, then further investigation should be conducted on how to structure the surcharge – should the surcharge be added to base rate, commodity rate, or a combination of both.

Financing the Treatment Facilities Upgrade Through Bond Sales

The annual payment is $642,000 for debt service on $8 million of principal, with terms of 5 percent annual interest and 20-year pay back period.

Forecasts indicate that a water rate increase of 15 percent will be sufficient to repay the bonds (or loans). CPI adjustments will be required in addition to the increase for bond repayment. Phasing the increase in over two years is recommended. Assuming annual CPI adjustments of 3 percent, and two, staged 7.5 percent increases for the treatment plant expansion, the combined rate increases would be approximately 10 percent a year for two years.

Water rates are listed in Table 2.4 for two 10 percent annual increases. The average water bill in Roseburg is currently estimated at $22 per month. The combined increases over the two years would result in an average monthly increase of approximately $4.70 per month for residential customers. CPI adjustments would continue thereafter (to maintain a projected annual expenditure of $1.5 million for other capital improvements).

Table 2.4. Phased Water Rate Increases

			Cubic Feet		
Description	Inside/ Outside	Monthly Base Rate	Minimum Number with Base	Usage Charge Per 100 Cu Ft	Cost per 7,500 Gallons or 1,000 cu ft
Current	Inside	$8.49	0	$1.35	$21.99
(Jan. 2008)	Outside	$18.49	0	$1.35	$31.99
Phase I	Inside	$9.34	0	$1.49	$24.24
10% Increase	Outside	$19.34	0	$1.49	$34.24
Phase II	Inside	$10.27	0	$1.64	$26.67
10% Increase	Outside	$20.27	0	$1.64	$36.67

Note: Dixonville outside customers pay an additional $10 monthly surcharge + pumping charge

2.4.4 Comparison of Roseburg Water Rates with Other Douglas County Communities

Ten other water purveyors in Douglas County were surveyed. Results of the survey are summarized in Table 6.6.

Comparison of rates with other communities is not a basis for setting rates, but does provide an indicator of the relative level of rates being charged. Compared to the communities surveyed, Roseburg's rates are approximately 50 percent (1/2) of the average charged (for 1,000 cubic feet or 7,500 gallons), both inside and outside the City limits.

2.5 IMPLEMENTATION PLAN

Table 2.5 includes an outline of the proposed implementation plan.

The City has sufficient cash reserves to pay for engineering design. For budgeting purposes, design services are expected to be in the range of approximately 10% to 12% of construction cost or approximately $900,000 (not including bid phase services, construction administration, or construction observation). The tentative schedule shown includes starting design in the fall of 2009, beginning construction in the spring of 2011, with construction of the treatment facilities upgrade finished by December 2012.

Table 2.5 Proposed Implementation Plan

Milestone	Tentative Completion Date
FY 2008-09 Budget Adopted; Including Budget for Engineering Design in FY 2009-10 as Water Enterprise Fund Capital Improvement	May 2009
MSA Completes Final Reports	Jun 2009
Award Consultant Contract	Oct 2009
One-Stop Meeting Scheduled Through OECDD Regional Coordinator	Nov 2009
10 Percent Engineering Design	Jan 2010
Public Education Program	Jan-Feb 2010
Secure Approval for Long-Term Financing	Feb 2010
Implement Rate Adjustments for Funding	Mar 2010
Draft FY 2010-11 Budget Submitted; Include Budget for Expansion of Treatment Facilities as Water Enterprise Fund Capital Improvement	Apr 2010
FY 2010-11 Budget Adopted	Jun 2010
90 Percent Engineering Design	Jul 2010
Design Complete	Sep 2010
Advertise for Construction Contracts	Nov 2010
Award Construction Bids	Jan 2011
Begin Construction	Mar 2011
Construction Complete, Facilities Operational	Dec 2012

Note: Dates are tentative and timelines may be extended as required

SECTION 3

TEN-YEAR CAPITAL IMPROVEMENT PROGRAM

SECTION 3

TEN-YEAR CAPITAL IMPROVEMENT PROGRAM

3.1 CURRENT TEN-YEAR CIP

Table 3.1 includes a summary of the current 10-year CIP for the City of Roseburg water system. The CIP was developed by City staff based on the 1993 Water Master Plan and current conditions, and was presented to the City of Roseburg Public Works Commission.

The 10-year CIP is divided into two five-year sections; projects that are expected to occur within the next 5 years and projects anticipated within the next 6 to 10 years. The funding evaluation for the water treatment plant expansion must also address funding of the overall CIP for the Roseburg water system.

Table 3.1. Water System Capital Improvement Program (CIP)

1 to 5-Year CIP, FY 2008/09 to FY 2012/13

PROJECTS	**Opinion of Probable Cost**	**Fiscal Year**
Treatment Plant Expansion (12 mgd to 18 mgd)	$7,800,000	2008 to 2010
Starmer Reservoir	$1,000,000	2008, 2009
Dixonville Pump Stations	$600,000	2008, 2009
Main Replacement Program (Annual Program)	$2,500,000	2008 to 2012
Mapping (GIS, Orthophotos, Surveys, Inventories)	$350,000	2008 to 2012
Water System Plan Update	$250,000	2009, 2010
Reservoir #7 Refurbishing	$500,000	2009
Demolition Reservoirs #1 and #2 (Reservoir Hill)	$350,000	2010
Main Reservoir Complex – Piping	$900,000	2010
Vehicles & Equipment	$250,000	2008 to 2012
Miscellaneous Projects	$500,000	2008 to 2012
Contingency/Other	$1,000,000	2008 to 2012
Total	$16,000,000	
Average Annual Expenditure, 1 to 5 Years	**$3,200,000**	
Average Annual Expenditure - Minus Plant Upgrade	**$1,640,000**	

Table 3.1. Continued, Water System CIP

6 to 10-Year CIP, FY 2013/14 to FY 2017/18

PROJECTS	**Opinion of Probable Cost**	**Fiscal Year**
Newton Creek/North Roseburg Water Improvements	$1,250,000	2013 to 2015
Diamond Lake Blvd Transmission Main Extension (UGB)	$1,500,000	2013, 2014
Sunshine Park Reservoir	$1,000,000	2015, 2016
Main Replacement Program (Annual Program)	$2,500,000	2013 to 2018
Transmission Main Replacement, Dee to Emerald	$700,000	2013
Mapping (GIS, Orthophotos, Surveys, Inventories)	$350,000	2013 to 2018
Reservoir Refurbishment	$500,000	2013 to 2018
Land Acquisition	$500,000	2013 to 2018
Vehicles & Equipment	$250,000	2013 to 2018
Miscellaneous Projects	$500,000	2013 to 2018
Contingency/Other (Annual Expense)	$1,000,000	2013 to 2018
Total	$10,050,000	
Average Annual Expenditure, 6 to 10-Years	**$2,010,000**	

Note: CIP is based on 1993 Water Master Plan with costs updated for inflation, input from City staff, and "City of Roseburg Water Treatment Facilities Preliminary Design Report". An update of the Water Master Plan is included in the 5-Year CIP.

