Report

Water System Master Plan

Prepared for

City of The Dalles

May 2006

Prepared by

CH2MHILL

Contents

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Exhibits

Acronyms and Abbreviations

µg/L	micrograms per liter
ADD	average day demand
ASR	aquifer storage and recovery
CAC	Citizens Advisory Committee
CCI	construction cost index [ENR]
cfs	cubic feet per second
CIP	capital improvements plan
DAF	dissolved air flotation
DBP	disinfection by-products
D/DBP	disinfectants/disinfection by-products
DSL	Division of State Lands
EA	Environmental Assessment
EIS	environmental impact statement
ENR	*Engineering News-Record*
EPA	U.S. Environmental Protection Agency
FONSI	Finding of No Significant Impact
gal/sf	gallons per square foot
gpcd	gallons per capita per day
gpm	gallons per minute
HAA5	five regulated haloacetic acids
HDPE	high-density polyethylene
IDSE	initial distribution system evaluation
IESWTR	Interim Enhanced Surface Water Treatment Rule
LRAA	locational running annual average
LT1 ESWTR	Long-Term 1 Enhanced Surface Water Treatment Rule
LT2 ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
MCLG	maximum contaminant level goal
MCLs	maximum contaminant levels
MDD	maximum day demand

MG	million gallons
mgd	million gallons per day
mL	milliliter
mg/L	milligrams per liter
MMD	maximum monthly demand
NEPA	National Environmental Policy Act
NTU	nephelometric turbidity unit
PHD	peak hour demand
PRV	pressure reducing valve
psi	pounds per square inch
PVC	polyvinyl chloride
RH	residential high [density]
RL	residential low [density]
RMH	residential mobile home
ROD	Record of Decision
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TDS	total dissolved solids
TTHM	total trihalomethanes
UFRV	unit filter run volume
UGB	urban growth boundary
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
UV	Ultraviolet
WRD	Water Resources Department
WTP	water treatment plant

Executive Summary

The City of The Dalles has a long history of reliably providing its customers with high-quality drinking water. This Water Master Plan provides a road map for the coming years, enabling The Dalles to continue meeting its customers' needs in a cost-effective manner. The plan is summarized in a capital improvements plan (CIP) that identifies specific project needs and approximate dates for their implementation. A financial plan to fund routine operation and maintenance as well as the capital projects has been developed as part of this master plan project and is presented in a separate document.

Master Plan Goals

Five of the highest priority goals for the 2006 Master Plan were as follows:

1. Develop a 20-year capital improvements plan
2. Based on the capital improvements plan, develop a financial plan that identifies rate needs and recommends system development charges
3. Provide a comprehensive analysis of the system, including demand forecasts, regulatory compliance, and condition assessment
4. Determine a long-term plan for water source and treatment development that considers operational and economic efficiencies in both near term and long term time frames.
5. Provide a comprehensive network model update—use the model to provide a thorough analysis of the distribution system needs and provide the updated version to the city for use as a tool by city staff

Population and Water Use

The Dalles' water system currently serves approximately 11,000 people, or approximately 88 percent of the city's population. The remainder of the city is served by the Chenowith Public Utility District (PUD). The water system service area is shown on the map in **Exhibit ES-1**.

The system currently supplies 1.1 billion gallons of water to the customers or an average of 3 million gallons per day (mgd). The city's commercial and industrial customers have always been metered. Meters at residences were installed between 1993 and 1995, and billing based on residential meter readings began in 1996.

As typical for Oregon utilities, The Dalles' water demands are twice as high during the summer months as they are during the winter months because of outdoor irrigation. Summer demands also fluctuate from one year to the next because of variations in temperatures and rainfall. Peak demands since 1995 have averaged 6.0 mgd with a high of 7.8 mgd on July 28, 2003.

On a per capita basis, the average use has been 275 gallons per person per day in recent years. It reaches 640 gallons per person on a peak summer day. These values include all water that is used (residential, commercial, and industrial), divided by the total service area population. Residential use accounts for more than 55 percent of the use within the system. Commercial, industrial, and public agencies (city parks, schools, etc.) use the remaining 45 percent.

Unaccounted-for water in The Dalles' system, or the difference between water delivered to the system and customer meter readings, has averaged 15 percent in recent years. This means that on average the city is not accounting for 440,000 gallons each day. The Water Resources Department's goal for municipal water suppliers is ten percent or less.

Projected Water Use

Future demands on the City of The Dalles' water system were projected by applying per capita water use values to population projections. The current service population is estimated at 11,000. At an annual population growth rate of 1.1 percent, The Dalles is expected to reach its buildout population of 14,400 in the year 2030. Buildout is limited by the amount of remaining developable area within the urban growth boundary (UGB). The proposed capital improvements plan and projected buildout demands will need adjustment if the actual growth rate is higher than the projected annual rate of 1.1 percent or if the UGB is expanded.

The projected buildout average day demand will equal approximately 4.0 mgd. This projection does not take into account an allowance for future industrial growth. With a 3.0 mgd industrial use allowance, the projected average day demand may reach 7.0 mgd.

The projected maximum day demand for system buildout equals 9.2 mgd. With the inclusion of the industrial allowance and a weather allowance, this value may equal 13.2 mgd. The weather allowance accounts for the variations in summer demands related to temperature and rainfall patterns.

Exhibit ES-2 summarizes both the historic values and the projections for average and maximum water use within The Dalles' service area.

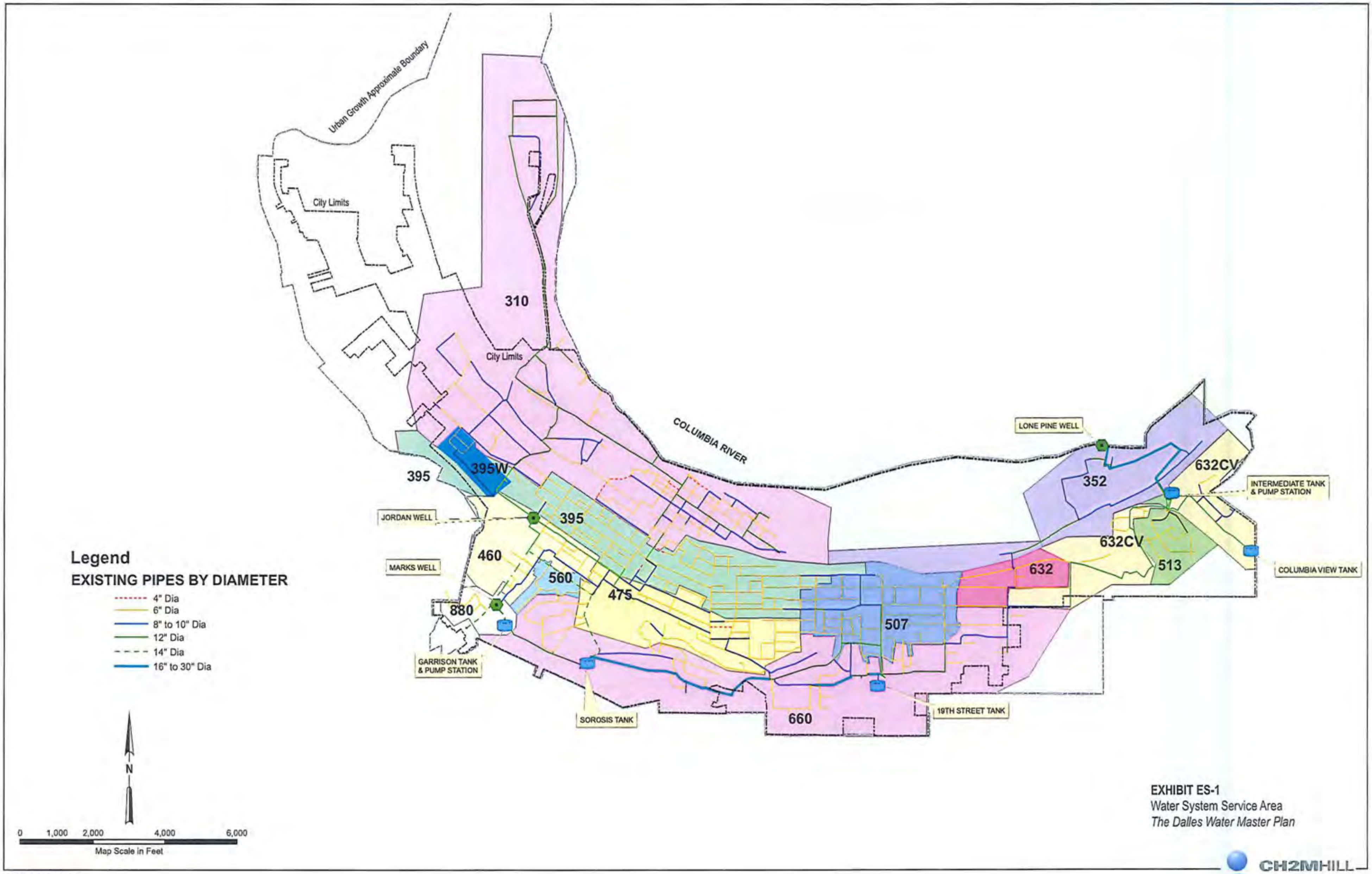

EXHIBIT ES-1
Water System Service Area
The Dalles Water Master Plan

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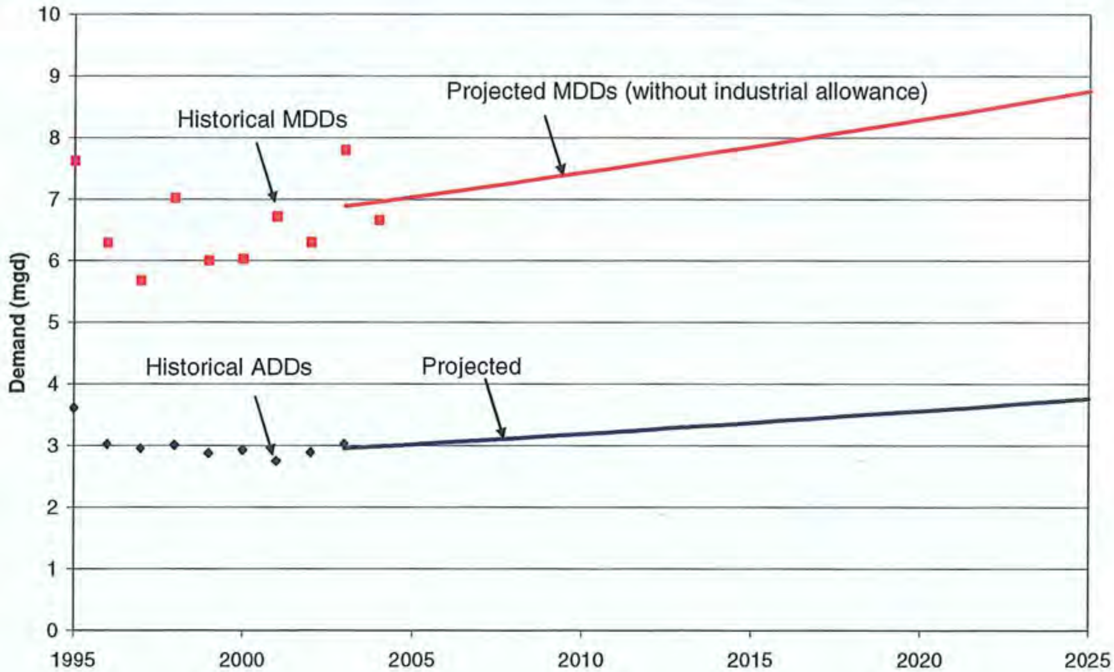
EXHIBIT ES-2 The Dalles Historical and Projected ADDs and MDDs



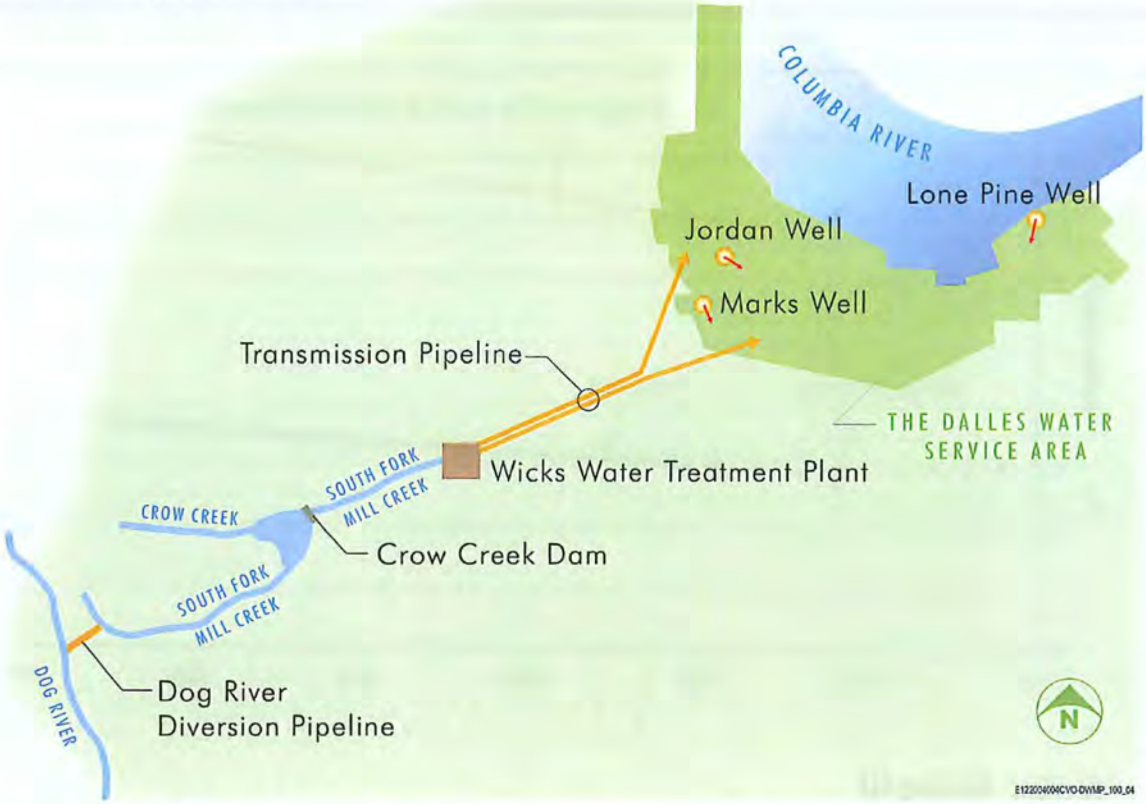
Water Supply

From January 1995 to June 2004, 94 percent of the water provided to the city's customers was obtained from the city's surface water source on South Fork Mill Creek. The remaining 6 percent was provided by three wells located within the city. **Exhibit ES-3** summarizes the city's water rights. **Exhibit ES-4** provides a schematic representation of the supply facilities.

EXHIBIT ES-3
Surface Water Rights

Source	Priority Date	Quantity	Description	Status
South Fork Mill Creek	1862	2 cfs (1.3 mgd)	Run-of-river right; diversion location is at the present Wicks WTP intake	Certificate
Dog River	1870	All water in Dog River at point of diversion	Allows diversion of Dog River flow into South Fork Mill Creek watershed	Certificate
Crow Creek Dam	1967	955 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Certificate
Columbia River	1986	40 cfs (26 mgd)	Point of diversion is upstream of dam; city has not used this right	Permit
Crow Creek Dam	1999	2100 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Permit

EXHIBIT ES-4
The Dalles Water Supply Schematic



Surface Water Supply

The city's surface water supply system consists of the following major components (in order from upstream to downstream):

- Dog River Diversion Pipeline. This three and one-half mile pipeline diverts up to 8 mgd from the Dog River Watershed to the South Fork Mill Creek Watershed.
- Crow Creek Dam. This dam, located at the confluence of the South Fork Mill Creek with Crow Creek, impounds up to 800 acre-feet (260 MG).
- South Fork Mill Creek Intake. The intake facility was constructed in 2002 to comply with fish screening requirements. It has a capacity of 12 mgd. Water flows by gravity from the intake to the Wicks Water Treatment Plant (WTP).
- Wicks WTP. Surface water is treated at the Wicks WTP. The evaluation and recommendations relating to the plant are summarized in paragraphs that follow.
- Finished Water Transmission Pipelines. Two pipelines carry water by gravity from the Wicks WTP to the city distribution system.

The Wicks WTP, located about 7 miles south of the city, was constructed in 1947. This plant provides a high quality drinking water to the community at a relatively low cost. The finished water meets all state and federal standards. Production costs have been minimized because water flows to and from the plant by gravity, and because the plant has not required any significant capital investments in recent years.

The plant uses two parallel trains of flocculation, sedimentation, and tri-media filtration. It was designed to allow the addition of a third parallel train, thereby increasing the capacity by 50 percent. The plant is located in a narrow ravine, bounded by steep, rock walls on the east side and Mill Creek on the west side. These physical limitations leave room for a third parallel train, but not a fourth train.

A capacity-rating of the plant was performed as part of this master plan. The most limiting process was found to be filtration, having a recommended capacity of 3.4 mgd. The flocculation process is the second most limiting process, at 4.9 mgd.

The plant has exceeded these flow limits on some occasions. It has treated flows up to 6.4 mgd (gross production). This has only been possible for limited periods by careful operation and at the expense of generating high waste flows (approximately 16 percent compared to a typical goal of 5 percent). At 16 percent waste flow, the net production from a gross production of 6.4 mgd has equaled approximately 5.4 mgd.

The following major surface water supply improvement projects are included in the CIP:

- Replace and expand the capacity of the Dog River diversion pipeline: this pipeline, which is more than 100 years old, needs replacement because of its physical condition and to increase its capacity
- Expand and improve the Crow Creek Dam: raising the dam 35 feet will more than double the capacity of the dam. In addition, it is necessary to upgrade the spillway to enable it to pass the maximum probable flood.
- Implement a group of near-term improvements at the Wicks WTP to increase the capacity from 3.4 to 5.2 mgd. The largest cost item is the installation of a clearwell tank, which is needed to achieve compliance with new drinking water standards. Other improvements include filter and flocculation upgrades, and new solids drying beds.
- Expand the WTP to an ultimate capacity of 10 mgd using a modified treatment process, as described in Chapter 5. Simply adding a third parallel train will only achieve an ultimate capacity of 7.5 mgd; the modified approach yields 10 mgd.
- Replace the existing two finished water transmission pipelines. This project replaces the two aging pipelines, which have limited remaining useful lives, with a single pipeline having greater capacity than the combined capacity of the two lines.

Groundwater Supply

The Dalles currently uses three wells as part of its water supply: Lone Pine, Marks, and Jordan Wells. All three wells are located within the city limits. The Jordan and Marks Wells are located in the west-central area and Lone Pine Well is located on the east edge of the city near the I-84 Freeway.

The city has groundwater rights totaling 12.7 mgd. However, the actual groundwater supply capacity is significantly lower because of three factors:

- Production from the wells is limited to the pumping capacities of individual wells and the capacity of the distribution system at the locations to which the wells are connected.
- The area in which the city's wells are located has been designated The Dalles Critical Groundwater Area by the Oregon Water Resources Department (OWRD) because of declining water levels in the aquifer. Although the levels have stabilized in recent years, the annual withdrawal by The Dalles and other groundwater users may be restricted by the state if water levels begin again to decline.
- The groundwater quality from the Marks and Jordan Wells is undesirable, so the city limits their contribution to the system. Both wells produce water with high manganese levels; therefore, these wells are not considered a reliable part of the city's supply.

The Lone Pine Well provided an average of 1.1 mgd during August 2005. This was the highest sustained production ever obtained from this well. The aquifer and pump capacity exceed 1.1 mgd. However, the distribution piping limits the use of this well until additional transmission piping is added. The master plan recommends expansion of the groundwater supply by equipping the Lone Pine Well with a larger pump and motor, and adding a second well in the Lone Pine Well area. It will also be necessary to implement distribution improvements (pipelines and a pump station) to allow use of the expanded well capacity.