3.2 TEN- YEAR HISTORICAL CAPITAL EXPENDITURES

Historical expenditures on capital improvements during the last 10 years for the water system are summarized in Table 3.2. Expenditures averaged approximately $1.5 million each year during the 10-year period (no adjustment for inflation). A range of $1.5 million to $2 million for future capital expenditures (not including treatment plant expansion) is similar to historical expenditures.

Table 3.2. Summary of Water Capital Expenditures During Last 10 Years

Year	**Capital Expenditure**
FY 1998-99	$1,726,942
FY 1999-00	$1,466,889
FY 2000-01	$1,890,048
FY 2001-02	$1,452,493
FY 2002-03	$1,397,914
FY 2003-04	$1,170,691
FY 2004-05	$620,548
FY 2005-06	$1,918,171
FY 2006-07	$1,048,949
FY 2007-08 Adopted	$2,213,000
10-Year Average	**$1,490,565**

SECTION 4

FINANCE OPTIONS

SECTION 4

FINANCE OPTIONS

4.1 INTRODUCTION

A preliminary overview and assessment of the options is provided in this section. More detailed analysis of potential impacts on user fees follow in subsequent sections. Options for funding include:

- Federal or State Grants and/or Loans
- Sale of Bonds
- System Development Charges
- Capital Improvement (Sinking) Funds
- User Fees
- Special Assessments
- Local Improvement Districts
- Serial Levies

4.2 PUBLIC WORKS FINANCING PROGRAMS

Many of the Federal and State programs that provide grant and loan financing for public works projects were developed to provide assistance to a) rural residents, b) residents with low incomes, and c) projects that address a health and/or safety issue. Roseburg may not qualify for funding through most of these programs because the City's resident population and average income levels may exceed the eligibility limits. In addition, Roseburg currently has no compliance issues related to water treatment.

The water treatment plant upgrade will provide capacity for economic development. Funding programs tied to economic development may be the best potential source of grants and low interest loans. Projects that demonstrate a firm commitment for job creation or retention have the highest probability of receiving funding. Maximum grant funding typically ranges from $500,000 to $1,000,000.

The Oregon Economic and Community Development Department (OECDD) sponsors "One Stop" meetings to discuss projects and potential financing with a variety of funding agencies. It is recommended that Roseburg schedule a "One Stop" meeting through the OECDD regional coordinator for this area. The meeting will help determine Roseburg's eligibility for grant and low interest loan funding for upgrading the water treatment facilities.

Four grant programs and five loan programs, which provide financing for water improvement projects are listed below. All of the programs listed will be discussed at the "One Stop" meeting coordinated through OECDD.

Grants

Federal	● Economic Development Administration ● Rural Development
Federal Administered by State	● Community Development Block Grants
State	● Special Public Works Fund ● Water/Wastewater Financing Program

Loans

Federal	● Rural Development
State	● Special Public Works Fund ● Water/Wastewater Financing Program ● State Revolving Loan Program

Each of the programs varies in the extent and complexity of the application process. A brief overview and assessment of the programs follows.

4.2.1 Economic Development Administration

The emphasis of the Economic Development Administration is on infrastructure improvements needed for business retention and expansion. Results from a survey of businesses must demonstrate that creation of jobs will occur, in sufficient numbers, by virtue of building the improvements. There is a higher chance of receiving grant funding if the community demonstrates that the existing system is at capacity. Funding through this program has been limited in recent years due to budget constraints at the federal level.

4.2.2 Rural Development

The Water and Wastewater Disposal Grants and Loans program is under the administration of the U.S. Department of Agriculture, Rural Development. Interest rates and maximum grant percentages are based on a community's median household income. The program is generally limited to rural communities with populations of less than 10,000 people. The certified population estimate in Roseburg was 21,069 people on June 30, 2007. Thus, it does not appear that Roseburg meets the population eligibility criteria.

4.2.3 Community Development Block Grant Program

The Oregon Economic and Community Development Department (OECDD) administers the Community Development Block Grant (OCDBG) program. This program is funded by the U.S. Department of Housing and Urban Development. Funds allocated under this grant program are provided for projects designed to specifically improve the conditions of low and moderate income housing areas. The maximum grant for a project is $1,000,000, which includes planning, engineering, and construction.

To qualify for a grant, the project must meet at least one of three national objectives. In general, the primary objective for public water systems is that the project principally benefits low and moderate income residents. For projects that include capacity for future development, grant participation is limited to the portion of the project cost that is necessary to serve the current population.

At least 51% of Roseburg's population must have low and moderate incomes to be eligible for funding. Roseburg's low to moderate income percentage is 42.8% based on the 2000 Census data. It does not appear that the water treatment facilities upgrade will qualify for OCDGB funding.

4.2.4 Special Public Works Fund

OECDD administers the Oregon Special Public Works Fund (SPWF) program. The SPWF program provides funding for municipally-owned facilities that support economic and community development in Oregon.

While primarily a loan program, grants may be available for projects that create or retain jobs. Grants are limited to $500,000 or 85% of the project cost, which ever is less. Projects must build public infrastructure to assist in a business expansion, thus creating jobs, or build needed infrastructure for future economic growth in the community. Interest rates for loans typically range from 5% to 6.5%.

4.2.5 Water/Wastewater Financing Program

A loan and grant program for the design and construction of public infrastructure to ensure compliance with the Safe Drinking Water Act or the Clean Water Act. The state funded program is administered by OECDD. Loans up to $15,000,000 and grants up to $500,000 are the maximum available for eligible projects.

To be eligible, a system must have received, or is likely to soon receive, Notice of Non-Compliance by the appropriate regulatory agency associated with the Safe Drinking Water Act or the Clean Water Act. Roseburg's water treatment facilities are currently in compliance with state and federal water quality statutes and standards and, therefore, the proposed project does not appear to qualify for funding at this time through the Water/Wastewater Financing program.

4.2.6 Safe Drinking Water Revolving Loan Program

The Safe Drinking Water Revolving Loan Program is a loan program that provides low-cost financing for construction and/or improvements of water systems. Funding is capitalized by annual grants from the U.S. Environmental Protection Agency (EPA) and matched with state resources. The program is jointly managed by the Department of Human Resources (Drinking Water Program) and OECDD.

To be eligible for funding, a project must resolve an existing or potential health hazard or non-compliance under state/federal standards related to the public provision and conveyance of water for human consumption. Currently, there are no potential health issues related to water treatment in Roseburg. Therefore, the proposed upgrade of the water treatment facilities does not appear to qualify for this program.

4.3 LOCAL FUNDING SOURCES

A significant portion of the project will need to be financed with local sources. Local funding sources include:

- General Obligation Bonds
- Revenue Bonds
- Improvement Bonds (LID)
- Serial Levies
- Assessments
- Ad Valorem Tax
- Sinking Funds
- System User Fees
- System Development Charges

A description for each of the preceding listed funding sources follows. It will be important to work closely with the City Finance Director and Bond Counsel when proceeding with financing of improvements.

4.3.1 General Obligation Bonds

Financing of water improvements by general obligation (G.O.) bonds is accomplished by the following procedures:

1. Consulting engineer provides a detailed cost estimate to determine total moneys required to complete project. The total cost includes engineering, administration, contingencies, interim financing, bond sale costs, and other project related costs in addition to estimated construction costs.

2. An election is held.

3. If voter approval is granted (by majority of voters), bonds are offered for sale.

G.O. bonds are backed by the full faith and credit of the issuer and authorize the issuer to levy ad valorem (property) taxes. The issuer can make the required payments on the bonds solely from the new tax levy or may instead use revenue from assessment, user charges, or some other source. Oregon Revised Statutes limit the maximum term of G.O. bonds to 40 years for cities and 25 years for special districts. The realistic term for G.O. bonds is typically 20 years, but should not exceed the life of improvement.

Ballot Measure 5 limited the ability of communities to levy property taxes. Capital improvement projects, such as the proposed water system improvements, are exempt from the property tax limitations if an election is held and public hearing requirements are met.

4.3.2 Revenue Bonds

A revenue bond is one that is payable solely from charges made for the services provided. Such bonds can not be repaid from tax levies or special assessments, and their only security is the borrower's promise to operate the water system in a way that will provide sufficient net revenue to meet the obligations of the bond issue. Revenue bonds are most commonly retired with revenue from user fees rather than property taxes, and interest rates may be slightly higher than G.O. bonds.

Under provisions of the Oregon Uniform Revenue Bond Act, municipalities may elect to issue revenue bonds for revenue producing facilities without a vote of the electorate. In this case, certain notice and posting requirements must be met, including a mandatory 60-day waiting period. A petition signed by 5 percent of the municipality's registered voters may cause the issue to be referred to an election.

4.3.3 Improvement Bonds (Local Improvement District)

Improvement bonds may be issued to assess certain portions of water improvements directly against the parties being benefited. An equitable means of distributing assessed cost must be utilized so that all property, whether developed or undeveloped, receives the assessment on an equal basis. Cities are limited to improvement bonds not exceeding 3 percent of true cash value.

Improvement bond financing requires that an improvement district be formed, the boundaries be established, and the benefited properties and property owners are determined. The property to be assessed must have a true cash valuation of at least 50 percent of the total assessments levied. The financing is impacted by Measure 5 tax limitations because the improvement bonds are backed or guaranteed by the city's authority to raise revenue via taxation. The City administers a revolving loan fund for local improvement district's, with a fund balance of approximately $1,000,000, that is available without selling bonds. Local improvement district funding should not be considered for improvements to satisfy City needs in general, such as the water treatment plant expansion, but is a consideration for future expansions to annexation areas.

4.3.4 Serial Levies

Under Oregon Revised Statutes, if approved by voters, a city can levy taxes for a fixed period of time to construct new facilities and to maintain existing facilities. Generally, when a serial levy is presented to voters, it is based upon a specific program and listing of planned improvements. Due to the magnitude of the treatment plant expansion, it is doubtful that a serial levy of sufficient size to provide needed construction revenues would be supported by voters.

4.3.5 Ad Valorem Tax

Communities sometimes utilize an ad valorem tax as the basis for repaying G.O. bonds for water projects and supplement with additional water use charges. This is a means of financing that reaches all properties ultimately benefited by the water system, whether the property is developed or not. Also, property taxes are generally tax deductible for homeowners and businesses. The Tri City Water District and City of Jacksonville are examples of water system improvement projects funded through a combination of ad valorem tax and user fees. Although there are some benefits to customers for repayment of G.O. bonds with an ad valorem tax, it is difficult in recent years to develop community support for payment of water system improvements with property taxes in lieu of user fees.

4.3.6 Sinking Funds

Sinking funds can be established by budget for a particular capital improvement need. Budgeted amounts from each annual budget are carried in a sinking fund until sufficient revenue is available for a needed project. Funds can also be developed from revenue derived from system development charges or serial levies.

Although a separate sinking fund was not established, a similar funding mechanism was utilized in 1985 to finance the construction of the Roseburg water treatment facilities. The City Council approved a surcharge of up to $700,000 per year to fund the construction in conjunction with cash reserves and revenue from existing fees. The City had the flexibility to build the new plant, in phases, as major components of the old plant were replaced. The surcharge was dropped seven years later, in 1992, after the plant construction was complete. The total project cost was $8 million.

The option of utilizing a sinking fund concept will be evaluated further in Section 6 as an alternative for financing the treatment plant upgrade. However, circumstances are significantly different today then when the plant was originally constructed. The proposed plant upgrade does not lend itself to multiple phases and will likely be constructed as a single project. Also, interest rates were relatively high in the mid-1980's when design of the plant was initiated. Thus, significant savings were realized by paying for the project in cash. Today, interest rates are relatively low and potential savings in interest could be offset by increased costs from inflation.

4.3.7 System User Fees

Monthly charges are made to all residences, businesses, etc. that are connected to the water system. Water usage charges are established by resolution, and can be modified as needed to serve increased or decreased operating costs. The City could repay the local share of bond amortization with user fees.