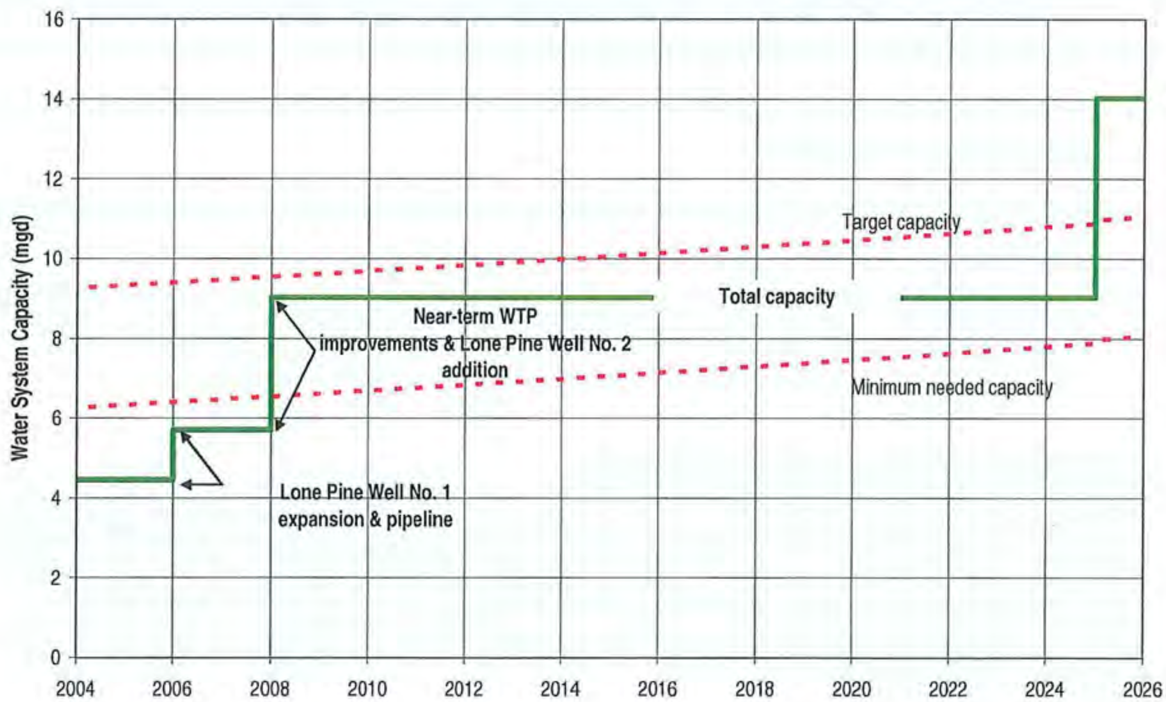
Supply Development Plan

A citizens advisory committee participated in the supply planning element of the project. They recommended maximizing the city's investment in the Wicks surface water supply system. The final supply plan, which balances long-term investment in the Wicks system with cost-effective investments in the groundwater supply, consists of the following components:

- Implement near-term improvements to the Wicks WTP to increase its capacity from 3.4 to 5.2 mgd.
- Expand the groundwater supply in the Lone Pine Well area by increasing the pumping capacity of Lone Pine Well No. 1 and adding a second well in the area.
- Implement the improvements needed to eventually achieve a capacity of 10 mgd from the Wicks system: expansion of the Crow Creek Dam, replacement of the Dog River Pipeline, Expansion of the Wicks WTP, and replacement of the Finished Water Transmission Pipelines.

The timing for these projects depends on permitting, available financing, and demand growth. The approximate dates are illustrated in **Exhibit ES-5**.

EXHIBIT ES-5. Capacity Development Plan for The Dalles



Regulatory Review

Through its success in meeting or exceeding state and federal water quality standards and additional criteria, The Dalles has been granted the Director's Award for completion of the Phase III program of the Partnership for Safe Water, a voluntary quality assurance program instituted by EPA and the American Water Works Association.

Several new rules are expected in the coming years. The federal rules of primary importance to The Dalles are the Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESTWR), the Stage 2 Disinfection By-Products Rule (Stage 2 D/DBPR), the Ground Water Rule, and the Lead and Copper Rule. Although these new rules will require increased monitoring, it is not expected that they will require significant capital investments. The one exception is that the new clearwell, which is one component of the Wicks WTP near-term improvements, is needed to ensure compliance with the Stage 2 D/DBPR.

Distribution System Analysis

A significant task of the water master plan was updating and revising the hydraulic model of The Dalles' distribution system and using this model to evaluate the distribution system's capability to meet both current and projected needs. This model was used to evaluate alternative solutions for deficiencies that were found. The model was submitted to The Dalles at the conclusion of the project for continued use by staff.

The primary recommendations from the distribution system analyses follow:

- Add pipelines (8-inch and 16-inch diameter) to transmit water from the Lone Pine Well area to the downtown area. This will enable the city to increase the groundwater supply from this well and a proposed new well in the area.
- Install a small booster pump station and control valves to improve the use of the existing Columbia View Reservoir.
- Combine two of the existing service levels to simplify operation and maintenance of the system.
- Install a new reservoir to serve the higher elevation area above the hospital as this area develops. This reservoir will also provide critically needed backup to the existing Sorosis Reservoir so that it can be removed from service for repainting.

Design and Operating Criteria

Design and operating criteria represent specific system goals to assure the water system complies with regulations and meets customer expectations. Criteria were developed by comparing current city practices and operational goals with regulatory standards for Oregon utilities and those for other states. Recommended design and operations criteria are detailed in Chapter 9.

Capital Improvements Plan

One of the goals for the City of The Dalles' Water Master Plan was to develop long-term guidance for decision-making: what facilities to build and when to build them; how to prioritize investments in the maintenance, repair, and rehabilitation of existing facilities; and how to adjust to changing conditions or intervening events. The outcome is presented in a comprehensive CIP table in Chapter 10. Some of the major projects identified in the CIP are listed **Exhibit ES-6**.

EXHIBIT ES-6
Major Projects Identified in Capital Improvements Plan

Start Date	End Date	Project Title	Description	Total Capital Cost
2006	2008	760 Zone: Supply reservoir tank	Add steel reservoir tank to serve a new development located above the hospital and to provide backup for Sorosis Reservoir.	$1,470,000
2006	2008	WTP near-term improvements	4.3 million gallon (MG) steel clearwell, as designed in Nov. 2003.	$3,750,000
2006	2025	Annual pipeline replacement	Allowance for distribution pipeline replacements ($75,000 per year for 20 years)	$1,560,000
2007	2008	New well	New production well in the area of Lone Pine Well	$1,380,000

EXHIBIT ES-6
Major Projects Identified in Capital Improvements Plan

Start Date	End Date	Project Title	Description	Total Capital Cost
2011	2012	Dog River Pipeline design & construction	18,500 feet of ductile iron pipeline, placed along existing alignment	$3,310,000
2011	2013	Crow Creek Dam raise	Raise dam by 35 feet and implement spillway improvements	$9,050,000
2018	2020	Finished Water Pipeline replacement	Replace existing two lines with a single, 24-inch diameter pipeline	$10,050,000
2022	2025	WTP expansion	Add rapid mix, new flocculation basin, new plate sedimentation basin, and 2 filters	$7,450,000
2026	2026	Iron/manganese treatment for Jordan Well	Install iron and manganese treatment facility for the Jordan Well	$1,260,000

The CIP project dates are approximate. The Dalles will adjust the projects and their implementation schedules to ensure that the system is managed efficiently to meet customer needs.

Financial Plan

A water system financial plan was prepared at the conclusion of the master plan project. It provides rate and system development charge (SDC) plans that will be sufficient to fund the capital projects that have been developed in this master plan. The financial plan was reviewed in a series of workshops and meetings with the city staff and city council. The financial plan is summarized in a separate document that is to be issued by mid-2006.

CHAPTER 1

Introduction

This Water System Master Plan provides a comprehensive, updated plan for the City of The Dalles water system. It builds on previous planning studies, including the Water Supply Study (1991) and other studies specific to the Crow Creek Dam and groundwater evaluations. This plan is a roadmap to the future, to help ensure that The Dalles continues to provide high-quality and reliable service in a cost-effective manner.

The Water System Master Plan is intended as a recommended plan and long-term guide. It includes discussion of specific projects and preparation of an updated, 20-year capital improvements plan (CIP). Although it presents specific projects and proposed dates for implementing these projects, it must be recognized that the plan is intended as a guide. The projects and their implementation schedules will be adjusted annually to ensure that the system is managed efficiently to meet customer needs.

Financial Plan

A financial plan, developed by Galardi Consulting, LLC, was prepared based on the CIP developed in this master plan. A separate document has been prepared that summarizes the financial plan, including both a rate analysis and a review of the city's system development charges.

Acknowledgements

Preparation of this plan was a joint effort between The Dalles and CH2M HILL. CH2M HILL expresses its gratitude to The Dalles staff for their critical input and enthusiastic cooperation during preparation of the Water System Master Plan. The following individuals provided major contributions.

City Staff

- Nolan Young, City Manager
- Brian Stahl, Public Works Director
- Dave Anderson, Water Quality Manager
- Dale McCabe, City Engineer
- Karen Skiles, Regulatory Compliance Manager
- Dan Durow, Community Development Director

Citizen Advisory Committee

- Greg Weast, Downtown Business Association
- Richard Elkins, Urban Renewal Agency
- Andrea Klass, Port of The Dalles
- Eric Gleason, Historic Landmarks Commission

- Jack Evans, Chamber of Commerce and Finance Community
- Bruce Irwin, Northern Wasco County Parks and Recreation
- Ivan Frasier, citizen-at-large
- Mary Ann Davis, citizen-at-large

City Council

- Robb Van Cleave, Mayor
- Mike Tenney, Councilor
- Chris Zukin, Councilor
- Jim Broehl, Councilor
- Dorothy Davison, Councilor
- Joe Seckora, Councilor

CH2M HILL'S project team included the following:

- Bob Fuller, Senior Engineer
- Paul Berg, Project Manager
- Skip Martin, Network Analysis
- Jennifer Henke, Network Analysis
- Mark Carlson, Water Treatment Plant Analysis
- Jim McWade, Transmission Pipeline Evaluations
- Sheryl Stuart, Project Engineer

CHAPTER 2

System Description

The City of The Dalles operates a community water system serving most residents of The Dalles, Oregon, and a small number of customers located outside the city limits. The city's water system has been assigned the state and federal Public Water System Identification No. 4100869.

This chapter provides an overview of the system by describing the customer base, recent water use history, water rights, and the facilities that make up the system.

Service Area and Population

Exhibit 2-1, at the end of this chapter, provides an overview map of The Dalles service area. The estimated service population for year 2004 is 11,000 out of a total city population of approximately 12,500. Most of the city's residents who are not served by the The Dalles water system obtain their drinking water from the Chenowith PUD.

Water Use

The city's system provides an average of 3.0 million gallons per day (mgd) of drinking water to the community. The average annual production has remained relatively constant since 1996, which was the first year that the city began billing customers according to their metered consumption. Prior to that time, all residential customers were charged a fixed monthly rate. The metered rate has resulted in reduced consumption. Since 1996, the average annual use has remained nearly constant even though the city's service population has increased.

The Dalles' demands show a marked increase during the summer months because of outdoor irrigation. The maximum summer day demand is approximately 2.2 times the annual average. The highest recorded single day demand for the system since 1996 was 7.8 mgd in 2003.

About 55 percent of water use in The Dalles system is by residential customers, with the remaining 45 percent used by commercial, industrial, and governmental customers.

Water Supply

The city has in recent years obtained approximately 94 percent of its water from surface water supplies located south of the city. The remaining 6 percent is provided by groundwater wells. The proportion of groundwater was slightly higher in 2005 because of a drought that limited the surface water supply.

Surface Water Supply

The city's surface water supply system, illustrated in **Exhibit 2-2**, consists of the following major components (in order from upstream to downstream):

- Dog River Diversion Pipeline. This wood-stave, three and one-half mile pipeline diverts up to 8 mgd from the Dog River watershed to the South Fork Mill Creek watershed. It was constructed 100 years ago.
- Crow Creek Dam. This dam is located at the confluence of the South Fork Mill Creek with Crow Creek. It impounds up to 800 acre-feet (260 million gallons).
- South Fork Mill Creek Intake. The intake facility, which was constructed in 2002 to comply with fish screening requirements, is located near the Wicks Water Treatment Plant (WTP). It has a capacity of 12 mgd. Water flows by gravity from the intake to the plant.
- Wicks WTP. This plant, described in the Chapter 5, has a rated capacity of 3.4 mgd.

The city holds five surface water right permits, three of which have been perfected (certificated). **Exhibit 2-3** summarizes information pertaining to these rights.

EXHIBIT 2-3
Surface Water Rights

Source	Priority Date	Quantity	Description	Status
South Fork Mill Creek	1862	2 cfs (1.3 mgd)	Run-of-river right; diversion location is at the present Wicks WTP intake	Perfected
Dog River	1870	All water in Dog River at point of diversion	Allows diversion of Dog River flow into South Fork Mill Creek watershed	Perfected
Crow Creek Dam	1967	955 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Perfected
Columbia River	1986	40 cfs (26 mgd)	Point of diversion is upstream of dam; city has not used this right	Undeveloped (permit only)
Crow Creek Dam	1999	2100 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Undeveloped (permit only)

Groundwater Supply

The Dalles currently uses three wells as part of its water supply: Lone Pine, Marks, and Jordan. Together, the three wells have supplied approximately 6 percent of the city's water supply on an annual basis over the past 10 years. All three wells are located within the city limits. The Jordan and Marks wells are located in the west-central area and Lone Pine Well is located on the east edge of the city near the I-84 Freeway. In addition, the 660-gpm Wicks

Well located near the Wicks WTP provides an emergency supply. Water from this well requires treatment at the Wicks WTP prior to distribution.
Exhibit 2-4 summarizes the city's groundwater rights. The city has nearly 13 mgd of groundwater permits and registrations and transfers and 8.4 mgd of perfected rights.

The available capacity for meeting the city's needs is significantly lower than the city's groundwater rights because of three factors:

- The production from the wells is limited to the pumping capacities of individual wells and the capacity of the distribution system at the locations to which the wells are connected.
- The area in which the city's wells are located has been designated The Dalles Critical Groundwater Area by the Water Resources Department (WRD) because of declining water levels in the aquifer. Although the levels have stabilized in recent years, the annual withdrawal by The Dalles and other groundwater users may be restricted by the state if water levels begin again to decline.
- The groundwater quality from the Marks and Jordan wells is undesirable, so the city limits their contribution to the system. Both wells produce water with high manganese levels. In addition, extended periods of pumping from the Marks Well results in high turbidity values from that source.

EXHIBIT 2-4
The Dalles Groundwater Rights

Well Name	Priority Date of Right	Rate (cfs)	Rate (gpm)	Rate (mgd)
Lone Pine	1959	4.46	2,000	2.88
City Hall	1923	5.12	2,300	3.31
Jordan	1953	5.50	2,468	3.55
Marks	1940	2.68	1,203	1.73
Stadelman	1910	0.37	165	0.24
Mill Creek	1945	1.50	673	0.97
Total Rights		**19.63**	**8,809**	**12.68**

Note: The water rights vary in their use of either cfs or gpm to describe the maximum allowed rate, so both units are presented, above. In addition, the units of mgd are used because this is the most common terminology used for describing water system capacity.

Water Treatment Plant

The Wicks WTP was constructed in 1947 and is located about 7 miles south of the city. It treats water from South Fork Mill Creek, which is fed by the Crow Creek Dam Reservoir and the Dog River diversion.

The plant has produced up to 5.4 mgd. However, using current design standards, the plant's rated capacity is 3.4 mgd. Near-term improvements have been described in this master plan that will increase the capacity to a reliable 5.0 mgd.

The Wicks WTP consistently provides a high quality drinking water to the community at a relatively low cost. The finished water quality meets all current drinking water standards. A primary reason for the low cost is that water flows by gravity to and from the plant. In addition, the plant has not required any significant rehabilitation in recent years.

Finished Water Transmission Pipelines

Water from the Wicks WTP is delivered to the city's customers by gravity flow, without the need for pumps. Two finished water transmission pipelines connect to the outlet of the plant clearwell, and transmit water approximately 7 miles north to the city limits. One is called the High Line and the other the Mill Creek Line. Their combined capacity is approximately 7.5 mgd.

The pipelines parallel each other and are located along Mill Creek for the first approximately 4.5 miles from the plant. At this point, the High Line alignment turns northeast and runs across private and public lands on a mostly direct route to Sorosis Reservoir, which is located in Sorosis Park. The Mill Creek pipeline continues along Mill Creek Road right-of-way to 16th Street, just west of Skyline Road, where its alignment turns east. This pipeline connects to Garrison Reservoir.

In addition to supplying water to the city's distribution system, each transmission line serves a limited number of customers on properties adjacent to the pipeline alignments. These services were generally granted many years ago in exchange for having the customers provide pipeline easements.

Distribution System

Exhibit 2-5 provides a schematic of the city's distribution system.

Service Zones

The Dalles' distribution system is divided into 13 service zones. They are labeled numerically, the label generally being reflective of the hydraulic grade line for service within the zone. The zones have been developed to provide acceptable pressures to customers.

Water from the Wicks WTP enters the system by gravity through the two transmission lines: one feeds Garrison Reservoir, and one feeds Sorosis Reservoir. The higher elevation zones, which are fed from these two reservoirs or directly from one of the transmission pipelines, feed the lower zones through pressure reducing valves (PRVs).

A smaller portion of water is supplied from the city's three wells. As shown in the system schematic, **Exhibit 2-5**, the Marks and Jordan Wells pump directly into the distribution system, and the Lone Pine Well has a dedicated pump line to the Intermediate Reservoir.

Exhibit 2-6 lists the existing services zones, elevation ranges for customer connections, and minimum and maximum static pressures.

EXHIBIT 2-6
Water System Service Zones

Service Zone Label	Lower Customer Elevation (Maximum Pressure)	Upper Customer Elevation (Minimum Pressure)
310	80 feet (100 psi)	193 feet (51 psi)
352	80 feet (118 psi)	241 feet (48 psi)
395	154 feet (104 psi)	300 feet (41 psi)
395W	160 feet (102 psi)	311 feet (37 psi)
460	190 feet (117 psi)	355 feet (45 psi)
475	285 feet (82 psi)	385 feet (39 psi)
507	243 feet (114 psi)	411 feet (42 psi)
513	311 feet (87 psi)	416 feet (42 psi)
560	335 feet (97 psi)	415 feet (63 psi)
632	335 feet (97 psi)	415 feet (63 psi)
632CV	302 feet (143 psi)	511 feet (52 psi)
660	348 feet (135 psi)	615 feet (19 psi)
880	220 feet (110 psi)	407 feet (29 psi)

The largest demand area is the 310 zone. This area encompasses the downtown as well as the port. It includes the majority of commercial and industrial customers. During peak summertime demand periods, this zone accounts for approximately one-third of the total system demand.

The gorge on the east side of the city, through which Highway 197 is located, restricts water movement in the east and west directions. This physical barrier does not completely eliminate movement of water into or out of the eastern zones (632CV, 513, and 352 zones), but it does limit the transfer of water. This results in two system conditions. One is that the city is unable to make full use of the Lone Pine Well because the demand in 632CV, 513, and 352 zones is insufficient to fully use the 2 mgd pumping capacity of this well. The second, related condition is that there is insufficient turnover in the Columbia View Reservoir when all water is being supplied from the Wicks WTP and the Lone Pine Well is off line. The city had removed this reservoir from service until summer 2005, when the Lone Pine Well was used, because of water quality concerns.