4.3.8 Assessments

In some cases, the beneficiary of a public works improvement can simply be assessed for the cost of the project. Industrial or commercial developers might provide up-front capital to pay for a community administered improvements which serve the development. The proposed upgrade of the water treatment facilities will provide an area-wide benefit, and currently there are no specific developments that could be targeted for a special assessment.

4.3.9 System Development Charges

System development charges (SDC's) are charges assessed against new development to recover costs incurred by local government to provide the capital facilities required to serve new development. Preliminary computations to update the water SDC's for the proposed treatment plant expansion will be evaluated further in Chapter 5.

4.4 PRELIMINARY SCREENING OF FINANCE OPTIONS

It is recommended that Roseburg schedule a "One Stop" meeting through the regional coordinator of OECDD to discuss potential grant and low interest loan funding in more detail. Representatives from most of the public works funding agencies are typically present at the "One Stop" meetings. City representatives generally would include at least the Finance Director, Public Works Director, and a representative from MSA (engineering consultant).

Although it is prudent for the City to secure the maximum amount of grant funding available for the project, based on a preliminary review it does not initially appear that Roseburg will qualify for significant grant funding. Therefore, rate evaluations in subsequent sections initially assume no grant monies will be received for the project.

The Finance Director will work closely with Bond Counsel, City staff, City Manager and the City Council to determine the most feasible option for issuing bonds. It has been assumed for rate analysis that bonds will be repaid solely with revenue from the Water Enterprise Fund.

SECTION 5

SYSTEM DEVELOPMENT CHARGES

SECTION 5

SYSTEM DEVELOPMENT CHARGES

5.1 INTRODUCTION

The adopted FY 2007-08 budget for the Water Enterprise Fund includes an annual revenue projection of $225,000 from water SDCs.

It is important to evaluate whether new development will pay their fair share of the cost for expanding the water treatment plant. The methodology used previously to compute the treatment portion of the water SDC will be reviewed in this Section 5 to determine whether the treatment plant expansion will justify an increase in SDCs. The City should also consider whether increases in water SDCs will be implemented as part of the overall financing to fund the 10-year CIP.

5.2 REVIEW OF METHODOLGY TO COMPUTE WATER SDCS

The methodology implementing water and storm water SDCs was established by Resolution No. 91-17, and subsequently revised by Resolution No.'s 2004-27, 2005-20, and 2006-09.

The water SDC currently being charged is $1,800 per equivalent residential unit (ERU). A water ERU is defined by meter flow capacity, with the capacity of a 5/8"x3/4" meter (standard residential meter) equal to one ERU.

There are two categories of SDCs:

1. Improvement fee. Assessed for the costs associated with capital improvements to be constructed in the future.

2. Reimbursement fee. Buy-in fee to reimburse costs associated with capital improvements already constructed or under construction, essentially to pay back system users for money to support future customers.

The City currently only charges a ”reimbursement” SDC for water. The City has more flexibility for expenditures with revenue collected from reimbursement fees than improvement fees. Revenue collected from improvement fees can only be expended on projects identified in the capital improvement plan included with the SDC methodology. Revenue collected from reimbursement SDCs can be utilized for any water enterprise expenditure (operations, capital, debt retirement, etc).

A breakdown of the reimbursement SDC computation (updated for current costs based on a December 2007 ENR Construction Cost index of 8,089) is listed in Table 5.1.

Table 5.1. Breakdown of Reimbursement SDC Computation, December 2007 ENR Construction Cost Index of 8,089.

System Component	Reimbursement SDC Fee
Treatment	$1,085
Transmission	$542
Distribution	$134
Storage	$1,311
Combined Total	$3,072

Treatment Component – SDC Methodology, Existing Plant

The methodology for computing the treatment component of the reimbursement SDC for the existing treatment plant is as follows.

Total Cost (1991 dollars) = $8,356,383

Design Capacity (Water Treatment) = 12 mgd

Design Capacity (Service Population) = 14,215 ERUs

Number of ERUs in 1991 = 10,780 ERUs

Reserve Capacity in Plant (1991) = (14,215 ERUs – 10,780 ERUs) = 3,345 ERUs

Cost of Reserve Capacity = $8,356,383 x (3,345 ERUs/14,215 ERUs) = $2,195,645

Reimbursement SDC (1991 dollars) = $2,195,645/3,345 ERUs = $639 per ERU

ENR Construction Index, 1991 = 4,766

ENR Construction Index, Dec. 2007 = 8,089

Reimbursement SDC (2007 dollars) = $639 x (8,089/4,766) = $1,085 per ERU

Treatment Component – SDC Methodology, Proposed Facilities Upgrade

An identical analysis can be made for the proposed plant upgrade. For purposes of preliminary computations it is assumed that 100 percent of the cost is for reserve capacity. This assumption will result in computing a preliminary SDC that is too high, but the computation still provides useful information for comparison.

In reality, existing equipment and electrical controls that have reached their useful design life are being replaced. As the design proceeds, the consulting engineer should provide a detailed breakdown between costs for improvements that benefit all customers and those that only provide reserve capacity, so that the final reimbursement SDC for the treatment plant upgrade can be computed. Final computations should result in a lower SDC, when the costs that benefit all customers are subtracted out.

Total Cost (2007 dollars) = $7,800,800

Design Capacity (Water Treatment) = 18 mgd

Design Capacity (Service Population) = 21,320 ERUs

Design Capacity Prior to Upgrade (Service Population) = 14,215 ERUs

Reserve Capacity in Plant = (21,320 ERUs – 14,215 ERUs) = 7,105 ERUs

Cost of Reserve Capacity = $7,800,000 x (7,105 ERUs/7,105 ERUs) = $7,800,000

Reimbursement SDC (2007 dollars) = $7,800,000/7,105 ERUs = $1,098 per ERU

ENR Construction Index, Dec. 2007 = 8,089

The existing plant has a design capacity of 14,215 ERUs. Based on 2006 meter counts, there are currently 13,950 ERUs being served. There are approximately 265 ERUs of capacity remaining in the existing facility, based on these estimates (2 to 3 years of expansion at current growth rates).

It is important to recognize that the existing reimbursement SDC being assessed for treatment will paid off after the new plant is constructed. Preliminary computations indicate that the reimbursement SDC for the plant upgrade will be nearly identical to (or less than) the existing reimbursement SDC. Therefore, no increase to the water SDC methodology is anticipated due to the plant expansion. New developments will pay for their share of treatment plant capacity as they connect to the system.

5.3 COMPARISON OF SDCS WITH OTHER CITIES

Significant changes to the SDC methodology are not anticipated because of the treatment plant expansion (reimbursement SDC for expanded plant will be nearly identical to reimbursement SDC for existing plant). Currently, the City charges a water SDC of $1,800 per ERU, while the existing methodology justifies a charge of up to $3,070.

Very few communities in Oregon assess the maximum SDC that the methodology allows. The amount of SDC assessed is generally limited by market forces rather than the amount permitted by Oregon Revised Statutes. The cost of the SDC is passed through to either the developer or purchaser of the development, and could potentially impact the affordability of housing and marketability of the property.

A survey of SDCs charged by 14 Oregon communities was made in 2006 and results are summarized in Table 5.2. The communities were selected to provide a cross-section of cities with a variety of housing markets, and primarily include communities along the I-5 corridor. The SDC for residential development in Roseburg is similar to that charged in Eugene and Springfield, but less than other communities that are experiencing significant growth or have high demand properties. Sutherlin and Winston are in market competition with Roseburg for development because of their close proximity. At a minimum, annual increases tied to an inflation index are recommended to keep pace with inflation. The development market has slowed in Roseburg, and the community will need to weigh the estimated increase in revenue versus potential impacts to new development for increases greater than consumer price index.

Table 5.2. SDC Survey of 14 Oregon Communities for Residential ERU, July 2006

	City	Water	Storm	Parks	Transportation	Sanitary	Total
1	West Linn	$5,946	$455	$8,029	$4,897	$2,632	$21,959
2	Bandon	$6,546	$3,080	$0	$1,742	$2,382	$13,750
3	Bend	$3,385	$0	$3,340	$4,217	$1,973	$12,915
4	Salem	$4,002	$429	$2,963	$1,815	$2,682	$11,891
5	Ashland	$3,362	$507	$1,041	$2,044	$2,482	$9,436
6	Grants Pass	$2,321	$398	$1,245	$2,584	$2,455	$9,003
7	Medford	$948	$520	$2,544	$3,042	$1,692	$8,746
8	Myrtle Creek	$6,257	$0	$0	$0	$2,412	$8,669
9	Redmond	$2,092	$0	$834	$2,877	$2,105	$7,908
10	Springfield	$1,968	$1,189	$1,000	$1,036	$1,933	$7,125
11	Eugene	$2,110	$479	$1,345	$1,534	$1,546	$7,013
12	**Roseburg**	**$1,800**	**$825**	**$515**	**$1,816**	**$2,007**	**$6,963**
13	Winston	$2,305	$0	$150	$589	$1,913	$4,957
14	Sutherlin	$780	$0	$1,700	$1,098	$747	$4,325

SECTION 6

WATER RATE ANALYSIS

SECTION 6

WATER RATE ANALYSIS

6.1 INTRODUCTION

Historical expenditures in the Water Enterprise Fund and future projections for the next 5-years to fund the proposed capital improvement program will be evaluated in this section.

6.2 PHASED CONSTRUCTION OF EXISTING WATER TREATMENT FACILITIES

The phased construction of the existing water treatment facilities is an important consideration when evaluating historical expenditures and past rate increases in the Water Enterprise Fund. Table 6.1 includes a summary of the major phases of plant construction.

Table 6.1. Phasing of Treatment Facilities Construction

Phase	Description	Completion
I	New River Intake	1987
II	Treated Water Pumping Station	1988
III	Rapid Mix, Flocculation and Sedimentation Basins	1989
IV	Operations Building and Rapid Sand Filters	1992
V	Conversion to Onsite Generation of Liquid Chlorine and Intake Pump Upgrades	2002
VI	New Concrete Backwash Ponds and Solids Handling Facilities	2006

The treatment facilities were constructed to replace a treatment plant built in 1935. The ability to continue operating the old plant while constructing the new plant provided the flexibility of phasing construction. Interest rates were relatively high in the 1980's and the City chose to stage the construction, paying for each phase on a cash basis. The first four phases had a combined cost of $8 million.

New federal safety regulations relative to containment of chlorine gas led the City to replace the gaseous chlorination system with a system that generates liquid chlorine from a salt (NaCl) brine. The intake pumps were upgraded and the chlorination system was replaced in 2002 for $0.8 million. The City continued to use the earthen backwash pond left from the original 1935 plant, until new concrete basins and solids handling facilities were constructed in 2006 at a cost of approximately $1.5 million.

6.3 RATE HISTORY

The City of Roseburg adopted a surcharge in 1985 to generate up to $700,000 per year for treatment plant construction. The maximum surcharge implemented was $400,000 per year, less than the maximum authorized. The phased construction was financed with revenue from the surcharge, cash reserves, and existing user fee revenue.

The surcharge was dropped in 1992 after the plant construction was complete. Table 6.2 is a summary of rates charged from 1993 through January 2008. The City completed a Water Master Plan in 1993. Recommendations in the Master Plan included a 10-year CIP with an average annual expenditure of $2.1 million (1993 dollars) for transmission, storage, and distribution improvements. Combined rate increases of 21 percent were implemented in 1994 and 1995 to fund the CIP recommended in the Master Plan. There were no further rate increases until 2005. In 2005, a rate increase of 10 percent was adopted to fund projected needs for short-term capital improvements, and subsequent increases of 2.5 percent were implemented to offset cost increases due to inflation.

Table 6.2. Rate History, 1993 to 2008

Year	Resolution	Effective	Base Rate	Commodity Rate (100 cf)	Percent Increase
FY 1993/94			$5.85	$0.93	----
FY 1994/95	94-12	Jul-94	$6.69	$1.07	14.4%
FY 1995/96	95-11	Jul-95	$7.16	$1.14	7.0%
FY 1996/97			$7.16	$1.14	0.0%
FY 1997/98			$7.16	$1.14	0.0%
FY 1998/99			$7.16	$1.14	0.0%
FY 1999/00			$7.16	$1.14	0.0%
FY 2000/01			$7.16	$1.14	0.0%
FY 2001/02			$7.16	$1.14	0.0%
FY 2002/03			$7.16	$1.14	0.0%
FY 2003/04			$7.16	$1.14	0.0%
FY 2004/05	2004-15	Jan-05	$7.88	$1.25	10.1%
FY 2005/06	2005-26	Jan-06	$8.08	$1.28	2.5%
FY 2006/07		Jan-07	$8.28	$1.31	2.5%
FY 2007/08		Jan-08	$8.49	$1.35	2.5%

6.4 TEN-YEAR HISTORY, WATER ENTERPRISE FUND

Table 6.3 provides a 10-year historical summary of revenue and expenditures in the Water Enterprise Fund.