Storage

Distribution storage is provided in five reservoirs. **Exhibit 2-7** lists the reservoirs, including their overflow elevations, material type, and volume.

EXHIBIT 2-7
Reservoirs

No.	Name	Volume (million gallons)	Overflow Elevation (feet)	Material Type
1	Sorosis	3.0	660	Steel
2	Garrison	6.0	460	Steel
3	19th Street (also called Hospital)	3.0	507	Steel
4	Columbia View	3.0	632	Steel
5	Intermediate	1.0	352	Steel

Pump Stations

The Dalles system includes two booster pump stations: the Intermediate Pump Station, and the Garrison Pump Station.

The Intermediate Pump Station is located next to the Intermediate Reservoir. This pump station lifts water from the 352 zone, which is fed by Lone Pine Well, to the 632CV zone. It fills the Columbia View Reservoir. The Intermediate Pump Station has an approximate capacity of 3,500 gpm if both pumps are operating.

The Garrison Pump Station transfers water from the Garrison Reservoir (at an overflow elevation of 460 feet) into the Sorosis Reservoir (at an overflow elevation of 660 feet). The Garrison Pump Station is used only infrequently, during times when the Sorosis Reservoir is draining more quickly than it can be filled through the High Line.

Two small pressure enhancement stations located in the southern, 660 service zone, feed in parallel to provide adequate pressure for several houses located on the south ridge.

Distribution Pipe

The Dalles distribution system, not including the transmission pipelines, is comprised of 68 miles of pipelines. This represents about $25 million in replacement costs in today's dollars. Cast iron and ductile iron make up 95 percent of the pipe material that is in use.

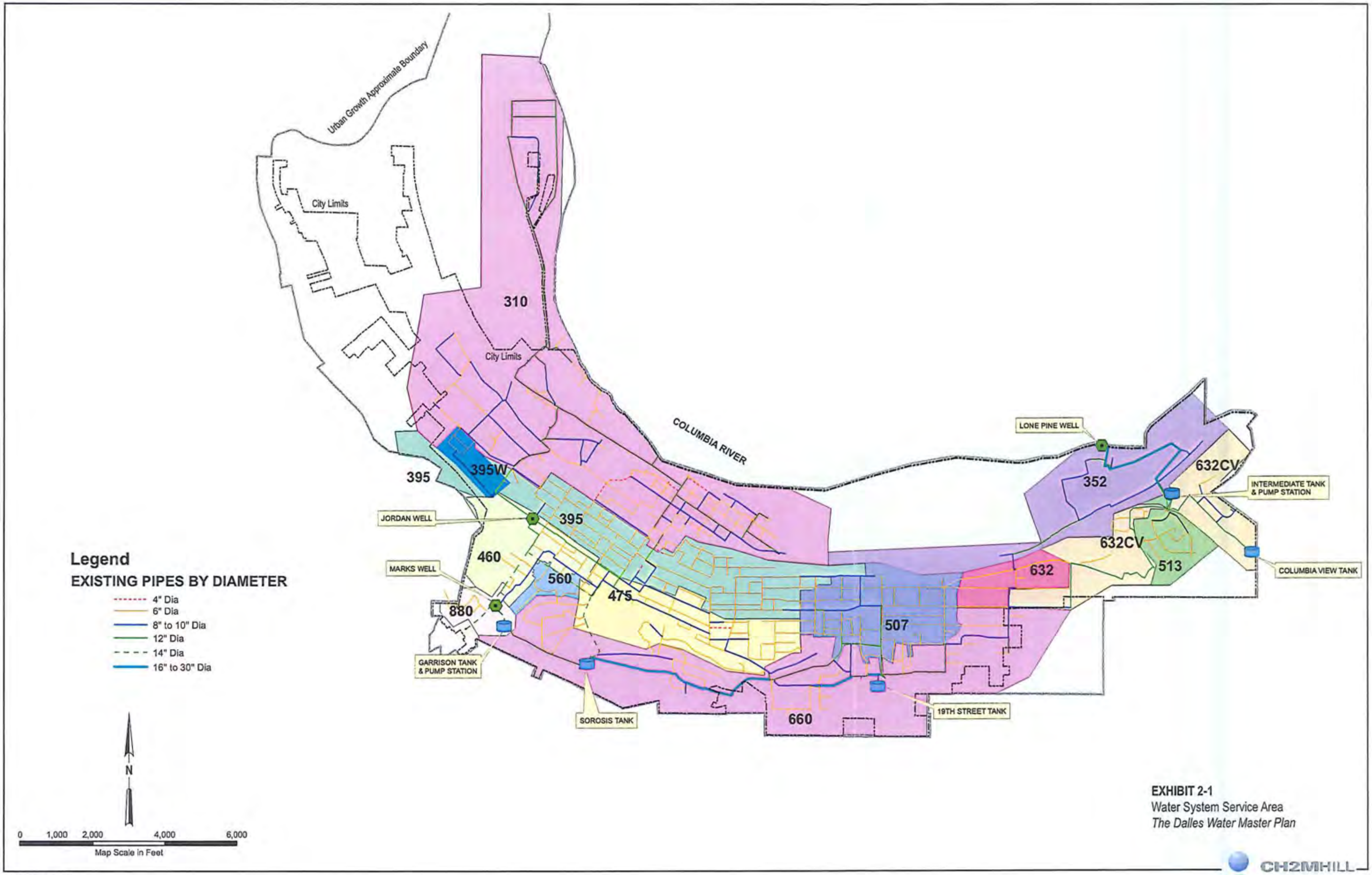


EXHIBIT 2-1
Water System Service Area
The Dalles Water Master Plan

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EXHIBIT 2-2
Existing Water Supply

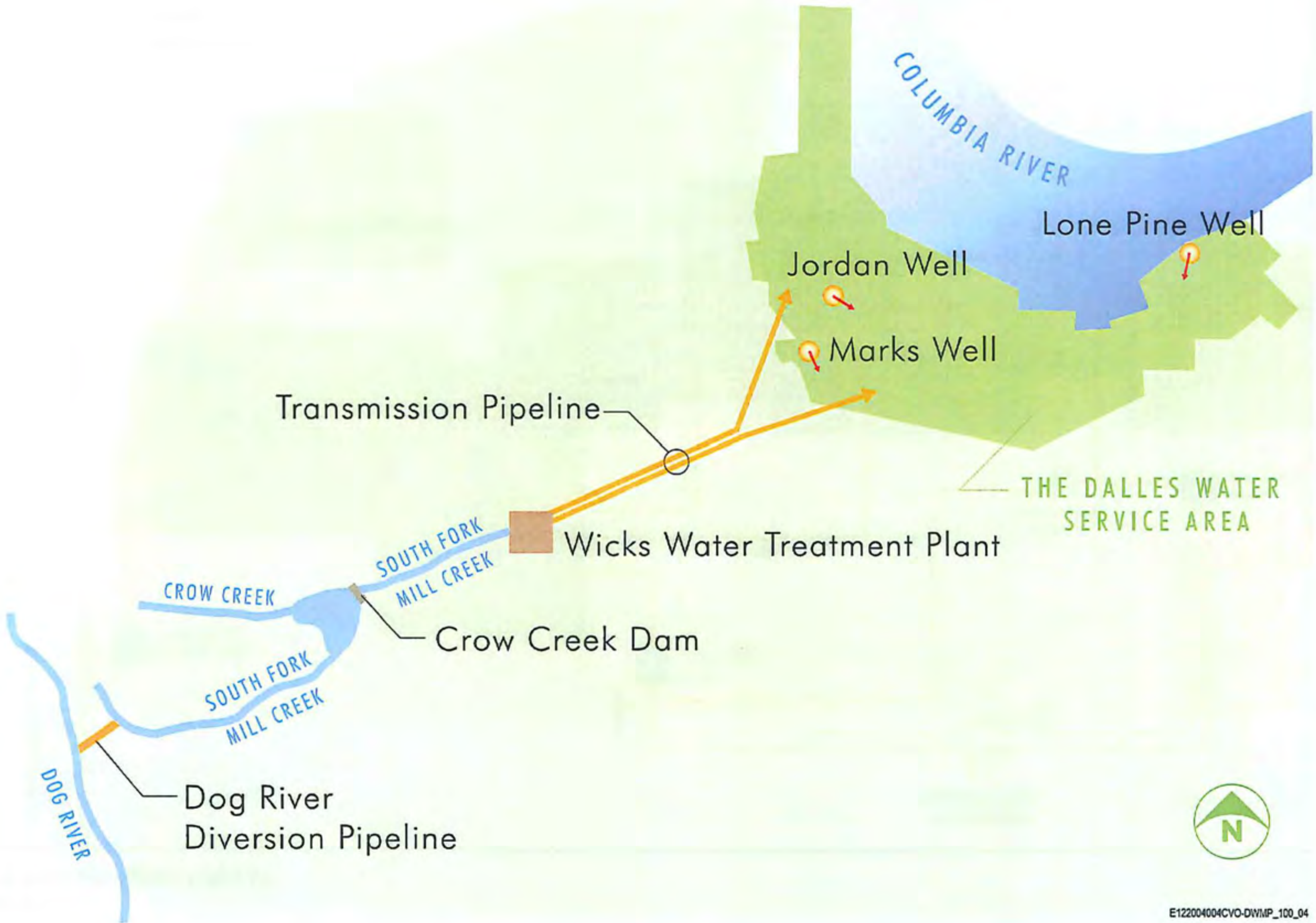

EXHIBIT 2-5
Existing Distribution System Schematic

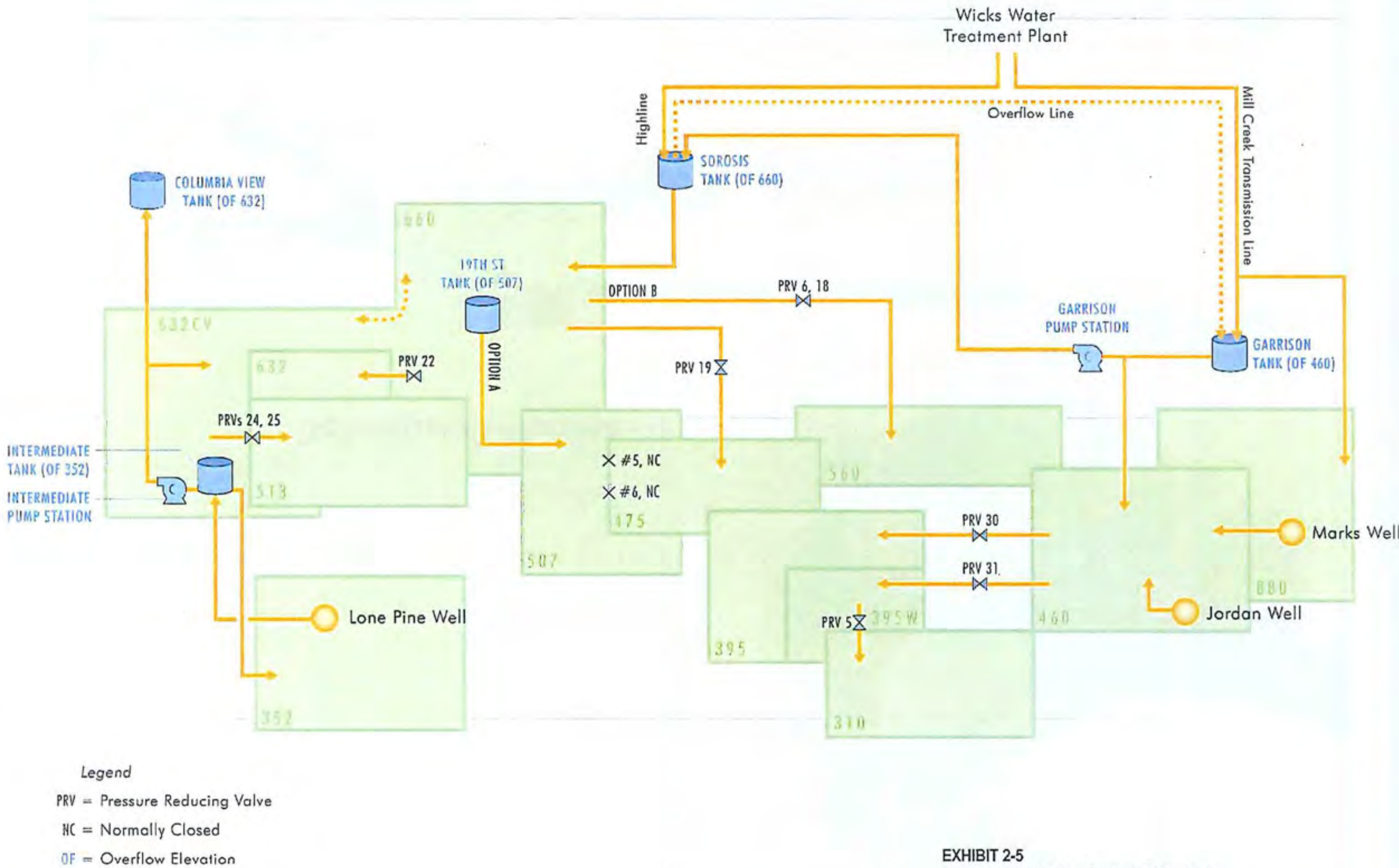

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EXHIBIT 2-5
Existing Distribution System Schematic
The Dalles Water Master Plan

CH2MHILL

CHAPTER 3

Water Requirements

Water Use History

This chapter describes the water use history for The Dalles' water system. This history encompasses average and maximum demands, per capita demands, metered consumption, and unaccounted-for water. Documentation of recent water use within The Dalles is essential for projecting future water use. Many of the exhibits referenced in this chapter are located at the end of the text.

Definition of Terms

Demand refers to total water use, the sum of metered consumption (residential, commercial, governmental and industrial), unmetered uses (for example, fire fighting or hydrant flushing), and water lost to leakage, reservoir overflow, and evaporation.

When discussing daily or annual water use, the terms *demand* and *production* are used synonymously in this report. Both refer to all water used in the system, the sum of metered and unmetered use. Demand equals production because both terms refer to all water that is delivered from the water treatment plant and wells to the distribution system.

The terms *demand* and *production* are not synonymous with respect to hourly demands. Water is produced at the WTP and wells at a relatively steady rate throughout the day. Hourly water demands fluctuate in response to water use patterns by residential, commercial, and industrial customers. For example, hourly demands typically exceed the production rate during morning and afternoon/early evening peaks. Hourly demands will be less than the production rate during nighttime hours. Hourly demands will be estimated and used for the distribution system modeling.

Metered use or *consumption* refers to the portion of water use that is recorded by customer meters.

Connection refers to a metered connection to a customer of The Dalles.

Unaccounted-for water refers to the difference between production and consumption. Unaccounted-for water includes unmetered hydrant use, other unmetered uses, and water lost to evaporation, reservoir overflow, and leakage. Meter inaccuracies (both production and customer) also contribute to unaccounted-for water.

Specific *demand* terms include

- *Average day demand (ADD):* total annual production divided by 365 days
- *Maximum day demand (MDD):* the highest system demand that occurs in any single day of a calendar year
- *Maximum monthly demand (MMD):* the highest monthly production during a calendar year

- *Peak hour demand (PHD):* highest hourly demand that is experienced

MDD is an important value for water system planning. The supply facilities (combination of intake, treatment plant, and transmission pipelines plus well facilities) must be capable of meeting the MDD. If the MDD exceeds the combined supply capacity on any given day, storage levels will be reduced. Consecutive days at or near the MDD will result in a water shortage.

The most common units for expressing demands are million gallons per day (mgd). One mgd is equivalent to 695 gallons per minute (gpm) or 1.55 cubic feet per second (cfs). Units of million gallons (MG) are also used.

Meter History

The city's commercial and industrial customers have always been metered, but residential connections were not metered until the mid 1990s. The city began installing residential meters in 1993 and completed the meter installation in 1995. Approximately 3,600 meters were installed in an 18-month period. Billings based on the meter readings were delayed until 1996 to give the customers an opportunity to monitor their consumption and anticipate billings based on their water use.

The addition of meters to the residential connections resulted in a decrease in per capita residential water use. Projections for future use are based on water use since 1996 to account for this significant system change.

Average and Maximum Demands

Exhibits 3-1 and 3-2 summarize ADD records for The Dalles for 1995 through 2003. The values have ranged from 2.7 mgd to 3.6 mgd. The highest value of 3.6 mgd occurred in 1995, just prior to when the city began billing residential customers on the basis of their metered consumption. From 1996, the ADD has trended downward slightly at a rate of 0.02 mgd/year. Possible contributing factors to this downward trend may include a relatively stable service population, public information conservation efforts, and an active leak detection and repair program.

EXHIBIT 3-1
Average Day Demands

Year	Average Day Demand (mgd)	Annual Production (MG)
1995	3.58	1,307
1996	3.09	1,131
1997	2.96	1,082
1998	3.06	1,116
1999	2.88	1,051
2000	2.97	1,087
2001	2.75	1,003
2002	2.90	1,060
2003	3.01	1,097
Minimum	**2.7**	**1,003**
Maximum	**3.6**	**1,307**
Average	**3.0**	**1,104**

Exhibit 3-3 summarizes MDD records for 1995 through 2004. Both the single-day MDD and the 3-day MDD are included. The 3-day MDD represents the demand on the day before, the day of, and the day following the MDD occurrence. It provides an indication of the duration of the peak demands.

Over this 10-year period, the MDD has ranged from a low of 5.7 mgd to a high of 7.8 mgd. The highest value of 7.8 mgd occurred in 2003. The 3-day MDD ranged from 5.4 to 7.2 mgd. The 3-day MDD has typically been 82 to 96 percent of the single-day MDD with an average of 91 percent.

Exhibit 3-4 shows MDD and 3-day MDD values from 1996, the first year of metered billing. A trendline included on Exhibit 3-4 indicates that the MDD is trending upward at the rate of 0.12 mgd per year. According to this trendline, the expected 1-day MDD for year 2004 was 7.05 mgd. The actual recorded MDD for 2004 was 6.66 mgd.

MDDs fluctuate from year to year because they are strongly influenced by weather patterns:

- Maximum temperatures
- The number of consecutive days at high temperatures
- When the high temperatures occur during the summer (early while residents are more consistent in their outdoor irrigation, or later when they are less so)
- Overall rainfall levels during the summer
- Consecutive days without rainfall

Because of the inherent variability of the MDD as a result of these factors, MDD values estimated from the trendline are used as the starting point for projecting future MDDs. The records for The Dalles suggest that the MDD for any particular year may vary from the trendline, either higher or lower, by as much as 1.0 mgd. It is recommended that the city include this allowance in planning for source development. In addition, some cities have chosen to maintain a reserve capacity above this allowance to account for sudden increases in the customer base. This enables a city to accommodate a new customer with a high water demand or a surge in residential development. To illustrate, the city received an inquiry in the summer of 2004 from a potential customer that would be located in the Port area that might use up to 3.0 mgd.

Average Summer and Winter Demands

Monthly demand records from January 1995 to June 2004 are displayed in **Exhibit 3-5.** Outdoor irrigation contributes to higher demands in the summer months. For the period shown, the average winter monthly demand (November through February) was 54 MG, while the average summer monthly demand (June through September) was 138 MG, or 2.5 times the average for the winter months.

Exhibit 3-6 shows the MMD from 1995 to 2003. The MMD has occurred in July in nine of the ten years of record and in August one year. The MMDs have ranged from 151 to 194 MG. The average for the period was 166 MG.