Table 6.3. Ten-Year History, Water Enterprise Fund, FY 1998-99 Through FY 2007-08

REVENUE

Description	Actual FY 1998-99	Actual FY 1999-00	Actual FY2000-01	Actual FY 2001-02	Actual FY 2002-03
User Fees	$3,388,999	$3,490,526	$3,593,327	$3,498,017	$3,616,863
Connection/SDC	$143,610	$252,160	$129,895	$186,245	$178,710
Interest	$126,031	$133,433	$127,233	$44,692	$28,580
Other	$40,833	$34,133	$149,156	$39,396	$19,500
Interfund Loan Repayment					
Total Revenues	**$3,699,473**	**$3,910,252**	**$3,999,611**	**$3,768,350**	**$3,843,653**

EXPENSES

Description	Actual FY 1998-99	Actual FY 1999-00	Actual FY2000-01	Actual FY 2001-02	Actual FY 2002-03
Operation and Maintenance	$1,740,404	$1,730,649	$1,874,473	$1,936,539	$2,042,172
Debt Service	$550,211	$553,027	$537,622	$648,256	$539,690
Capital Improvements	$1,726,942	$1,466,889	$1,890,048	$1,452,493	$1,397,914
Other	($2,185)				($6,209)
Total Expenses	**$4,015,372**	**$3,750,565**	**$4,302,143**	**$4,037,288**	**$3,973,567**

RECAP

Description	Actual FY 1998-99	Actual FY 1999-00	Actual FY2000-01	Actual FY 2001-02	Actual FY 2002-03
Revenues	$3,699,473	$3,910,252	$3,999,611	$3,768,350	$3,843,653
Expenses	$4,015,372	$3,750,565	$4,302,143	$4,037,288	$3,973,567
Net Change in Cash	**($315,899)**	**$159,687**	**($302,532)**	**($268,938)**	**($129,914)**

CASH RESERVE

Description	Actual FY 1998-99	Actual FY 1999-00	Actual FY2000-01	Actual FY 2001-02	Actual FY 2002-03
Beginning Cash	$2,592,795	$2,276,896	$2,436,583	$2,134,051	$1,865,113
Net Change in Cash	($315,899)	$159,687	($302,532)	($268,938)	($129,914)
Ending Cash	**$2,276,896**	**$2,436,583**	**$2,134,051**	**$1,865,113**	**$1,735,199**

NET CASH AVAILABLE PRIOR TO CAPITAL EXPENDITURES (REVENUE - O&M – DEBT - LOANS)

Description	Actual FY 1998-99	Actual FY 1999-00	Actual FY2000-01	Actual FY 2001-02	Actual FY 2002-03
Net	**$1,411,043**	**$1,626,576**	**$1,587,516**	**$1,183,555**	**$1,268,000**

Table 6.3. Continued, Ten-Year History, Water Enterprise Fund, FY 1998-99 Through FY 2007-08

REVENUE

Description	**Actual FY 2003-04**	**Actual FY 2004-05**	**Actual FY 2005-06**	**Actual FY 2006-07**	**Adopted FY 2007-08**
User Fees	$3,864,455	$3,798,683	$4,049,397	$4,473,799	$4,548,000
Connection/SDC	$118,800	$310,225	$220,870	$153,720	$225,000
Interest	$31,883	$60,732	$111,503	$128,367	$85,000
Other	$10,073	$41,326	$11,933	$26,155	$10,000
Interfund Loan Repayment		$0	$0	$0	*$1,130,000*
Total Revenues	**$4,025,211**	**$4,210,966**	**$4,393,703**	**$4,782,041**	**$5,998,000**

EXPENSES

Description	**Actual FY 2003-04**	**Actual FY 2004-05**	**Actual FY 2005-06**	**Actual FY 2006-07**	**Adopted FY 2007-08**
Operation and Maintenance	$2,058,384	$2,204,736	$2,552,372	$2,843,943	$3,478,476
Debt Service	$531,870	$533,420	$543,680	$531,930	$0
Capital Improvements	$1,170,691	$620,548	$1,918,171	$1,048,949	$2,213,000
Other	($160,000)		$23,000	$1,050,000	
Total Expenses	**$3,600,945**	**$3,358,704**	**$5,037,223**	**$5,474,822**	**$5,691,476**

RECAP

Description	**Actual FY 2003-04**	**Actual FY 2004-05**	**Actual FY 2005-06**	**Actual FY 2006-07**	**Adopted FY 2007-08**
Revenues	$4,025,211	$4,210,966	$4,393,703	$4,782,041	$5,998,000
Expenses	$3,600,945	$3,358,704	$5,037,223	$5,474,822	$5,691,476
Net Change in Cash	**$424,266**	**$852,262**	**($643,520)**	**($692,781)**	**$306,524**

CASH RESERVE

Description	**Actual FY 2003-04**	**Actual FY 2004-05**	**Actual FY 2005-06**	**Actual FY 2006-07**	**Adopted FY 2007-08**
Beginning Cash	$1,735,199	$2,159,465	$3,011,729	$2,368,209	*$1,675,428*
Net Change in Cash	$424,266	$852,262	($643,520)	($692,781)	$306,524
Ending Cash	**$2,159,465**	**$3,011,727**	**$2,368,209**	**$1,675,428**	**$1,981,952**

NET CASH AVAILABLE PRIOR TO CAPITAL EXPENDITURES (REVENUE - O&M - DEBT - LOANS)

Description	**Actual FY 2003-04**	**Actual FY 2004-05**	**Estimated FY 2004-05**	**Actual FY 2006-07**	**Adopted FY 2007-08**
Net	**$1,594,957**	**$1,472,810**	**$1,274,651**	**$356,168**	**$2,519,524**
			Inter Fund Loans	*$1,050,000*	*($1,130,000)*
			Net Without Loans	*$1,406,168*	*$1,389,524*

There are several important conclusions based on a review of the ten years of historical data, including:

- 2 percent average annual increase in water sales due to growth alone (independent of water rate increases)
- 6 percent average annual increase in O&M expenses. Note that O&M increases reflect not only annual increases in personnel costs, but also increases in utilities, chemicals, materials, fuel, contracted labor, etc.
- $1.4 million is currently available on an annual basis, after O&M expenses, to fund capital improvements, cash reserves, and contingencies
- $4.5 million annual revenue from user fees (current budget)
- $0.225 million annual revenue SDCs (current budget)

6.5 PAST PROJECTIONS FOR WATER ENTERPRISE FUND

The City Council approved a 10 percent rate increase which became effective January 1, 2005, and three subsequent increases of 2.5 percent per year. Funding projections presented to the City Council in April 2004 estimated that an initial 10 percent rate increase on July 1, 2004, followed by annual increases of 2 percent, would provide $1.66 million for capital improvements in FY 2007/08, after an outstanding bond issue was paid off. The updated estimate is $1.4 million in FY 2007/08; the difference from initial projections is within approximately 5 percent of the total budget for the Water Enterprise Fund.

Based on past estimates, it is believed that future financial projections can be made with sufficient accuracy for planning and budgeting purposes. The projections should be updated on a annual basis to reflect actual costs and new information.

6.6 FIVE-YEAR PROJECTIONS FOR WATER ENTERPRISE FUND

6.6.1 Five-Year Forecasts With Annual CPI Increases Only

Table 6.5 provides a five-year financial forecast for the Water Enterprise Fund, with the following assumptions:

1) Demand for water will increase at an average annual rate of 2%
2) Annual CPI of 3%
3) Annual increase in O&M of 5%
4) Maintain an average annual expenditure of $1.5 million for capital projects in 2008/09 dollars, with construction inflation increases equal to CPI
5) Annual water rate increase equal to CPI

The forecast indicates that annual CPI increases plus growth (increases in sales due to new development) will be adequate to fund projected O&M expenses plus an average annual expenditure of $1.5 million (2008/09 dollars). This is a critical finding from a budgeting standpoint since this demonstrates:

1) *Without the treatment plant expansion, annual CPI (inflation) adjustments appear adequate to fund projected operating costs plus an annual expenditure of $1.5 million for capital projects*

2) *Annual CPI increases are intended to offset inflationary increases – not to fund major capital projects, and rate increases to fund the water treatment plant expansion will be in addition to CPI adjustments*

6.6.2 Budget Forecasts to Finance Plant Upgrade with Loan Plus Average Annual Expenditure of $1.5 Million for Annual Capital Expenditures

Table 6.4 provides a five-year financial forecast assuming that the City borrows $8 million at an annual interest rate of 5 percent and a payback period of 20 years. The annual loan payment would be approximately $642,000.

Forecasts indicate a water rate increase of 15 percent will be sufficient to repay the loan. The average residential water bill in Roseburg is estimated at $22 per month. An increase of 15 percent is the equivalent to raising the average residential bill by $3.30 per month. Phasing the increase in over two years at 7.5 percent annually is recommended, resulting in an approximate increase of $1.65 per month for residential customers. CPI adjustments are recommended in addition to increases for loan payment. Assuming annual CPI adjustments of 3 percent, the total rate increases would be approximately 10 percent a year for two years, and then annual CPI adjustments thereafter.

Table 6.4. 5-Year Financial Projections, Water Enterprise Fund, Annual CPI Increases Only

Annual Increase Water, Demand	2.00%
Annual Increase O&M	5.00%
Annual CPI	3.00%

REVENUE

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
User Fees	$4,775,400	$5,014,170	$5,264,879	$5,528,122	$5,804,529
Connection/SDC	$231,750	$238,703	$245,864	$253,239	$260,837
Interest	$85,000	$87,550	$90,177	$92,882	$95,668
Other					
Total Revenues	**$5,092,150**	**$5,340,423**	**$5,600,919**	**$5,874,244**	**$6,161,033**

EXPENSES

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Operation and Maintenance	$3,652,400	$3,835,020	$4,026,771	$4,228,109	$4,439,515
Debt Service	$0	$0	$0	$0	$0
Capital Improvements	$1,500,000	$1,545,000	$1,591,350	$1,639,091	$1,688,263
Total Expenses	**$5,152,400**	**$5,380,020**	**$5,618,121**	**$5,867,200**	**$6,127,778**

RECAP

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Revenues	$5,092,150	$5,340,423	$5,600,919	$5,874,244	$6,161,033
Expenses	$5,152,400	$5,380,020	$5,618,121	$5,867,200	$6,127,778
Net Change in Cash	**($60,250)**	**($39,597)**	**($17,202)**	**$7,044**	**$33,255**

CASH RESERVE

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Beginning Cash	$1,981,952	$1,921,702	$1,882,105	$1,864,903	$1,871,947
Net Change in Cash	($60,250)	($39,597)	($17,202)	$7,044	$33,255
Ending Cash	**$1,921,702**	**$1,882,105**	**$1,864,903**	**$1,871,947**	**$1,905,202**

NET CASH AVAILABLE PRIOR TO CAPITAL EXPENDITURES (REVENUE - O&M - DEBT - LOANS)

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Net	**$1,439,750**	**$1,505,403**	**$1,574,148**	**$1,646,134**	**$1,721,519**

Table 6.5. 5-Year Financial Projections, Loan Money to Finance Treatment Plant Expansion, Annual Expenditure of $1.5 Million for Other Capital Projects, CPI Adjustments

Annual Increase Water, Demand	2.00%				
Annual Increase O&M	5.00%				
Annual CPI (Capital Increase)	3.00%				
Loan Principal	$8,000,000				
Loan Interest Rate	5.00%		Loan Payback Period		20
Annual Loan Payment	$641,941				
Average Annual CIP, 2008-09 $	$1,500,000				
		Capital	CPI	Total	
Rate Increase - First Year		7.50%	3.00%	10.50%	
Rate Increase - Second Year		7.50%	3.00%	10.50%	

REVENUE

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
User Fees	$5,116,500	$5,756,063	$6,043,866	$6,346,059	$6,663,362
Connection/SDC	$231,750	$238,703	$245,864	$253,239	$260,837
Interest	$85,000	$87,550	$90,177	$92,882	$95,668
Total Revenues	**$5,433,250**	**$6,082,315**	**$6,379,906**	**$6,692,180**	**$7,019,867**

EXPENSES

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Operation and Maintenance	$3,652,400	$3,835,020	$4,026,771	$4,228,109	$4,439,515
Debt Service	$0	$641,941	$641,941	$641,941	$641,941
Capital Improvements	$1,500,000	$1,545,000	$1,591,350	$1,639,091	$1,688,263
Total Expenses	**$5,152,400**	**$6,021,960**	**$6,260,061**	**$6,509,141**	**$6,769,719**