EXHIBIT 3-3
Maximum Day Demand Records (production records)

Date	Year	Mill Creek Production (mgd)	High Line Production (mgd)	Lone Pine Well Production (mgd)	Jordan Well Production (mgd)	Marks Well Production (mgd)	Maximum Day Demand (mgd)	3-Day Maximum (mgd)1	3-d/1-d MDD, as %	Year
29-Jun	1995	1.93	2.49	1.05	1.71	0.45	7.63	7.21	95%	1995
27-Jul	1996	1.88	3.16		1.25		6.29	5.56	88%	1996
07-Aug	1997	1.21	2.59		1.88		5.68	5.44	96%	1997
28-Jul	1998	1.94	2.92		2.16		7.02	5.93	84%	1998
11-Jul	1999	1.61	2.83		1.56		6.00	5.67	94%	1999
09-Aug	2000	1.53	3.11		1.40		6.04	5.74	95%	2000
11-Jul	2001	0.70	2.73		2.08	1.21	6.72	6.21	92%	2001
11-Jul	2002	1.80	2.83			1.68	6.31	5.80	92%	2002
28-Jul	2003	0.61	2.98		3.02	1.19	7.80	6.43	82%	2003
26-Jul	2004	0.35	2.70		2.66	0.95	6.66	5.74	86%	2004
Minimum		**0.4**	**2.5**				**5.7**	**5.4**	**82%**	
Maximum		**1.9**	**3.2**				**7.8**	**7.2**	**96%**	
Average		**1.4**	**2.8**				**6.6**	**6.0**	**91%**	

Notes:
1. 3-day maximum includes day before, day of, and day after maximum day, divided by 3
2. Maximum day demand column equals the combined production from the Wicks WTP plus the three wells.

EXHIBIT 3-6
Maximum Monthly Demands

Year	Month of Occurrence	Maximum Monthly Demand (MG)	Maximum Monthly Demand (mgd)
1995	Jul	178	5.7
1996	Jul	162	5.2
1997	Jul	153	4.9
1998	Jul	171	5.5
1999	Jul	159	5.1
2000	Jul	164	5.3
2001	Jul	151	4.9
2002	Aug	160	5.2
2003	Jul	194	6.3
	Minimum	**151**	**4.9**
	Maximum	**194**	**6.3**
	Average	**166**	**5.3**

Exhibit 3-7 shows the average of the monthly demand divided by annual demand for the years 1995-2003. On average, the 4-month summer period, June through September, accounted for 51 percent of total annual demand.

Peaking Factors

Peaking factors, the ratios of MDD:ADD, MMD:ADD, and MMD:MDD are useful for hydraulic modeling of the system and for demand forecasting. **Exhibit 3-8** summarizes the peaking factors for 1995-2003. The MDD:ADD has averaged 2.2, the three-day MDD:ADD has averaged 2.0, the MMD:ADD has averaged 1.8, and the MMD:MDD has averaged 0.8.

EXHIBIT 3-8
Peaking Factors
(For Maximum Day, 3-day Maximum Day, and Maximum Monthly Demands)

		1-day MDD		3-day MDD		MMD		
Year	ADD	Value	Ratio MDD:ADD	Value	Ratio 3-d MDD:ADD	Value	Ratio MMD:ADD	Ratio MMD:MDD
1995	3.6	7.6	2.1	7.2	2.0	5.7	1.6	0.8
1996	3.1	6.3	2.0	5.6	1.8	5.2	1.7	0.8
1997	3.0	5.7	1.9	5.4	1.8	4.9	1.7	0.9
1998	3.1	7.0	2.3	5.9	1.9	5.5	1.8	0.8
1999	2.9	6.0	2.1	5.7	2.0	5.1	1.8	0.9
2000	3.0	6.0	2.0	5.7	1.9	5.3	1.8	0.9
2001	2.7	6.7	2.4	6.2	2.3	4.9	1.8	0.7
2002	2.9	6.3	2.2	5.8	2.0	5.2	1.8	0.8
2003	3.0	7.8	2.6	6.4	2.1	6.3	2.1	0.8
Minimum			**1.9**		**1.8**		**1.6**	**0.7**
Maximum			**2.6**		**2.3**		**2.1**	**0.9**
Average			**2.2**		**2.0**		**1.8**	**0.8**

Note: The peak hour peaking factor, PHD:ADD, assumes 1.5 x MDD:ADD, or **3.3**.

The MDD:ADD peaking factors for 1995-2003 are displayed in **Exhibit 3-9**. The trend has been upward, reflecting the increasing trend in MDD and the slight decreasing trend in ADD during this period.

Per Capita Demands

The Dalles' water service population was estimated by applying typical unit household size values (from local planning agencies) to the number of residential connections. The average size for single family residences in The Dalles is 2.4 people per household and the average size for multiple family residences is 2.1 people per household. The service area population was estimated by multiplying the number of single and multiple family connections by the appropriate factors and summing to yield the following service area populations:

- Calendar year 2002: 10,710
- Calendar year 2003: 10,768

To determine ADD per capita water use values, recorded ADDs (2.9 mgd in 2002 and 3.0 mgd in 2003) were divided by the service area population for each year. MDD per capita water use values were calculated in a similar manner. However, to minimize the effect of MDD variability, trendline MDD values (6.8 mgd in 2002 and 6.9 mgd in 2003) were used instead of actual values. The per capita water use values for 2002 and 2003 were as follows:

- ADD: 271 gallons per capita per day (gpcd) in 2002, and 280 gpcd in 2003
- MDD: 636 gpcd in 2002, and 643 gpcd in 2003

Future ADDs were projected using the average of the ADD per capita values for 2002 and 2003, or 275 gpcd. Likewise, future MDDs were projected using the average MDD per capita estimate for these two years of 640 gpcd.

These per capita values represent the total system demand divided by the service population. Therefore, they include commercial, industrial, and governmental demands as well as residential demands.

Production by Source

Exhibit 3-10 summarizes maximum production values for the Wicks WTP, including the delivery values through the Mill Creek and High Line transmission pipelines. The highest recorded production from the plant was 5.20 mgd in 1995, followed by values of 5.07 mgd in 1998 and 5.04 mgd in 1996. The peak production averaged 4.8 mgd for the period of 1995-2003.

EXHIBIT 3-10
Maximum Day Wicks WTP Production Records

Date	Year	Mill Creek Production (mgd)	Mill Creek Production (gpm)	High Line Production (mgd)	High Line Production (gpm)	Combined Production (mgd)	3-Day Maximum (mgd)*
25-Jun	1995	2.40	1,670	2.80	1,940	5.20	4.91
27-Jul	1996	1.88	1,300	3.16	2,200	5.04	4.91
14-May	1997	2.15	1,490	2.23	1,550	4.37	4.08
30-Jun	1998	2.58	1,790	2.49	1,730	5.07	5.00
29-Jul	1999	1.44	1,000	3.23	2,240	4.67	4.54
30-Jun	2000	1.68	1,170	3.23	2,240	4.91	4.79
1-Jun	2001	1.44	1,000	3.14	2,180	4.58	4.46
11-Jul	2002	1.80	1,250	2.83	1,960	4.63	4.56
28-May	2003	2.34	1,630	2.32	1,610	4.66	4.30
5-Jun	2004	2.23	1,550	2.39	1,660	4.62	4.35
Minimum		**1.4**	**1,000**	**2.2**	**1,550**	**4.4**	**4.1**
Maximum		**2.6**	**1,790**	**3.2**	**2,240**	**5.2**	**5.0**
Average		**2.0**	**1,390**	**2.8**	**1,930**	**4.8**	**4.6**

* 3-day maximum includes day before, day of, and day after maximum day, divided by 3

The delivery through the High Line on these peak days averaged 2.8 mgd (1,930 gpm) while the delivery through the Mill Creek Line averaged 2.0 mgd (1,390 gpm).

The production of the Wicks WTP compared to production from the wells (combined production from the Lone Pine, Jordan, and Marks wells) is shown in **Exhibit 3-11.** The wells have produced as much as 70 MG in a month (in 2003), which is about 2.3 mgd.

For the period of January 1995 through June 2004, the Wicks WTP has supplied 93.7 percent of the total production, with the wells contributing the remaining 6.3 percent. These percentages have remained relatively consistent throughout the 10-year period.

Consumption

Exhibits 3-12 and 3-13 display monthly consumption (metered use) patterns for the city. During the past three fiscal years (FY 01-02, FY 02-03, and FY 03-04), consumption averaged approximately 40 MG a month for November through February, and approximately 124 MG per month during June through August. **Exhibit 3-13** shows monthly consumption as a percentage of annual consumption.

Exhibit 3-14 displays consumption by the major customer categories: residential, industrial, commercial and governmental. The single industrial customer during these years has been Kerr McGee Corporation. The governmental category includes facilities owned by the city, county, Port and college. The city facilities include the wastewater treatment plant, sewer lift stations, and irrigation of city properties. During these three fiscal years, residential use has averaged 55 percent of the total use. Commercial use has averaged 40 percent of the total. The governmental use has averaged 5 percent.

Residential Consumption

Exhibit 3-15 displays the average residential use by month over the 3-year period from July 2001 through June 2004. While the average residential consumption during the month of July for this period was 81 MG, residential consumption reached a high of approximately 95 MG in July 2003. The average monthly residential consumption for the months of November through February was approximately 20 MG.

Exhibit 3-16 displays per capita residential consumption for calendar years 2002 and 2003. Residential per capita values are lower than the total system per capita values calculated previously, because only the residential consumption was included in the calculation. Metered residential use was summed and divided by the service population. During the period from January 2002 through December 2003, the average per capita residential consumption was 127 gpcd. The minimum monthly value was 57 gpcd. The maximum monthly value was 282 gpcd.

Unaccounted-for Water

A comparison of the demand data with the consumption data provides a value for the unaccounted-for water, which is the difference between production and metered use. The percentage of unaccounted-for water equals the production minus the metered use, divided by the production. The causes of unaccounted-for water include meter inaccuracies, evaporation, reservoir overflows, unmetered hydrant use, and leakage.

Exhibit 3-17 illustrates the unaccounted-for water percentage for January 2002 through December 2003. The value has ranged from –9 percent to 31 percent, with an average of 15 percent. The negative values occur because the meter readings are not aligned exactly with the first and last days of each month. (The consumption values were shifted back by one month to align more closely with the timing for the production values, but this did not align the two sets of figures exactly.)

The average of 15 percent exceeds the WRD's municipal goal of 10 percent or less for unaccounted-for water. A value of 15 percent means that on an annual basis, the city is not accounting for an average of 440,000 gallons per day.

Water Use Projections

The per capita approach was used for projecting demands within The Dalles' water system. Recent per capita ADD and MDD values (ADD: 275 gpcd; MDD: 640 gpcd) were applied to population projections for The Dalles to estimate future demands. The rate of population growth was obtained from local planning agencies.

The following planning criteria and assumptions were used in developing the demand projections:

1. Service population growth rate equals 1.1 percent annually. This rate was the estimated rate through year 2010 in the Wasco County Comprehensive Plan Amendment completed in 1994.

2. The High Density Residential zone will be developed with half single family dwellings and half multiple family dwellings.
3. Multiple family dwellings will build out at a density of 15 units per acre, with a household size of 2.1 people per household.
4. Single family dwellings will build out at a density of 4 units per acre, with a household size of 2.4 people per household.
5. Residential mobile homes (RMHs) in the RMH zones will build out at 6 units per acre, with a household size of 2.1 people per household.
6. Low Density Residential zoning will build out at a household size of 2.25 people per household. This assumes that 50 percent will be single family dwellings at 2.4 people per household and 50 percent will be multi-family dwellings at 2.1 people per household.
7. RMHs in the Low Density Residential areas will build out at single family dwelling densities.
8. Densities and household sizes for multi-family, single family, and RMH developments were based on the city's Comprehensive Plan update background document titled *Task 4 Growth Forecasts and Land Use Requirements* (Spencer and Kupper, Portland, Oregon, November 1992).
9. A developmentally constrained lands map, originally developed to support the 1992-1993 *Comprehensive Plan Update*, was provided by City Community Development.
10. Vacant land areas were based on lands determined to be vacant and not developmentally constrained in the city's buildable lands inventory done as part of the 1992-1993 *Comprehensive Plan Update*. This map was updated using 2003 aerial photo and fall 2004 assessor data to eliminate areas developed since 1992.
11. Data layers used to calculate developable acres by land use category in each pressure zone include a) pressure zone overlay (produced by CH2M HILL), b) City of The Dalles Zoning Map (County GIS), c) 1992/1993 Buildable Lands Map (City of The Dalles Comprehensive Plan Map input into County GIS), d) 2003 Aerial Photo (County GIS), e) 2004 Assessor Data (County GIS).
12. Zoning was aggregated into the following broader land use categories: a) All commercial zones were aggregated into commercial land use, b) all industrial zones were aggregated into industrial land use, c) Residential High Density (RH), d) Residential Low Density (RL), e) RMH. Other zones did not comprise a significant land area or represent significant water demands so were not classified for projection purposes.

The estimated population growth rate is one of the most critical factors for projecting future water demands. The 1.1 percent annual rate, derived from previous planning studies for the city, was checked against available census information to determine its current validity. The City of The Dalles (entire city, not just water system service area) 1990 census population was 11,050. The year 2000 census population was 12,156. This represents a 1.0 percent annual growth rate, or just slightly less than the projected rate of 1.1 percent.

The city should regularly check the actual rate of growth compared to the projected rate of growth—for both population and water demands.

Developable Lands and Buildout Population

Exhibit 3-18 summarizes land use within the city's water system service area by pressure zone and by zoning category. It lists land area that is already developed and land that is yet to be developed. Lands that are not feasible for development, because of slope or other factors, are not considered to be developable lands. By applying the projected population densities to the developable land areas, the Exhibit 3-18 generates a buildout population growth for each pressure zone. The total population increase from 2003 to buildout is estimated to equal 3,645. This will equal a total service area population at buildout of 14,413.

The developed lands in Exhibit 3-18 provided the basis for estimating current (year 2004) demands within each pressure zone for hydraulic modeling.

Projected Water Demands

Maximum Day Demand Allowances

The MDD fluctuates from year to year, primarily related to summer weather conditions. The MDD has exceeded the trendline by as much as 0.9 mgd (year 2003). For planning purposes, an allowance of 1 mgd above the projected trendline is added to account for such variations.

A second allowance is added to the MDD to account for the potential of a new industrial user locating in The Dalles. The developable lands inventory, included in **Exhibit 3-18,** provides an estimate of residential population potential. As shown, the city also has developable industrial and commercial lands. Demand projections based on population growth do not directly account for future industrial use. Per capita projections assume that the ratio of commercial and industrial use to residential use remains constant. This ratio may change if a single industrial customer requiring large quantities of water locates within The Dalles' service area.

A 3 mgd allowance is shown to account for this potential future demand. During the fall of 2004, when this master plan was being prepared, the city had received an inquiry from an industry that was considering locating in The Dalles. This one customer estimated its water needs as 3 mgd. Therefore, city staff believed that it was appropriate to include a 3 mgd industrial allowance.

Projection Criteria

Exhibit 3-19 summarizes the criteria used for projecting demands. The service population, trendline-ADD, and trendline-MDD for 2003 were used as the baseline for projecting future demands.

As discussed above, the trendlines used for determining the starting values of ADD and MDD (for year 2003) were based on the city's demand history for 1996 through 2003. The year 1996 was the first year that customers were billed based on their metered use, and the second year that metered use was tracked and reported to them.

EXHIBIT 3-19
Demand Projection Criteria for The Dalles Water System Service Area

Criterion	Value
2003 service population	10,768
Buildout population	14,413
2003 trendline-ADD	3.0 mgd
2003 trendline-MDD	6.6 mgd
Per capita ADD	275 gpcd
Per capita MDD	640 gpcd
Rate of population growth	1.10 percent
MDD weather allowance	1.0 mgd
MDD industrial allowance	3.0 mgd

System-Wide Projections

Exhibit 3-20 and 21 provide trendline demand projections through 2025. Three-day MDDs were projected at 91 percent of MDD, based on historical data from 1995 to 2003. In addition, Exhibit 3-20 provides estimates of demands at buildout. If the annual growth rate remains constant at 1.1 percent, the buildout population will be reached in the year 2030.

Exhibit 3-22 shows projected monthly demands for the years 2010, 2025, and buildout. These projections were obtained by multiplying average monthly production as a percentage of annual production for the period 1995 to 2003 (shown in Exhibit 3-7), by the ADD projections shown in Exhibits 3-20 and 3-21. Monthly projections indicate that the maximum average day demand by month at buildout will be approximately 7.0 mgd, or 220 MG in the month of July.

EXHIBIT 3-2. Average Day Demand

EXHIBIT 3-4. Maximum Day Demands

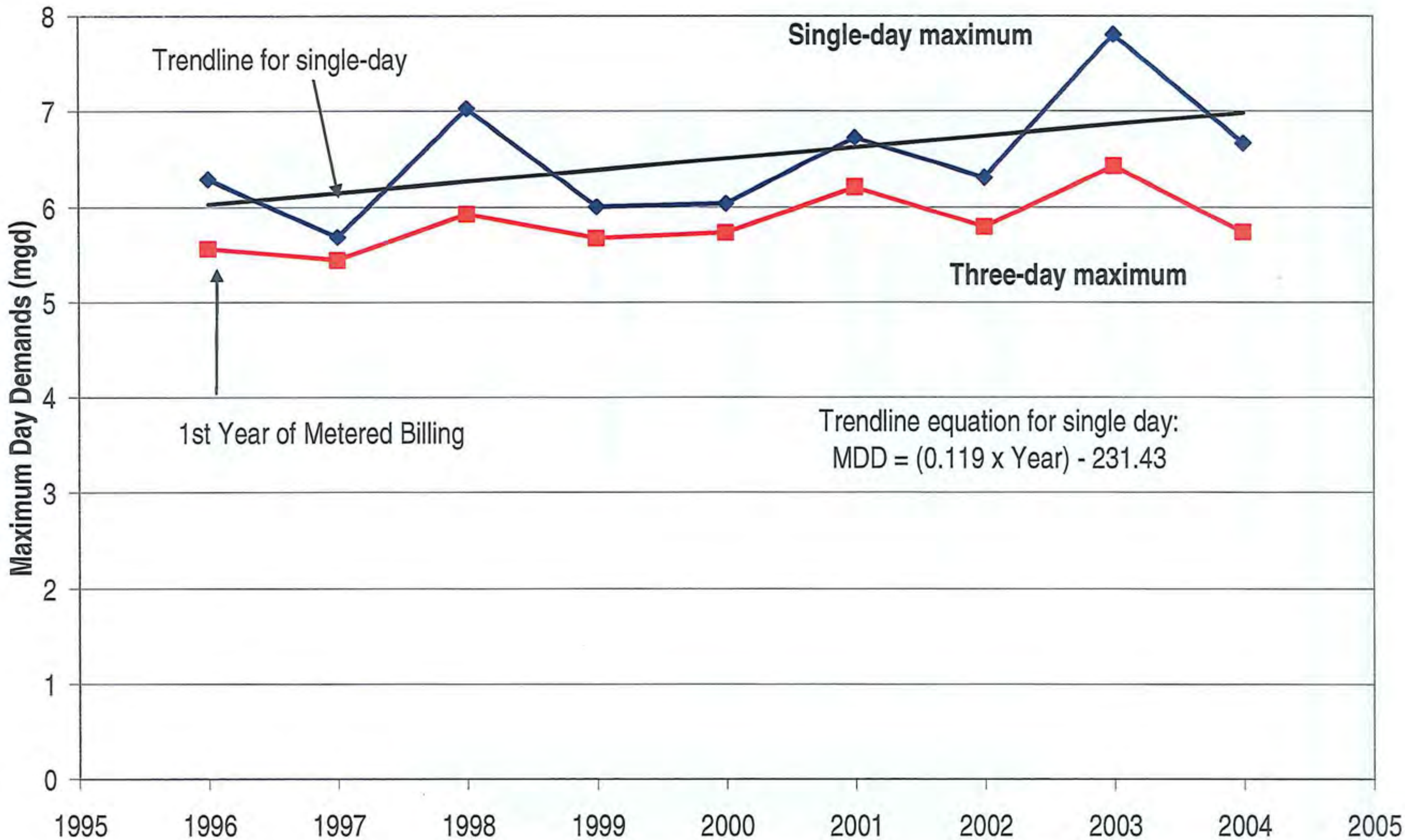


EXHIBIT 3-5. Monthly Demand Records
(Total includes production from both Wicks WTP and Wells)