RECAP

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Revenues	$5,433,250	$6,082,315	$6,379,906	$6,692,180	$7,019,867
Expenses	$5,152,400	$6,021,960	$6,260,061	$6,509,141	$6,769,719
Net Increase (Decrease) in Cash	**$280,850**	**$60,355**	**$119,844**	**$183,040**	**$250,148**

CASH RESERVE

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Beginning Cash	$1,981,952	$2,262,802	$2,323,157	$2,443,001	$2,626,041
Net Change in Cash	$280,850	$60,355	$119,844	$183,040	$250,148
Ending Cash	**$2,262,802**	**$2,323,157**	**$2,443,001**	**$2,626,041**	**$2,876,189**

NET CASH AVAILABLE PRIOR TO CAPITAL EXPENDITURES (REVENUE - O&M - DEBT - LOANS)

Description	Projected FY 2008-09	Projected FY 2009-10	Projected FY 2010-11	Projected FY 2011-12	Projected FY 2012-13
Net	**$1,780,850**	**$1,605,355**	**$1,711,194**	**$1,822,130**	**$1,938,411**

6.7 REIMBURSEMENT SDC

The proposed treatment plant expansion will address: a) regulatory and maintenance issues related to existing facilities approaching their 20-year design life and b) provide additional treatment capacity. A reimbursement SDC of approximately $1,100 per ERU (Section 5.2) will be assessed to new development to recover costs of the reserve capacity included in the treatment plant expansion. SDC revenue is included in financial forecast presented in Table 6.5.

6.8 WATER RATE SURCHARGE

Instead of issuing bonds, another option for financing the treatment plant upgrade is to generate a large enough cash reserve, with a water rate surcharge, so that no borrowing is required. The existing facilities were financed with a water rate surcharge. Circumstances are different today than when the treatment plant was originally constructed in phases. Components of the old 1935 plant were used for treatment, providing the flexibility to build the replacement plant in phases from 1987 through 1992. Today, the proposed plant upgrade is an addition to the existing facility, and the project does not lend itself to phasing. The upgrade will be constructed as a single project.

To pay cash for the upgrade would require a sinking fund to generate adequate cash reserves prior to commencing construction. A surcharge of $2 million per year (approximately 45 percent of the current revenue) for 4 years would be necessary to generate a cash reserve of $8 million. The surcharge would be the equivalent of a $10 per month increase for an average residential customer. If the City decides to pursue financing the project with a sinking fund rather than issuing bonds, the structure of the surcharge should be evaluated in more detail – i.e., should the surcharge be added to the base rate, the commodity charge, or a combination of both.

Interest rates were relatively high in the 1980's when the design of the plant was initiated, and significant savings were realized by paying cash and building the project in phases. Today, interest rates are relatively low and potential savings by paying with cash would be offset somewhat from increased costs due to inflation.

6.9 WATER RATE COMPARISON

Ten other water purveyors in Douglas County were surveyed. Results of the survey are summarized in Table 6.6. Some communities charge for water consumption by the cubic foot (as in Roseburg), while others charge by the gallon. To simplify the comparison, rates are listed both in gallons and cubic feet. One cubic foot equals 7.5 gallons. An average residential customer in Roseburg uses approximately 1,000 cubic feet (7,500 gallons per month). Comparison of rates with other communities is not a basis for setting rates, but the comparison provides an indicator of the relative level of the rates being charged. Compared to the communities surveyed, Roseburg's rates are approximately 50 percent of the average, both inside and outside the City limits.

Table 6.6. Water Rate Comparison, Douglas County Communities, January 2008 Rates

			Gallons		Cubic Feet		
Water Purveyor	Inside/ Outside	Monthly Base Rate	Minimum Number with Base	Usage Charge Per 1000 Gallons	Minimum Number with Base	Usage Charge Per 100 Cu Ft	Cost per 7,500 Gallons or 1,000 cu ft
Canyonville	Inside	$28.00	9,000	$1.60	1,200	$1.20	$28.00
	Outside	$56.00	9,000	$3.20	1,200	$2.40	$56.00
Glendale	Inside	$24.67	3,000	$5.00	400	$3.75	$47.17
	Outside	$37.00	3,000	$5.00	400	$3.75	$59.50
Myrtle Creek	Inside	$20.00	3,000	$2.67	400	$2.00	$32.00
	Outside	$40.00	3,000	$5.33	400	$4.00	$64.00
Oakland	Inside	$59.06	0	$2.20	0	$1.65	$75.56
	Outside	$79.20	0	$2.20	0	$1.65	$95.70
Riddle	Inside	$40.00	7,500	$3.00	1,000	$2.25	$40.00
	Outside	$45.00	7,500	$3.00	1,000	$2.25	$45.00
Roberts Creek	Inside	$11.50	400	$2.10	53	$1.58	$26.20
Water District	Outside	$23.00	400	$2.60	53	$1.95	$41.20
Sutherlin (1)	Inside	$0.00	0	$6.90	0	$5.18	$51.75
	Outside	$0.00	0	$6.90	0	$5.18	$51.75
Umpqua Water	Inside	$14.00	1,000	$4.29	133	$3.22	$41.89
Basin	Outside	NA	NA	NA	NA	NA	NA
Winston-Dillard	Inside	$27.00	4,000	$2.25	533	$1.69	$34.88
Water District	Outside	$54.00	4,000	$2.25	533	$1.69	$61.88
Yoncalla	Inside	$26.00	1,500	$1.75	200	$1.31	$36.50
	Outside	$52.00	1,500	$3.50	200	$2.63	$73.00
Average	**Inside**	**$25.02**	**2,940**	**$3.18**	**392**	**$2.38**	**$41.39**
	Outside	**$42.91**	**3,156**	**$3.78**	**421**	**$2.83**	**$60.89**
Roseburg	**Inside**	**$8.49**	**0**	**$1.80**	**0**	**$1.35**	**$21.99**
	Outside	**$18.49**	**0**	**$1.80**	**0**	**$1.35**	**$31.99**

Notes:
1. Information was collected by telephone survey. Computations have not been confirmed and should be considered preliminary.
2. Sutherlin charges on equivalent residential unit basis, but rates are computed as shown
3. Tri City Water District not included because revenue includes property taxes
4. Reedsport not included because of unfiltered water and some unmetered customers
5. Elkton not included because of relatively low number of connections
6. Dixonville customers pay an additional $10 monthly surcharge in addition to outside rate for Roseburg

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