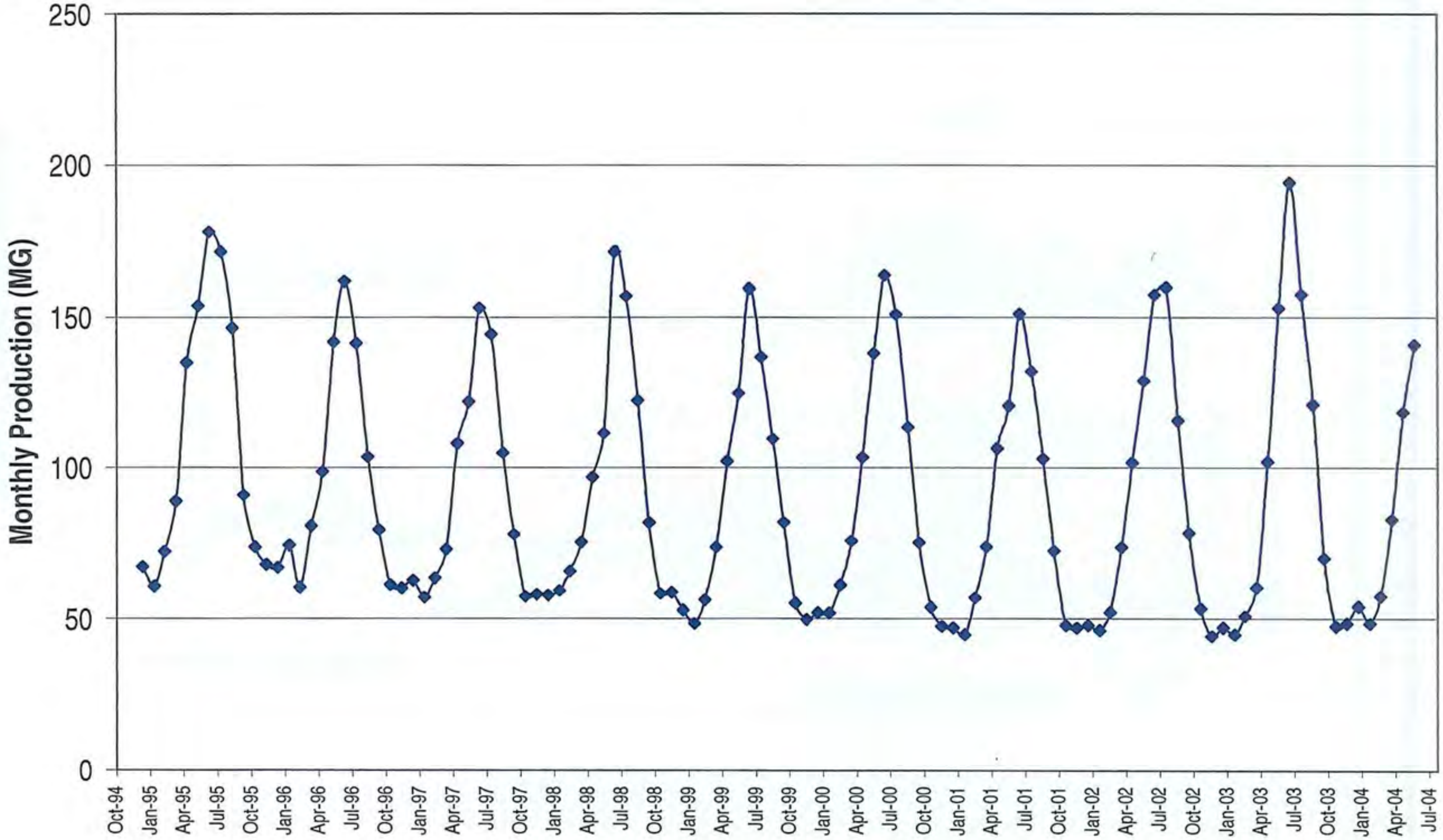


EXHIBIT 3-7. Monthly Production as Percentage of Annual Production
(Averages for years 1995-2003)

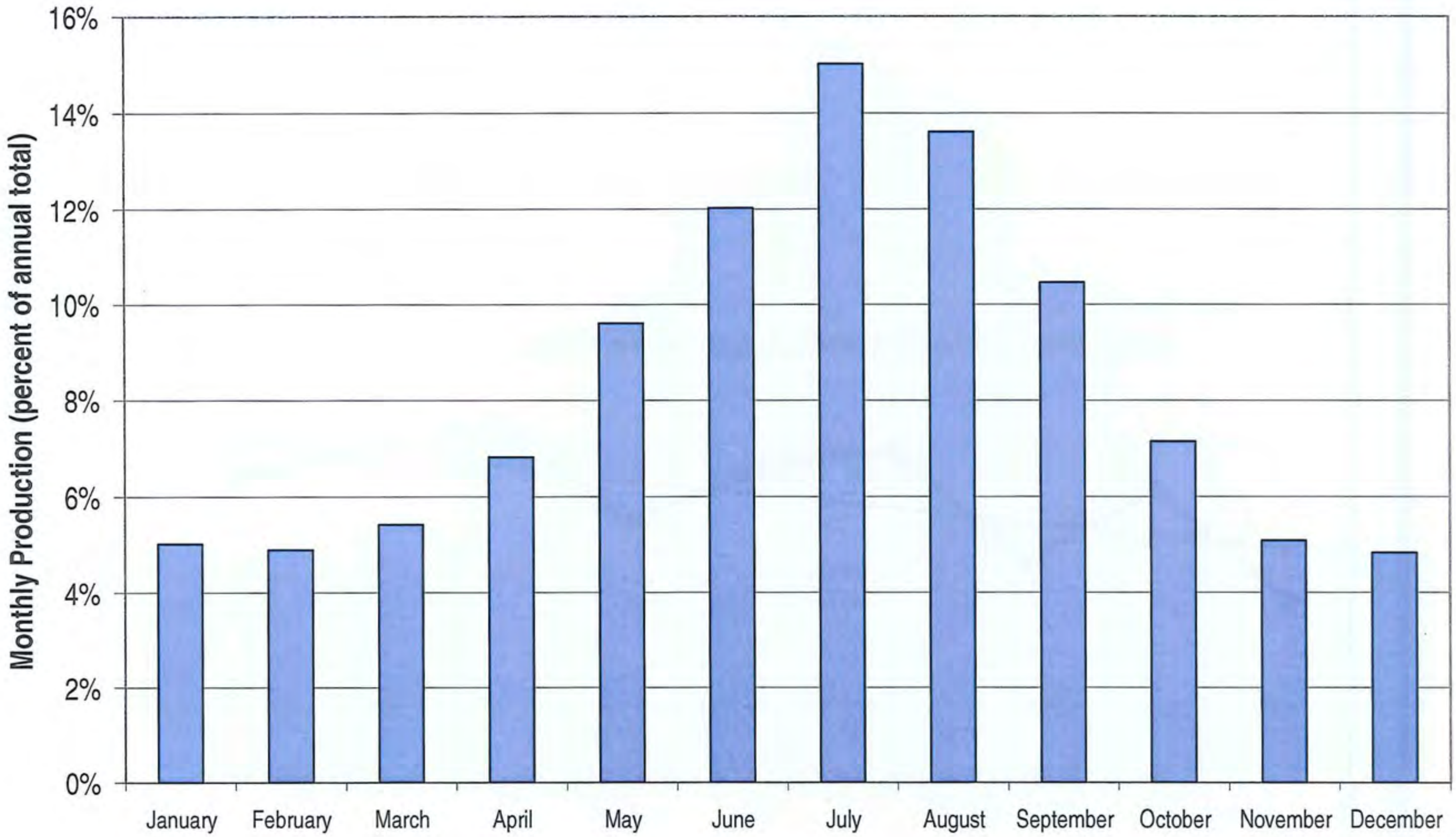


EXHIBIT 3-9. Maximum Day Demand to Average Day Demand Peaking Factors

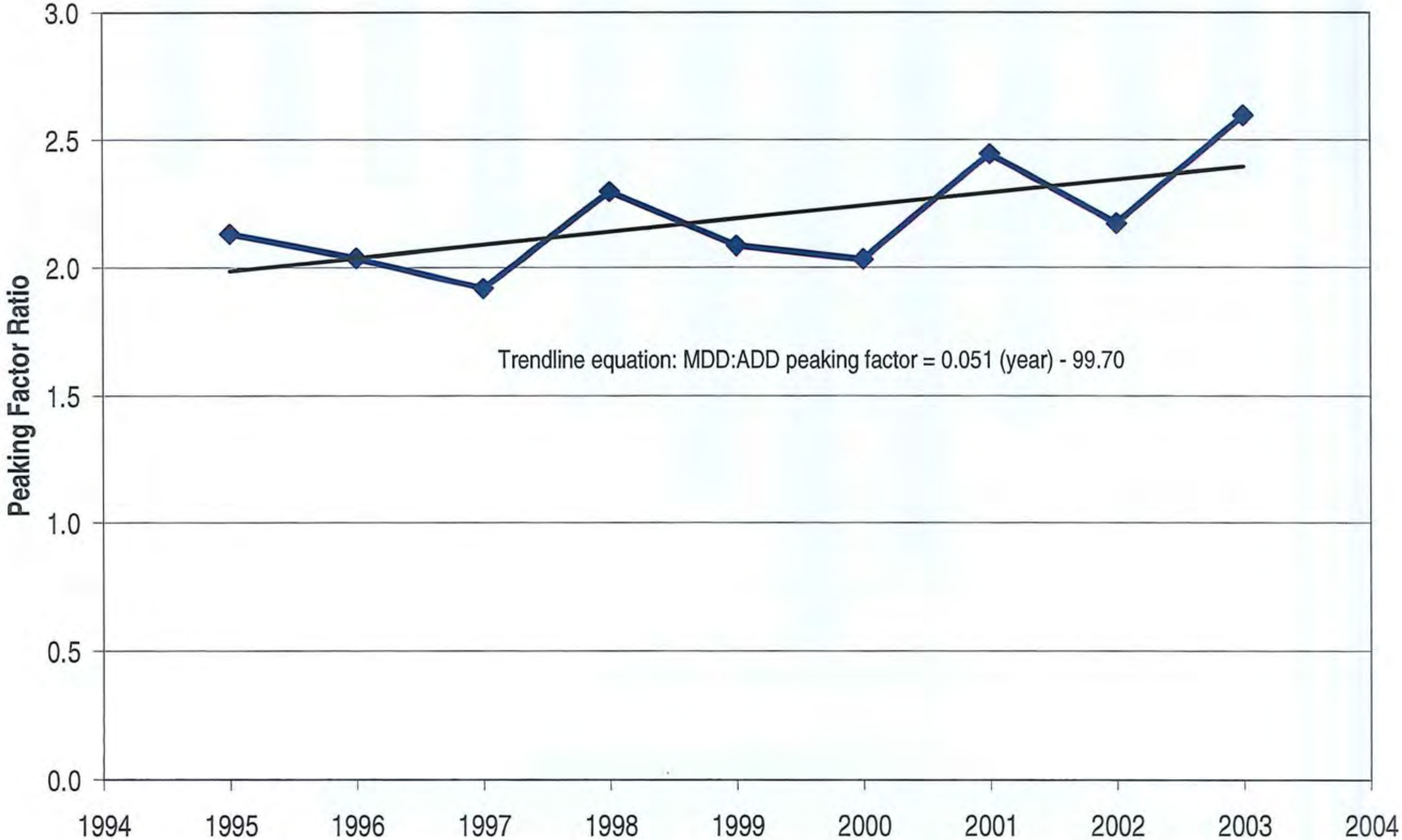


EXHIBIT 3-11. Monthly Production for WTP and Wells

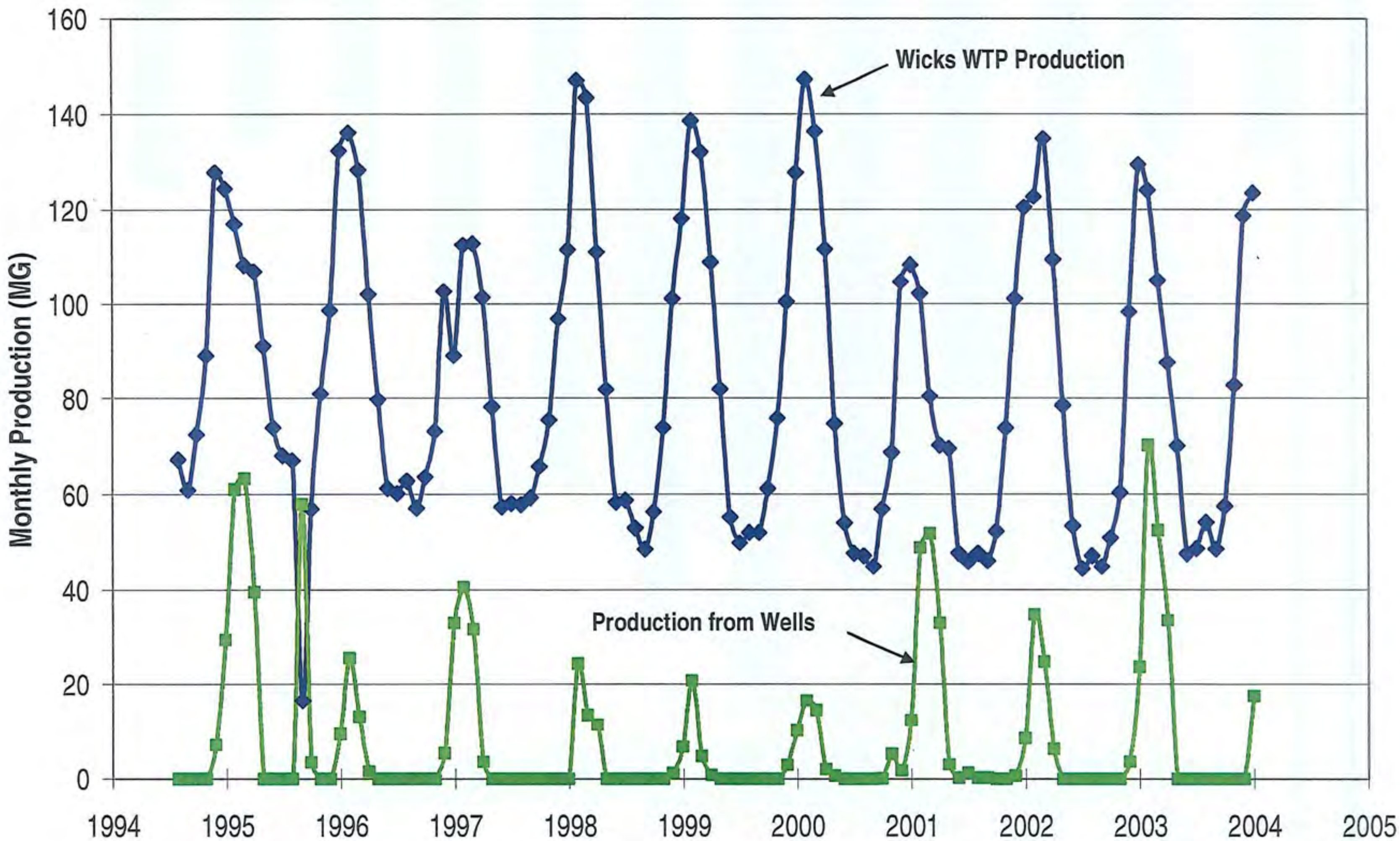


EXHIBIT 3-12. Average Monthly Consumption for FY 01-02 to FY 03-04
(Total of all customer categories)

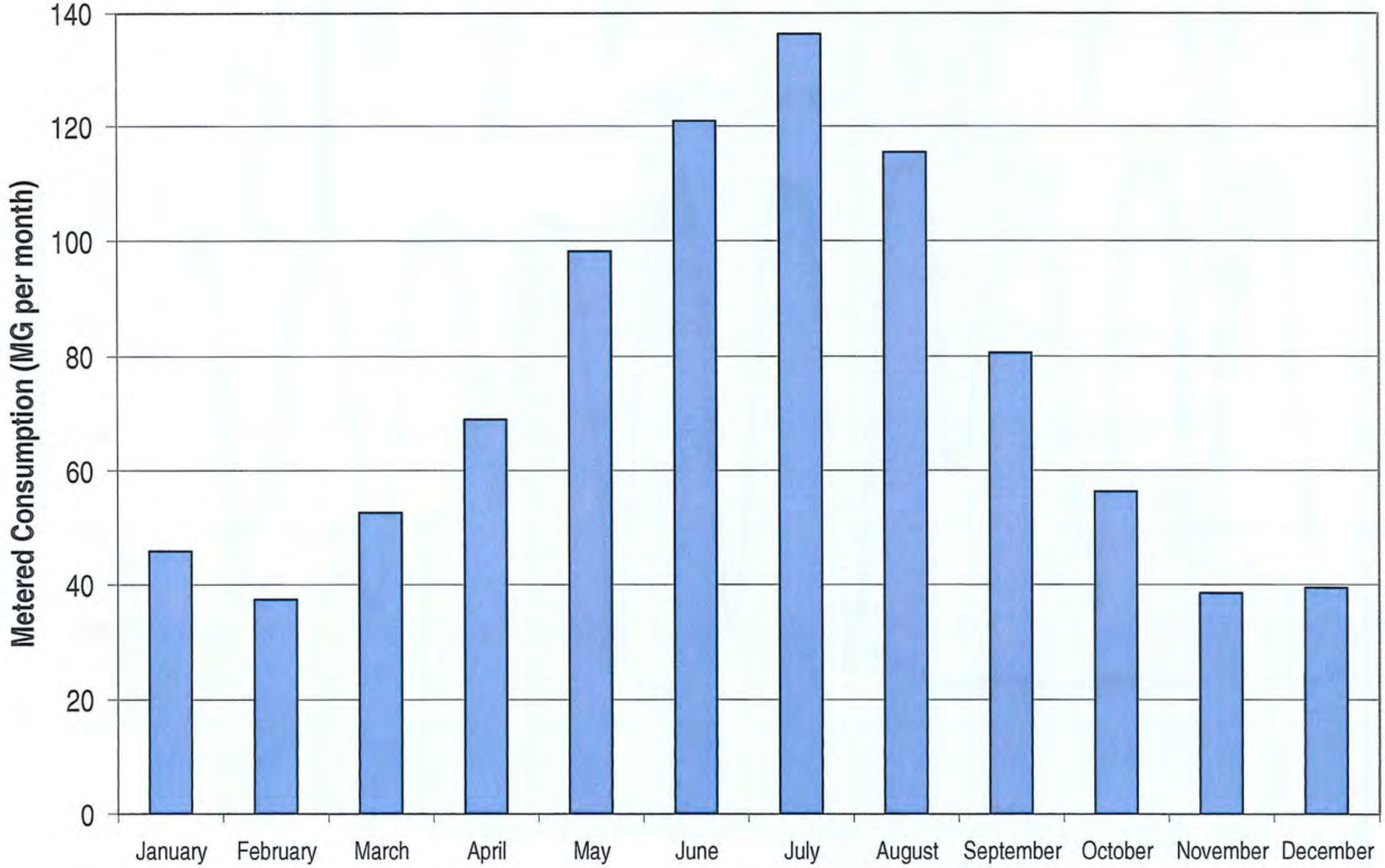


EXHIBIT 3-13. Average Monthly Consumption for FY 01-02, FY 02-03, and FY 03-04
(Total of all customer categories)

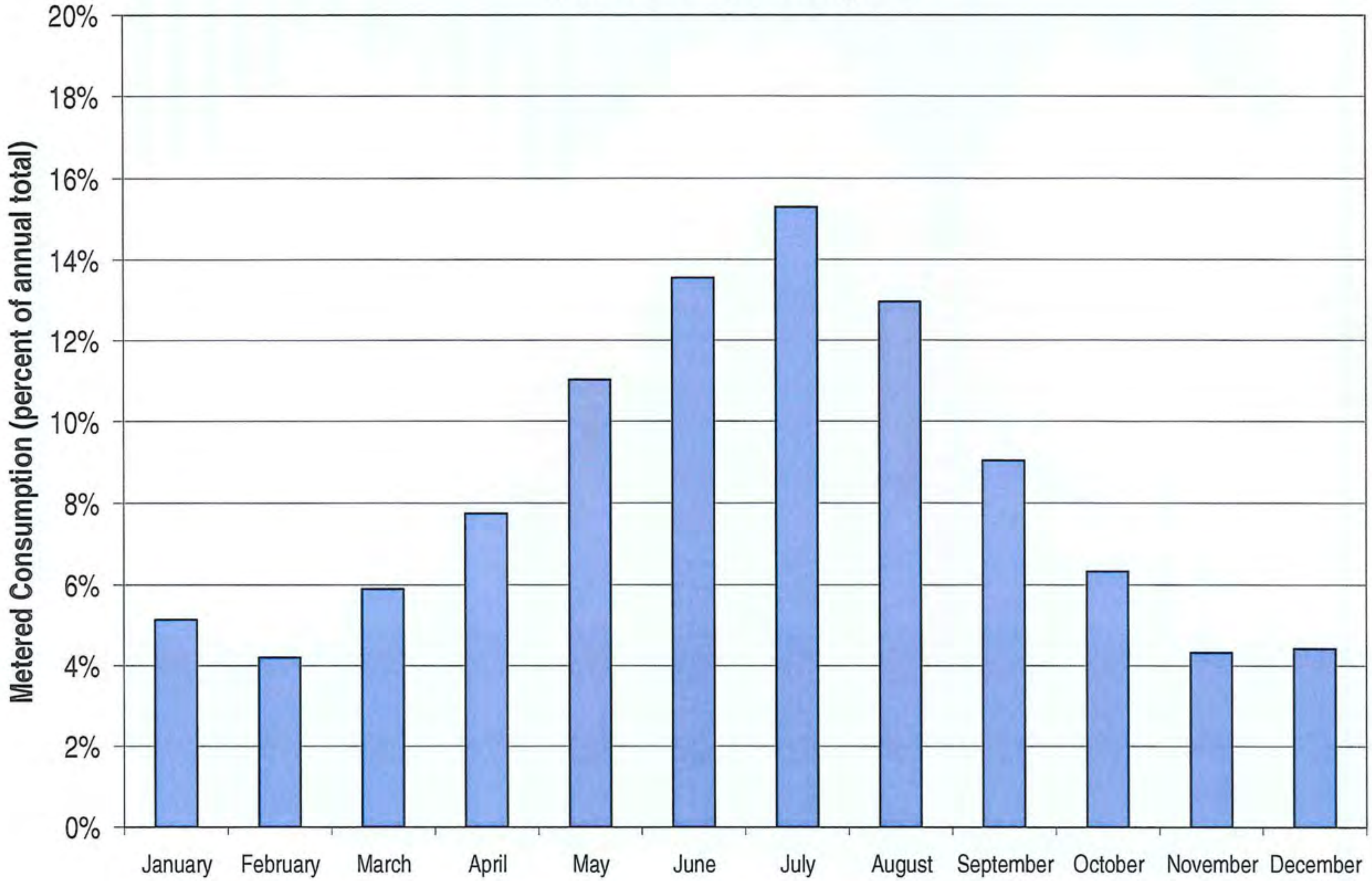


EXHIBIT 3-14. Monthly Consumption for FY 01-02 Through FY 03-04

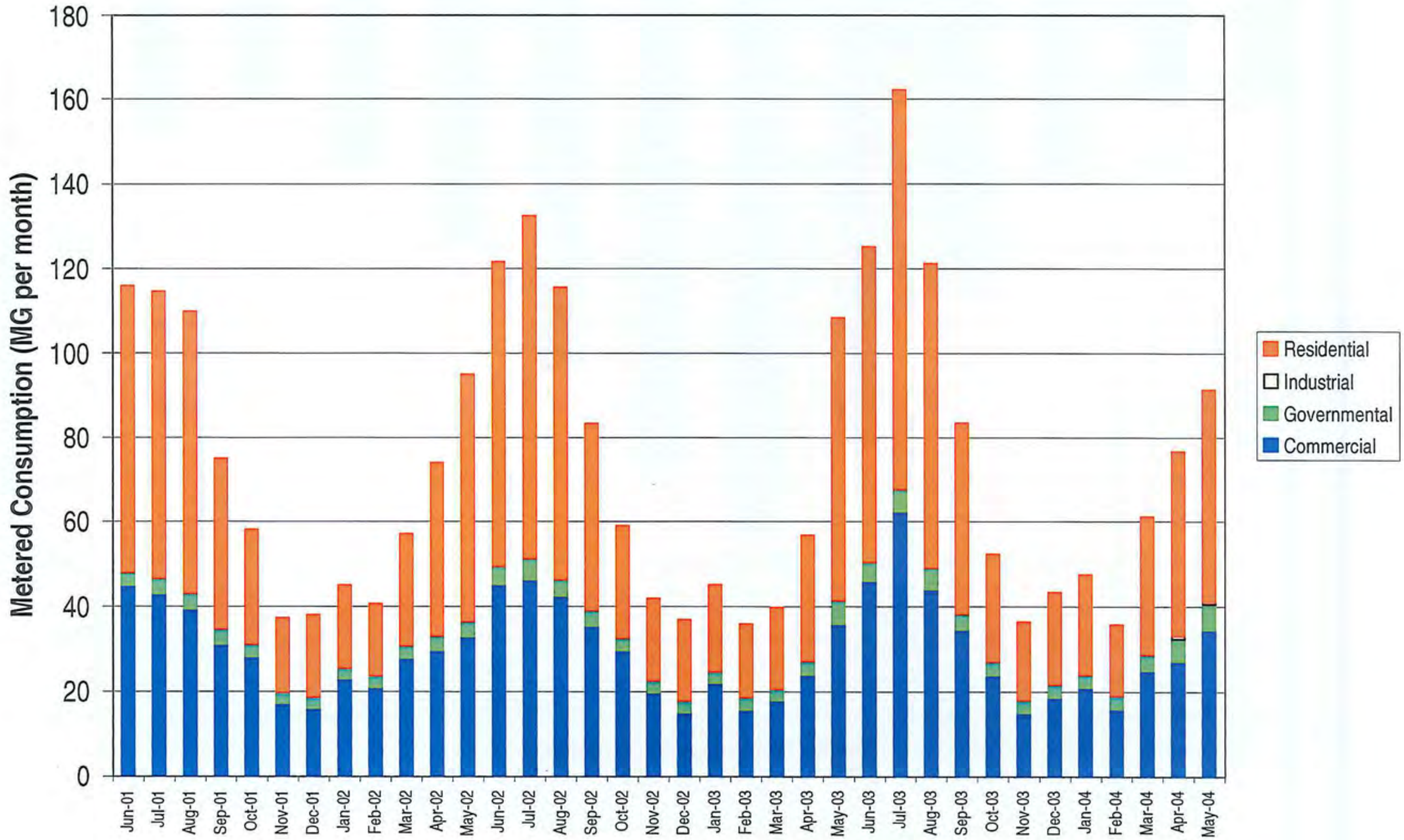


EXHIBIT 3-15. Average Residential Monthly Consumption for FY 01-02 to FY 03-04

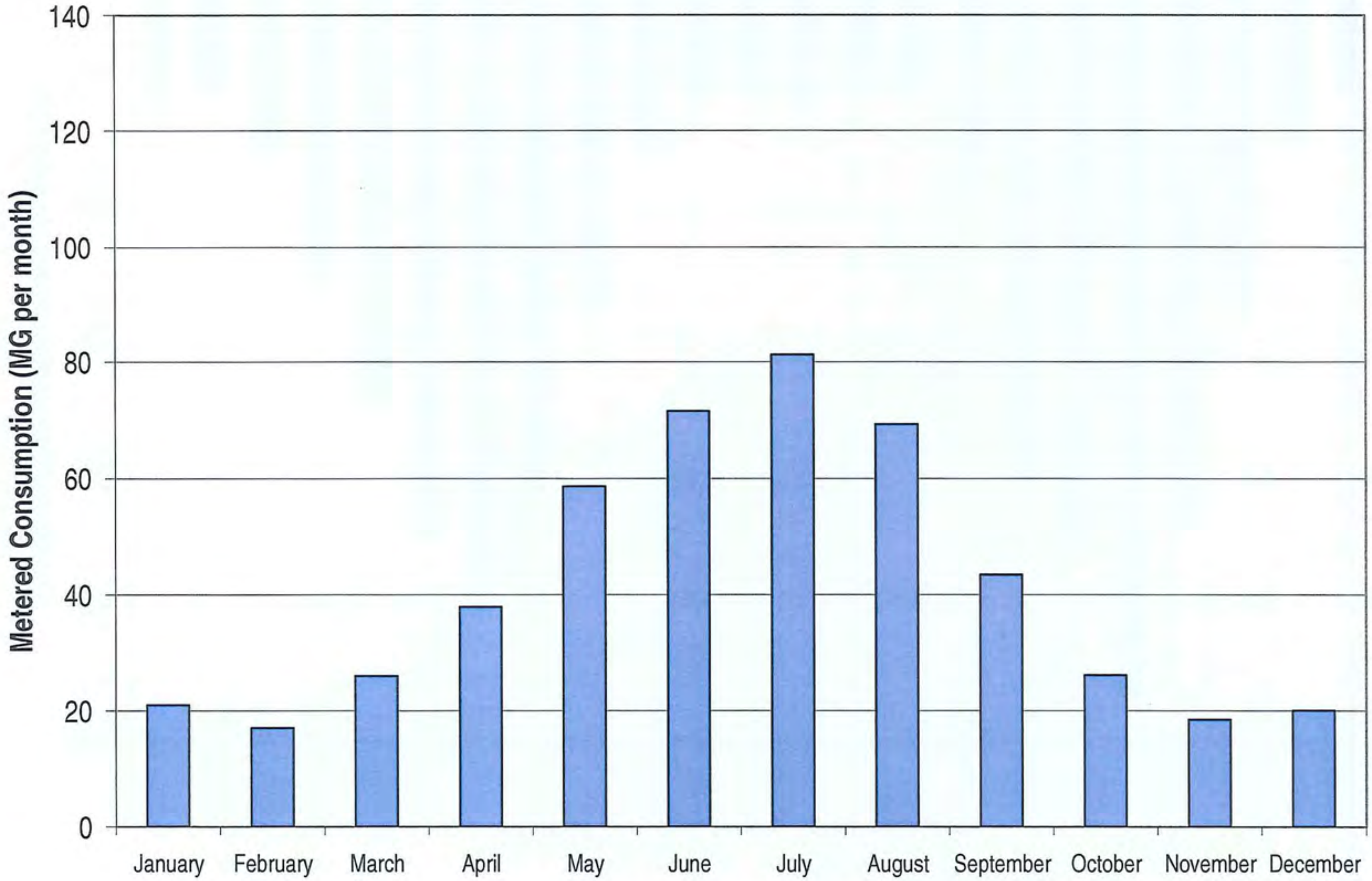


EXHIBIT 3-16. Per Capita Residential Use

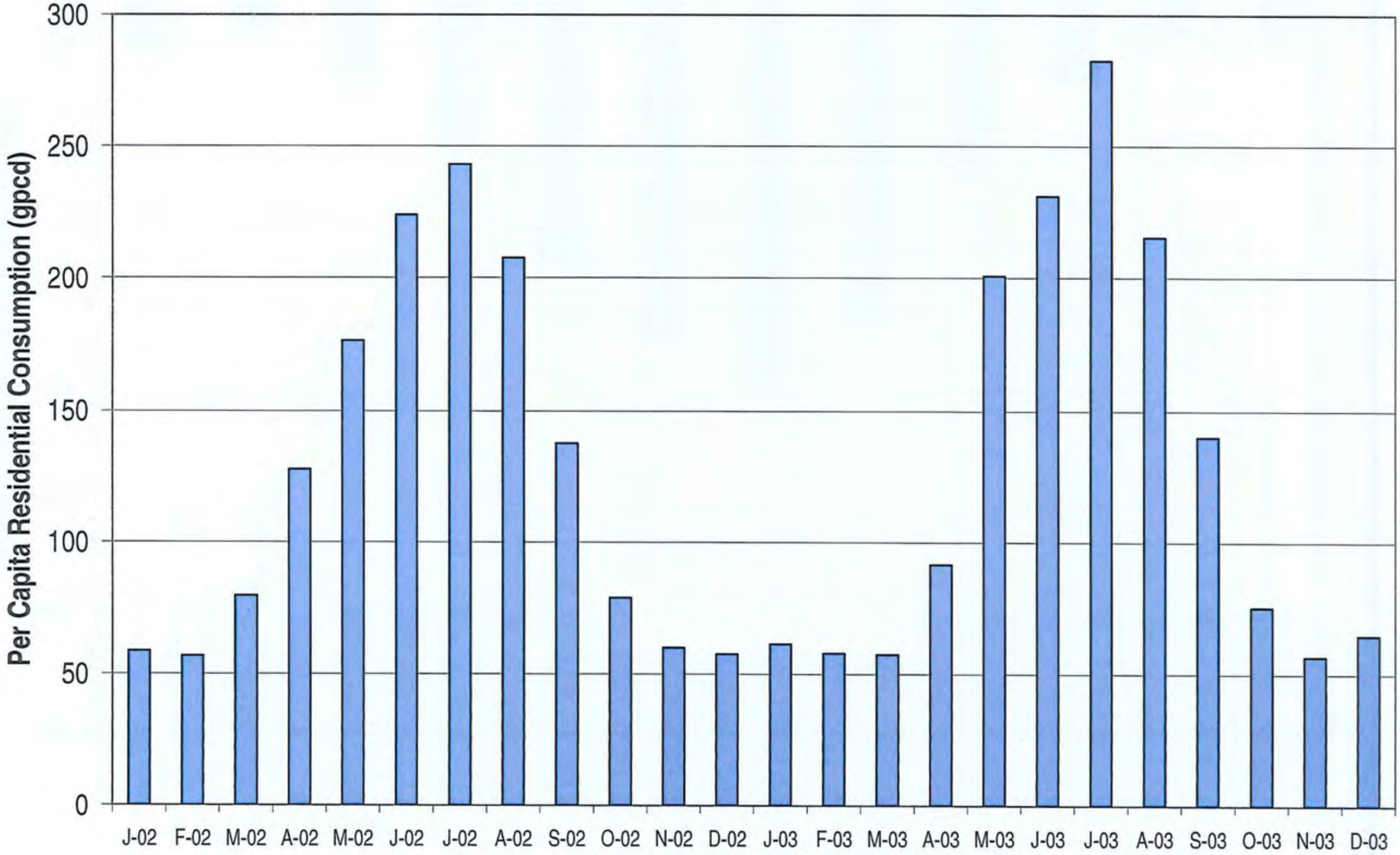


EXHIBIT 3-17. Unaccounted-for Water

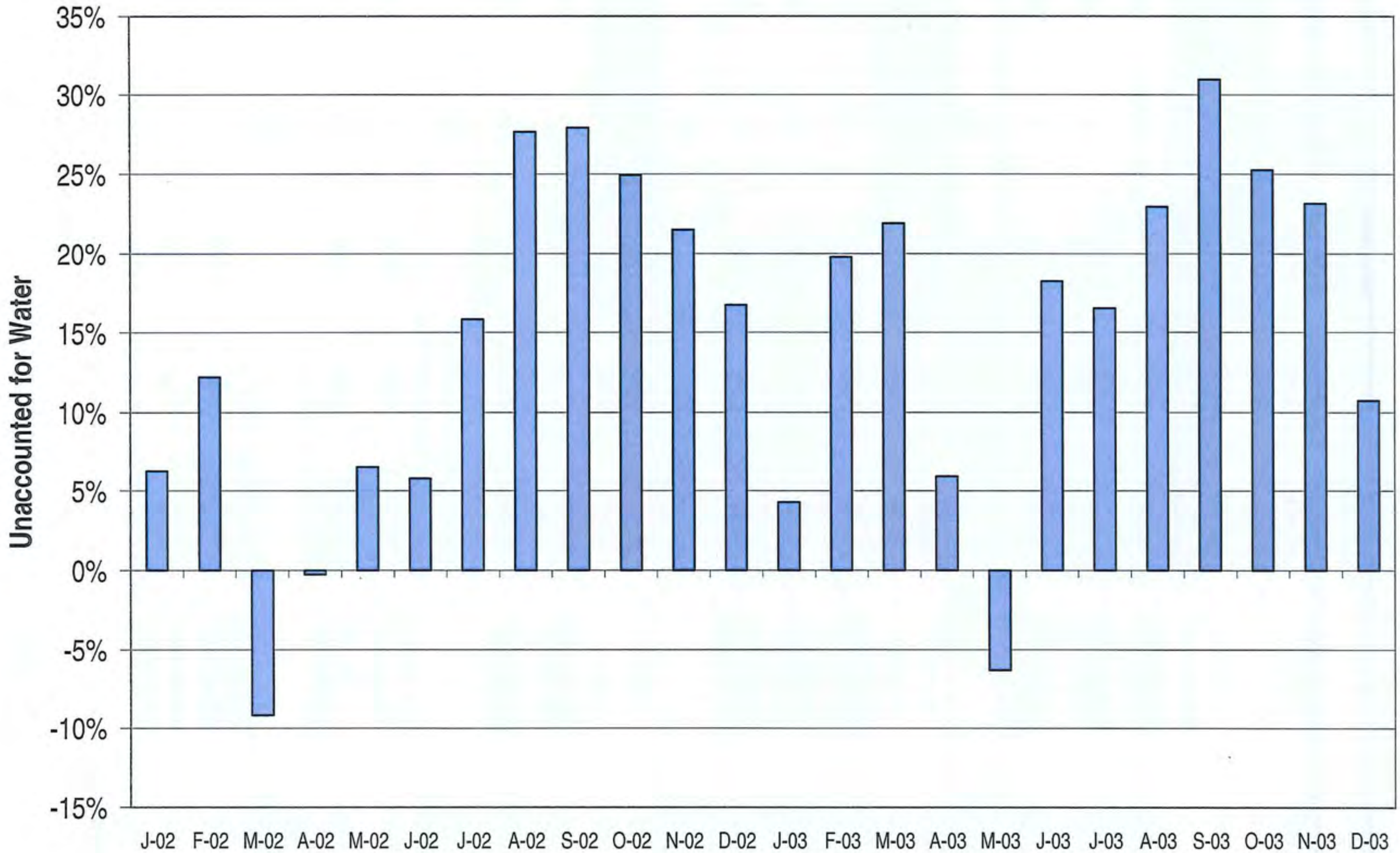


EXHIBIT 3-18
Buildout Projections by Pressure Zone

Pressure Zone	Land Use Category	Developed Acres	Developable Acres	Projected Density (units per acre)	Additional Households at Buildout	Household Size	Service Population Addition at Buildout
310	Commercial	214.48	26.22				
310	Industrial	188.50	194.85				
310	Government						
310	High Density Residential	88.02	7.77	10.00	78	2.25	175
310	Low Density Residential	15.69	0.20	4.00	1	2.40	2
310	Residential Mobile Home	13.69	0.25	6.00	1	2.40	4
310 Subtotal					**80**		**181**
352	Commercial	26.42	0.00				
352	Industrial	18.16	28.73				
352	High Density Residential	0.18	1.55	10.00	15	2.25	35
352	Low Density Residential	1.13	1.52	4.00	6	2.40	15
352	Residential Mobile Home	10.80	0.00	6.00	0	2.40	0
352 Subtotal					**21**		**50**
395	Commercial	4.07	0.00				
395	Industrial	33.65	0.00				
395	High Density Residential	191.67	0.02	10.00	0	2.25	0
395	Low Density Residential	11.81	2.52	4.00	10	2.40	24
395 Subtotal					**10**		**24**
460	High Density Residential	20.70	0.00	10.00	0	2.25	0
460	Low Density Residential	72.13	9.95	4.00	40	2.40	95
460 Subtotal					**40**		**95**
475	High Density Residential	66.71	0.00	10.00	0	2.25	0
475	Low Density Residential	69.63	1.44	4.00	6	2.40	14
475 Subtotal					**6**		**14**
507	Commercial	0.32	0.00				
507	High Density Residential	80.17	1.88	10.00	19	2.25	42

EXHIBIT 3-18
Buildout Projections by Pressure Zone

Pressure Zone	Land Use Category	Developed Acres	Developable Acres	Projected Density (units per acre)	Additional Households at Buildout	Household Size	Service Population Addition at Buildout
507	Low Density Residential	7.72	0.00	4.00	0	2.40	0
507 Subtotal					**19**		**42**
513	Commercial	2.59	0.00				
513	High Density Residential	5.40	1.83	10.00	18	2.25	41
513	Low Density Residential	69.68	100.30	4.00	401	2.40	963
513	Residential Mobile Home	3.57	0.77	6.00	5	2.40	11
513 Subtotal					**424**		**1,015**
560	Low Density Residential	15.93	0.11	4.00	0	2.40	1
560 Subtotal					**0**		**1**
632	Commercial	13.87	0.00				
632	High Density Residential	96.72	32.16	10.00	322	2.25	724
632	Low Density Residential	22.99	51.05	4.00	204	2.40	490
632 Subtotal					**526**		**1,214**
660	Low Density Residential	223.58	101.20	4.00	405	2.40	972
660 Subtotal					**405**		**972**
880	High Density Residential	3.90	0.00	10.00	0	2.25	0
880	Low Density Residential	25.38	3.83	4.00	15	2.40	37
880 Subtotal					**15**		**37**
Total Population Increase at Buildout							**3,645**

EXHIBIT 3-20
Demand Projections

Year	Service Population	ADD (mgd)	MDD (mgd)	3-day MDD (mgd)	MDD + Weather & Industrial Allowances (mgd)
2003	10,768	3.0	6.9	6.3	10.9
2004	10,886	3.0	7.0	6.3	11.0
2005	11,006	3.0	7.0	6.4	11.0
2006	11,127	3.1	7.1	6.5	11.1
2007	11,250	3.1	7.2	6.6	11.2
2008	11,373	3.1	7.3	6.6	11.3
2009	11,499	3.2	7.4	6.7	11.4
2010	11,625	3.2	7.4	6.8	11.4
2011	11,753	3.2	7.5	6.8	11.5
2012	11,882	3.3	7.6	6.9	11.6
2013	12,013	3.3	7.7	7.0	11.7
2014	12,145	3.3	7.8	7.1	11.8
2015	12,279	3.4	7.9	7.2	11.9
2016	12,414	3.4	7.9	7.2	11.9
2017	12,550	3.5	8.0	7.3	12.0
2018	12,688	3.5	8.1	7.4	12.1
2019	12,828	3.5	8.2	7.5	12.2
2020	12,969	3.6	8.3	7.6	12.3
2021	13,112	3.6	8.4	7.6	12.4
2022	13,256	3.6	8.5	7.7	12.5
2023	13,402	3.7	8.6	7.8	12.6
2024	13,549	3.7	8.7	7.9	12.7
2025	13,698	3.8	8.8	8.0	12.8
Buildout*	14,413	4.0	9.2	8.4	13.2

*Using the 1.1% growth rate, the buildout population of 14,413 will be reached in year 2030.

EXHIBIT 3-21. Average and Maximum Day Demand Projections

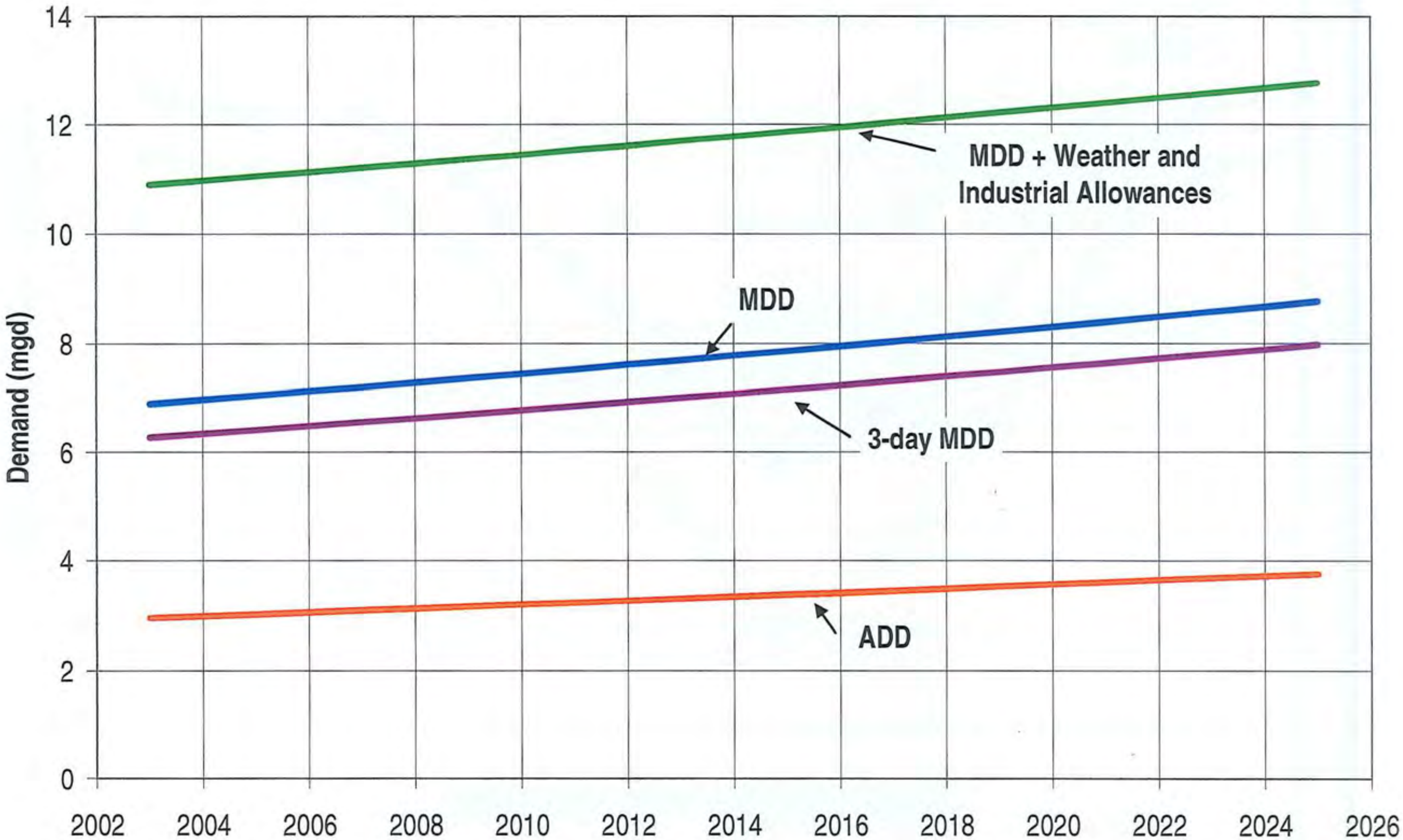


EXHIBIT 3-22. Projected Monthly Demands

CHAPTER 4

Water Supply

The City of The Dalles obtains its water supply from both surface water and groundwater sources. This chapter describes the current supplies and presents recommendations for development and expansion of these supplies. The topics include water rights, capacities, and expansion potential. The water quality of the groundwater sources is also discussed, as the water quality directly influences usable groundwater capacity.

Overview of Supply

For the period of January 1995 through June 2004, 94 percent of the water provided to the city's customers was obtained from the city's surface water source on South Fork Mill Creek. The remaining 6 percent was provided by three wells located within the city. The wells were used to meet peak demands during the summer season. The monthly production values from the surface and groundwater sources are summarized in **Exhibit 4-1**.

Previous Supply Evaluation

The city conducted a comprehensive water supply study in 1990-1991 (*City of The Dalles Water Supply Study*, April 1991, by James M. Montgomery). The study presented the following recommendations:

- The city should install meters on all customer connections and revise the billing structure from a flat rate to a volume rate. The city completed implementation of this recommendation by 1996.
- The city should obtain its future water supply through a combination of using existing and new supplies:
  - Maximize the production from the Jordan and Lone Pine Wells (to 3.55 and 4.35 mgd, respectively).
  - Begin planning for abandonment of the Wicks WTP. Replace this plant with a new plant, located near Sorosis Reservoir, which would draw its water supply from both Mill Creek and the Columbia River.

Subsequent to this study, the city rejected the plan of constructing a new WTP near Sorosis Reservoir. One of the primary concerns was using Columbia River water because it has an uncontrolled watershed and the city's withdrawal point is downstream of the Hanford nuclear site. The city council made the decision to develop the existing Wicks water supply to the maximum extent possible and postpone development of the Columbia River until needed due to growth, catastrophic loss of the Mill Creek supply, or future groundwater restrictions within the Critical Groundwater Area. This decision was not revisited during the preparation of this master plan.

Surface Water Supply

The city obtains its surface water supply for the Wicks WTP from the South Fork Mill Creek. The flow in South Fork Mill Creek is supplemented by water diverted from the Dog River watershed. The city installed Crow Creek Dam on South Fork Mill Creek to store the higher flows of winter and spring for release during the drier summer and fall. It is located downstream of the Dog River diversion. A schematic representation of the surface water supply system is shown in **Exhibit 4-2.**

Surface Water Rights

The city holds five surface water right permits, three of which have been perfected (certificated). **Exhibit 4-3** summarizes information pertaining to these rights.

EXHIBIT 4-3
Surface Water Rights

Source	Priority Date	Quantity	Description	Status
South Fork Mill Creek	1862	2 cfs (1.3 mgd)	Run-of-river right; diversion location is at the present Wicks WTP intake	Certificate
Dog River	1870	All water in Dog River at point of diversion	Allows diversion of Dog River flow into South Fork Mill Creek watershed	Certificate
Crow Creek Dam	1967	955 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Certificate
Columbia River	1986	40 cfs (26 mgd)	Point of diversion is upstream of dam; city has not used this right	Permit
Crow Creek Dam	1999	2100 acre-feet of storage and release	Allows storage in impoundment and release at rate desired by city, with capture at present intake facility	Permit

Surface Water Supply Facilities

The city's surface water supply system consists of the following major components (in order from upstream to downstream):

- Dog River Diversion Pipeline. This three and one-half mile pipeline diverts up to 8 mgd from the Dog River watershed to the South Fork Mill Creek watershed. The evaluation of this pipeline and recommendations for its replacement are presented in Chapter 6.
- Crow Creek Dam. This dam, located at the confluence of the South Fork Mill Creek with Crow Creek, impounds up to 800 acre-feet (260 MG).
- South Fork Mill Creek Intake. The intake facility, which was constructed in 2002 to comply with fish screening requirements, is located near the Wicks WTP. It has a capacity of 12 mgd. Water flows by gravity from the intake to the plant.

- Wicks WTP. The evaluation and recommendations relating to the WTP are presented in Chapter 5.
- Finished Water Transmission Pipelines. Two pipelines carry water by gravity from the WTP to the city distribution system. Their evaluation and recommendations relating to their replacement are presented in Chapter 6.

Crow Creek Dam Improvements and Expansion

Crow Creek Dam was constructed in 1967 and 1968. It is located about 15 miles southwest of the city. The dam is an earth and rockfill dam with a maximum height of 100 feet above the streambed. It has a crest length of 780 feet. The spillway is located on the left abutment. It is an unlined, open cut spillway channel.

Exhibit 4-4 summarizes the level recordings for the impoundment behind Crow Creek Dam. It has filled every year for the period of record, 1984 through 2005. It filled later in 2005 than in any previous year because of a dry winter and spring. However, significant rains in late March did fill the reservoir.

The feasibility of expanding Crow Creek Dam and its spillway capacity was evaluated in a study prepared for the city in 1996 (*Crow Creek Dam Seismic Stability and Hydrologic Analyses Project*, June 28, 1996, by Woodward-Clyde.) The study determined that it is feasible to raise the dam height by 35 feet to increase the storage capacity from 800 acre-feet (260 MG) to 1,970 acre-feet (640 MG). The study also determined that the existing spillway capacity of 2,500 cubic feet per second is insufficient to handle the probable maximum flood. A spillway enlargement is necessary whether or not the dam is raised.

The 1996 study presented conceptual designs and estimated costs for increasing the spillway capacity and raising the dam. If just the spillway is improved, without a raise in the dam, the estimated cost is $2.63 million. If both the dam raise and the spillway improvements are implemented in a joint project, the estimated cost is $9.05 million. (Both costs have been escalated from the 1996 study by applying a multiplier of 1.31. This is the ratio of the *Engineering News-Record* Construction Cost Index for Seattle of 5572 for May 1996 compared to the value of 7312 for November 2004.) The investment in the spillway improvements, per the conceptual design, would be lost if the spillway was improved and then the dam was later raised.

As part of the Water System Master Plan, CH2M HILL staff considered whether the spillway capacity could be increased using an approach that would not be a lost investment if the dam was raised at a later date. The scope of work for the master plan did not allow development of conceptual designs to the level at which their feasibility could be confirmed. It does appear, however, that one of the following approaches might be used to allow staged construction of the spillway improvements followed by the dam raise, without sacrificing the investment in the spillway improvements:

- Enlarge the spillway capacity using an open-channel entrance, to which a labyrinth weir could be added when the dam is raised
- Use a tunnel or concrete outlet, to which a morning glory riser could be added when the dam is raised

- Use a bathtub-type entrance with a box culvert that could be raised when the dam is raised.

If the city's demand projections do not warrant a dam raise in the near future, the city may wish to further examine these alternatives to determine if the spillway enlargement could be implemented in a manner that uses the investment when the dam is eventually raised.

Surface Supply Capacity: Needs and Potential Increase

The city's surface supply is the sum of the available flows in South Fork Mill Creek and Dog River, reduced during times of storage in Crow Creek Dam and supplemented during times of release from the dam.

Based on discussions with city staff, it was determined that the watershed should be capable of sustaining supply capacity at 80-90 percent exceedence levels (8 to 9 years out of 10). The surface water facilities, at buildout, should be designed at a capacity that equals the 80-90 percent exceedence level of the watershed. During the one or two years out of ten that the surface water supply is inadequate to meet system demands, the city can use wells more often or address the shortfall through curtailment measures.

The U.S. Geological Survey (USGS) maintained a stream gauging station (No. 14113400) on Dog River, at the city's diversion location, from October 1959 through September 1971. The USGS has compiled and made available these values on its web site. In addition, the USGS developed a statistical summary of the monthly flows for 10 to 90 percent exceedence conditions (meaning flow values that are expected for 1 out of 10 years through 9 out of 10 years).

The USGS also operated a stream gauging station (No. 14105850) on South Fork Mill Creek, downstream of Crow Creek Dam, from October 1960 through September 1974. These records are also available on the USGS's web site. However, the USGS did not develop a statistical summary to predict flows in South Fork Mill Creek.

The following methodology was applied to develop statistical estimates of the flows in South Fork Mill Creek:

Only the records through 1967 were considered, because this was prior to contributions from the dam.

The daily flow records for South Fork Mill Creek were compared to the records for Dog River and Mosier Creek, two neighboring watersheds, to determine the best correlation. The USGS has developed statistical summaries for Dog River and Mosier Creek, but not for South Fork Mill Creek or Mill Creek.

The best correlation was between South Fork Mill Creek and Mosier Creek.

The degree of correlation between South Fork Mill Creek and Dog River or Mosier Creek was also compared for low flows (less than or equal to 14 cfs) and very low flows (less than or equal to 2 cfs). The correlation for low flows was not as strong as for all flows.

A similar correlation by month instead of by daily flow value was also performed. The monthly correlation provided a better correlation between the low flows for South Fork Mill Creek and the other two streams. The strongest correlation was for Mosier Creek.

Because the correlation between South Fork Mill Creek and Mosier Creek was the strongest both for overall daily flows and for low flow months, the statistical summaries for Mosier Creek were used to predict exceedence values for South Fork Mill Creek.

The correlation equation for each month was used to develop 90 percent exceedence values for South Fork Mill Creek.

Exhibit 4-5 summarizes the calculated 90 percent exceedence values for South Fork Mill Creek. It is anticipated that the flow in South Fork Mill Creek will exceed these values in nine out of ten years.

EXHIBIT 4-5
Predicted 90 Percent Exceedence Values for South Fork Mill Creek

Month	90% Exceedence (cfs)	90% Exceedence (mgd)
January	4.3	2.8
February	5.7	3.7
March	10.0	6.5
April	5.8	3.8
May	10.2	6.6
June	10.4	6.7
July	6.7	4.3
August	6.0	3.9
September	5.1	3.3
October	4.1	2.7
November	4.7	3.1
December	3.9	2.5

The sum of flows in South Fork Mill Creek and Dog River, compared to demand projections, provide an indication of the needed impoundment storage and the amount of water available to store. Additionally, it is necessary to take into account the channel losses between the dam and the WTP intake, and the minimum bypass flows for fish habitat. These reduce the flows available for withdrawal by the city.

The available watershed flows and projected 2025 demands are compared in **Exhibit 4-6.** The curves illustrate periods when the available stream flows exceed demands and water is available for filling the dam. Historically, there has been a high runoff time in March and again in May and June. The current demand curve is lower and the dam typically begins filling in February and continues filling through the spring. The curves also illustrate deficit periods—times when the demands will exceed the available stream flow and water must be withdrawn from the dam. The available stream flows for this example are based on 90 percent exceedence levels. Stream flows are expected to exceed these values in nine out of ten years.

Exhibit 4-7 summarizes the surplus and deficit quantities for varying combinations of streamflow and demand levels. For 2025 demands, the surplus available for filling the dam exceeds the required storage even at the 90 percent exceedence levels. The buildout demand condition shown in this table represents obtaining a 12 mgd maximum day production from the plant. At this demand level, the available surplus for filling the dam is less than the needed storage at the 90 percent exceedence levels. However, even if just the Dog River flow is increased to 50 percent exceedence levels, the surplus is adequate for meeting the storage need.

Detailed analyses that support these summary values are provided in Appendix A.

EXHIBIT 4-7
Annual Streamflow Surplus Versus Needed Storage

Demand Condition	Streamflow Conditions	Surplus Available for Filling Dam (MG)	Storage Needed In Dam to Meet Demands (MG)
2025	90% exceedence for both South Fork Mill Creek and Dog River	330	270
2025	90% exceedence for South Fork Mill Creek and 50% exceedence for Dog River	930	130
Buildout[1]	90% exceedence for both South Fork Mill Creek and Dog River	160	720
Buildout[1]	90% exceedence for South Fork Mill Creek and 50% exceedence for Dog River	620	430

[1] Buildout based on obtaining a 12 mgd maximum day supply from the Wicks system.

Surface Water Supply Findings

The capacity analysis of the city's surface water supply resulted in the following findings:

- The city's existing Crow Creek Dam impoundment is inadequate to meet 2025 demands. (The demand analysis indicates that 840 AF of storage is needed and Crow Creek Dam provides only 800 AF. Additionally, the city finds it desirable to leave 20 feet (61 AF) of depth over the outlet works to avoid freezing problems.)
- The South Fork Mill Creek and Dog River watersheds supply adequate water to meet 2025 demands in at least 9 out of 10 years.
- The proposed dam raise of 35 feet will provide a storage volume that is slightly inadequate for meeting ultimate demand needs for 9 out of 10 years. Furthermore, the watersheds are not capable of filling this larger dam at a 90 percent exceedence level. However, with carry-over storage from one year to the next, the increased dam raise will supply adequate water almost every year.
- It is reasonable to target a maximum day demand production of 12 mgd from the Wicks system, provided the dam raise is implemented. The city has targeted 12 mgd as the buildout of the Wicks system. The intake has already been sized for this capacity. Other

facilities—the WTP, the clearwell, the finished water transmission pipelines—will be sized for the buildout capacity as they are expanded.

Groundwater Supply

The Dalles currently uses three wells as part of its water supply: Lone Pine, Marks, and Jordan. Together, the three wells have supplied approximately 6 percent of the city's water supply on an annual basis over the past 10 years. All three wells are located within the city limits. The Jordan and Marks wells are located in the west-central area and Lone Pine Well is located on the east edge of the city near the I-84 Freeway.

The discussion and evaluation of the city's groundwater supply is based on data supplied by the city's operators, conversations with the Wasco County Water Resources Department Watermaster (Bob Woods), and from the city's groundwater study, *Groundwater Supply Capacity Evaluation*, July 1999, by Golder Associates.

Groundwater Rights and Capacities

Exhibit 4-8 summarizes the city's groundwater rights. The city has three perfected and certificated water rights (12.64 cfs), an unperfected transferred water right (1.5 cfs) and two groundwater registrations (5.49 cfs) for a total of approximately 19.63 cfs or 12.7 mgd. Because 1.5 cfs was transferred from the Mill Creek Well to the Marks Well and the Marks Well is not capable of pumping its right of 2.68 cfs plus 1.5 cfs, this transfer has not been completed. In addition, 2,300 gpm of the City Hall Well rights were not transferred to Lone Pine Well. Therefore, the city has 12.64 cfs (8.4 mgd) of perfected rights available for use at existing wells. Through completion of the existing transfer and by pooling existing rights, the city may be able to increase its water supply as described in Appendix B.

The available capacity for meeting the city's needs is significantly lower than the city's groundwater rights because of three factors:

- Production from the wells is limited to the pumping capacities of individual wells and the capacity of the distribution system at the locations to which the wells are connected.
- The area in which the city's wells are located has been designated The Dalles Critical Groundwater Area by the WRD because of declining water levels in the aquifer. Although the levels have stabilized in recent years, the annual withdrawal by The Dalles and other groundwater users may be restricted by the state if water levels begin again to decline.
- The groundwater quality from the Marks and Jordan Wells is undesirable, so the city limits their contribution to the system. Both wells produce water with high manganese levels; therefore, these wells are not considered a reliable part of the city's supply.

The Lone Pine Well provided an average of 1.1 mgd during August 2005. This was the highest sustained production ever obtained from this well. The aquifer and pump capacity exceed 1.1 mgd. However, the distribution piping limits the use of this well until additional transmission piping is added.

EXHIBIT 4-8
The Dalles Groundwater Rights

Well Name	Priority Date of Right	Rate (cfs)	Rate (gpm)	Rate (mgd)	Comments
Lone Pine	1959	4.46	2,000	2.88	Right of 4.46 cfs has been perfected.
City Hall	1923	5.12	2,300	3.31	Groundwater Registration 4107 identifies this well as the point of appropriation for 5.12 cfs for municipal use. The City Hall Well is not currently in use.
Jordan	1953	5.50	2,468	3.55	Right of 5.50 cfs has been perfected.
Marks	1940	2.68	1,203	1.73	Right of 2.68 cfs has been perfected. Mill Creek Well right of 1.5 cfs was transferred to Marks Well, giving it a total right of 4.18 cfs. However, transfer has not been completed, because pumping capacity is only 2.90 cfs (1300 gpm), so city has not been able to document use of full 4.18 cfs at Marks Well.
Stadelman	1910	0.37	165	0.24	Groundwater Registration 4106 identifies this well as the point of appropriation for 0.37 cfs for municipal use. The Stadelman Well is not currently in use.
Mill Creek	1945	1.50	673	0.97	Has seasonal use restriction (April 1 through October 1) because well was originally an irrigation well. Right was perfected for Mill Creek Well, but was transferred to Mark's Well. Mill Creek Well is no longer in use.
Total Rights		19.63	8,809	12.68	The city holds three perfected and certificated water rights (12.64 cfs), one unperfected transfer (1.50 cfs) and two groundwater registrations (5.49 cfs).

Note: The water rights vary in their use of either cfs or gpm to describe the maximum allowed rate, so both units are presented above. In addition, the units of mgd are used because this is the most common terminology used for describing water system capacity.

As noted in Exhibit 4-8, the city has transferred rights from abandoned wells to the active wells. The city could implement the further step of pooling all groundwater rights because the wells draw from the same aquifer. In the city's past informal discussions with WRD, the department had indicated that pooling the groundwater rights might be a good option.

Appendix B provides recommended actions for the city to follow to more fully use the groundwater rights.

The Critical Groundwater Area designation gives the WRD the right to limit pumping, even of perfected water rights, if the annual withdrawals from the aquifer result in declining water levels. This has not been the case in recent years. The groundwater levels have stabilized in the past 5-10 years. Although a detailed water balance has not been performed, the Wasco County Watermaster believes that the reason for the stable levels has been the decline in operations at the aluminum plant in the city. Northwest Aluminum holds the largest quantity water rights for the aquifer, apart from the city's rights. They hold three groundwater rights, all with a priority date of 1957, for a total of 16 cfs (10.3 mgd). In recent years, they have significantly reduced their pumping as the plant operations have declined.

Two of the city's certificated water rights and the unperfected transfer are senior to the aluminum plant's rights. The only right that is junior is the 1959 right for 4.46 cfs (2.88 mgd) from the Lone Pine Well. If necessary, it may be possible for the city to transfer more senior rights to this well. However, even if the city's rights are senior to other users, if the city increases its pumping of the aquifer, resulting in declining water levels in the aquifer, WRD may enforce limitations on the city for its groundwater withdrawals.

Exhibit 4-9 summarizes the current pumping capacities from the three active wells: Lone Pine, Jordan, and Marks. This table also indicates the potential pumping capacity for each well, based primarily on the findings of the 1999 groundwater study. The total potential capacity of 8.3 mgd is less than the city's groundwater rights of 12.7 mgd.

EXHIBIT 4-9
Groundwater Pumping Capacities: Existing and Potential

Well	Current Capacity (gpm)	Current Capacity (mgd)	Potential Capacity (gpm)	Potential Capacity (mgd)	Comments
Jordan	1,950	2.8	2,460	3.5	Will require new pump and motor to increase capacity; possibly also new piping at well and in nearby distribution system. See discussion about iron and manganese concerns in this well. Currently only used for a few hours per day on peak use days.
Marks	1,300	1.9	1,300	1.9	Does not appear to have additional capacity beyond current pumping rate. See discussion about iron and manganese concerns in this well. Currently only used for a few hours per day on peak use days.
Lone Pine	1,600	2.3	2,000	2.9	1999 study concluded that pump should produce more than 1,600 gpm. It may be worn pump or inefficient discharge piping. May require new pump and motor to obtain 2,000 gpm.
Total	4,850	7.0	5,760	8.3	

Groundwater Quality

The Jordan, Marks, and Lone Pine wells all draw water from the Columbia River Basalt Group. However, the water quality from the Lone Pine Well differs from the water quality from the other two wells. The Jordan and Marks wells exhibit high iron and manganese levels, particularly manganese. This is not the case from the Lone Pine Well. Iron and manganese are secondary drinking water quality standards, meaning that they do not have a negative public health implication. They do, however, cause staining of laundry and fixtures and are undesirable.

It may be that the wells draw water from different zones within the Columbia River Basalt Group that are hydraulically isolated from each other. The 1999 study suggested that the city could request the WRD to consider the Lone Pine Well as drawing from outside of the Critical Groundwater Area. Although this may be possible, it appears that the city would need to invest significant effort into this analysis and the outcome would be uncertain. This is not a recommended course of action, particularly because it may be advantageous for Lone Pine Well to be considered as drawing from the same aquifer so that the city can transfer senior rights to it if necessary.

Exhibits 4-10 through 4-15 provide historical data for iron and manganese levels from the three wells. The levels for Lone Pine Well have remained consistently below the secondary standards of 0.3 mg/L for iron and 0.05 mg/L for manganese. Water from both the Jordan and Marks wells regularly exceeds the manganese standard and occasionally exceeds the iron standard. The manganese levels have remained generally constant over the past 10 years. Because of the elevated levels of iron and manganese, the city has limited pumping of the Jordan and Marks wells to only a few hours per day for only the peak days of the summer.

The city receives inquiries from customers when the Jordan and Marks wells are brought on line because they notice the difference in taste, or water spotting occurs. The city used to receive complaints about black particles (oxidized manganese) in the well water from the two wells, but the complaints have ended since the city began sequestering iron and manganese with polyphosphates at the well heads. In addition to sequestering the iron and manganese, the polyphosphate treatment softens the deposits that line the pipes near the wells. The city's flushing program gradually removes the deposits. The flushing program does result in inquiries, if not complaints, because of the colored water that results.

In addition to iron and manganese, there is also a concern with turbidity levels in the Marks and Jordan wells, particularly the Marks Well. During the summer of 2003, the city used the Marks Well more than in previous years. It appeared that the increased use of the well resulted in higher turbidity levels than normally experienced. The city eventually curtailed pumping because of elevated turbidity. It is possible that the turbidity problem in Marks Well is caused by the elevation of the well pump. It appears that the primary aquifer contribution to the flow is from the upper water-bearing zone and the well pump is set in this zone. The pumping in the area of this water-bearing zone may contribute to turbidity. Turbidity may also result from aeration and oxidation of the upper water-bearing zone when the water level declines from pumping. It is possible that setting the pump lower in the well will reduce or eliminate the turbidity problem.

The 1999 groundwater study recommended additional testing of the well to determine the relative contribution of each zone. This is a sound recommendation in light of the turbidity problem.

In contrast to the Marks and Jordan wells, the Lone Pine Well produces water that has relatively low levels of iron, manganese, and turbidity. Its use has not resulted in customer inquiries or complaints.

Groundwater Rule

A federal rule for governing the treatment requirements for groundwater has been proposed by EPA and is expected to be finalized in 2006. It does not appear that this rule will impact the city's operation of its wells. This is further described in the regulatory section (Chapter 7) of this master plan.

Potential for Increasing Groundwater Supply

As noted above, the city's ability to increase its groundwater withdrawals is subject to the water levels in the aquifer remaining stable. Additional pumping, whether by the city or other users, that results in a lowering water level will result in the state taking action to limit withdrawals.

The city's best opportunity for increasing the contribution of groundwater for meeting demands is to maximize the production from Lone Pine Well. This well produces water that is of high quality and compatible with the city's surface supply. The limiting factor is that Lone Pine Well is located on the east side of the city, east of the gorge that bisects the city's distribution pressure zones. The current distribution piping does not allow the city to pump water from this well and distribute into areas west of the gorge. The hydraulic modeling and distribution evaluation task of the master plan analyzed improvements to enable the maximum use of this well (see Chapter 8). In addition, it may be necessary to replace the existing pump and motor, and possibly upgrade piping at the wellhead.

A second possible approach for the city to investigate is to install another well near the Lone Pine Well that would draw from a similar portion of the aquifer. This may require property ownership and will need consideration of wellhead protection issues. The 1999 study did not identify potential sites. The feasibility of this approach is currently unknown.

A third potential for increasing groundwater supply is to use aquifer storage and recovery (ASR) in the Jordan Well. ASR makes use of the storage capacity of the aquifer. It would involve treating excess water during the winter months at the Wicks WTP for storage in the aquifer through the Jordan Well. The same water can then be withdrawn during the summer months when the surface supply is inadequate to meet demands. The Jordan Well is recommended for ASR consideration because it has a high specific capacity, equal to approximately 85 gpm per foot of drawdown. The well could potentially produce as much as 5,000 gpm (7.2 mgd). The advantage of ASR is that the injected water displaces native groundwater and can potentially eliminate the problem of high iron and manganese. It will require detailed investigation of this well to confirm that it is a good candidate for ASR, as well as for permitting with the state. If it were appropriate and approved, it would also be necessary to make modifications to the well and wellhead. Even with these investments, this could provide a relatively inexpensive means to increase the city's water supply when compared to expanding the surface water supply. A disadvantage of ASR is the need to increase winter production from the Wicks system to provide the storage water.

Prior to developing new wells within the city or increasing the use of existing wells, consideration should be given to the ongoing cleanup efforts for the Kerr-McGee groundwater contamination plume. The analyses should take into account potential hazards from this contamination plume and if precautions are necessary to avoid impacting the movement of the plume and current extraction and treatment programs.

Supply Needs and Expansion Plan

As described in this chapter, the city obtains its supply from both surface water and groundwater. The Wicks WTP is the limiting component of the surface supply system, being rated at 3.4 mgd (see Chapter 5, Water Treatment Plant Analysis). By reducing filter run time between backwashes, operators have maintained a net production of over 4 mgd for periods lasting more than 30 days. However, using current industry standards for process sizing, 3.4 mgd is a reasonable, long-term capacity. The city's reliable groundwater supply is 1.1 mgd. This based on the flow that can be distributed into the system from the Lone Pine Well when the distribution limitations are considered. The production from the Marks and Jordan wells is not included because both produce water with undesirable quality.

Exhibit 4-16 provides a comparison of the supply capacity to the 2005 MDD. The total supply capacity equals 4.5 mgd (3.4 mgd from the Wicks WTP and 1.1 mgd from Lone Pine Well). This is less than the projected MDD for 2005 of 6.9 mgd. This deficit was met by a combination of approaches: 'borrowing' from distribution storage during the single peak day, using the Marks and Jordan wells, and running the Wicks WTP in excess of its rated capacity.

Expansion Alternatives

Three expansion alternatives have been described in this chapter:

1. Expand the surface water supply system (Wicks system) by raising the Crow Creek Dam, replacing the Dog River diversion pipeline with a larger line, and expanding the Wicks WTP.
2. Expanding the groundwater supply by expanding the capacity of the existing Lone Pine Well and by adding a second Lone Pine-area well.
3. Implementing an ASR program at the Jordan Well.

These alternatives were further analyzed and the findings were reviewed with city staff and the Citizens Advisory Committee (CAC) that was formed for the master plan project. The resulting capital improvement projects that are presented in this plan reflect the input from these groups. The CAC reaffirmed the priority of the city continuing to invest in the Wicks system. However, as a means to delay some capital expenditures in expanding the Wicks system, the decision was made to add a second Lone Pine-area well.

Exhibit 4-17 displays the expansion plan for the city. Current and projected supply capacities are overlain on the 3-day MDD and 3-day MDD plus industrial allowance demand curves. The 3-day MDD curve is considered a minimum level of supply that should be provided. The addition of the industrial allowance to this curve illustrates the minimum supply necessary should new industrial customers add demands to the city's system.

Three projects are planned for 2006-2008:

1. Expand the existing Lone Pine Well. This includes equipping the well with a larger pump and motor, and adding piping mains to enable the well to supply water to a larger portion of the distribution system.

2. Add a second well in the Lone Pine Well area. It appears that the aquifer and water rights will support a second well in this area, and it is expected that the water quality will be favorable, as is the water quality from the existing Lone Pine Well. A feasibility study is needed to confirm the preliminary concept and to identify a recommended site. The city may need to purchase land before proceeding with this project.
3. Implement near-term improvements at the Wicks WTP. A package of near-term improvements is described in Chapter 5 of this plan. In addition to addressing regulatory needs, these improvements will increase the rated capacity of the plant from 3.4 to 5.0 mgd.

EXHIBIT 4-1. Monthly Production for WTP and Wells

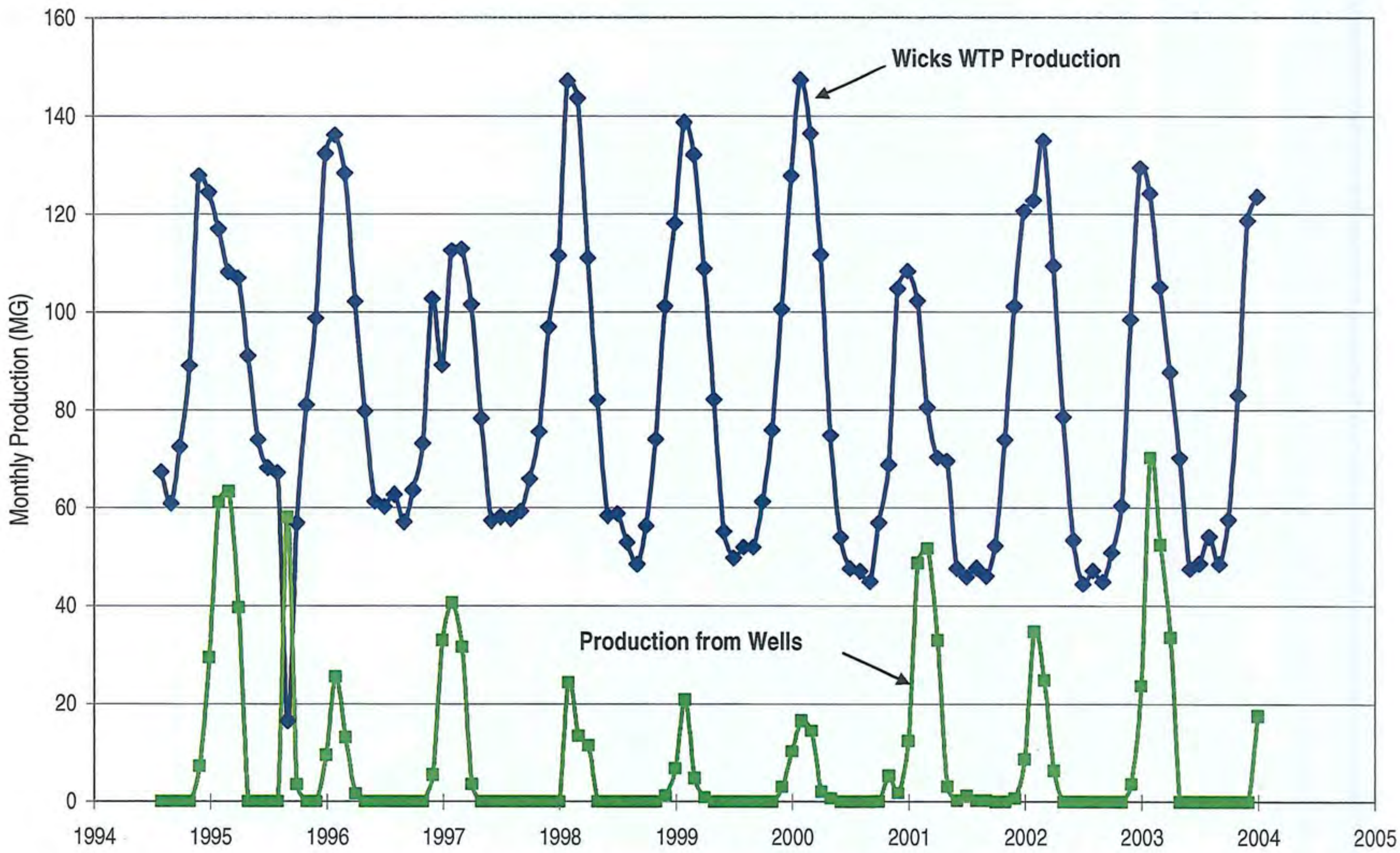


EXHIBIT 4-2
Existing Water Supply

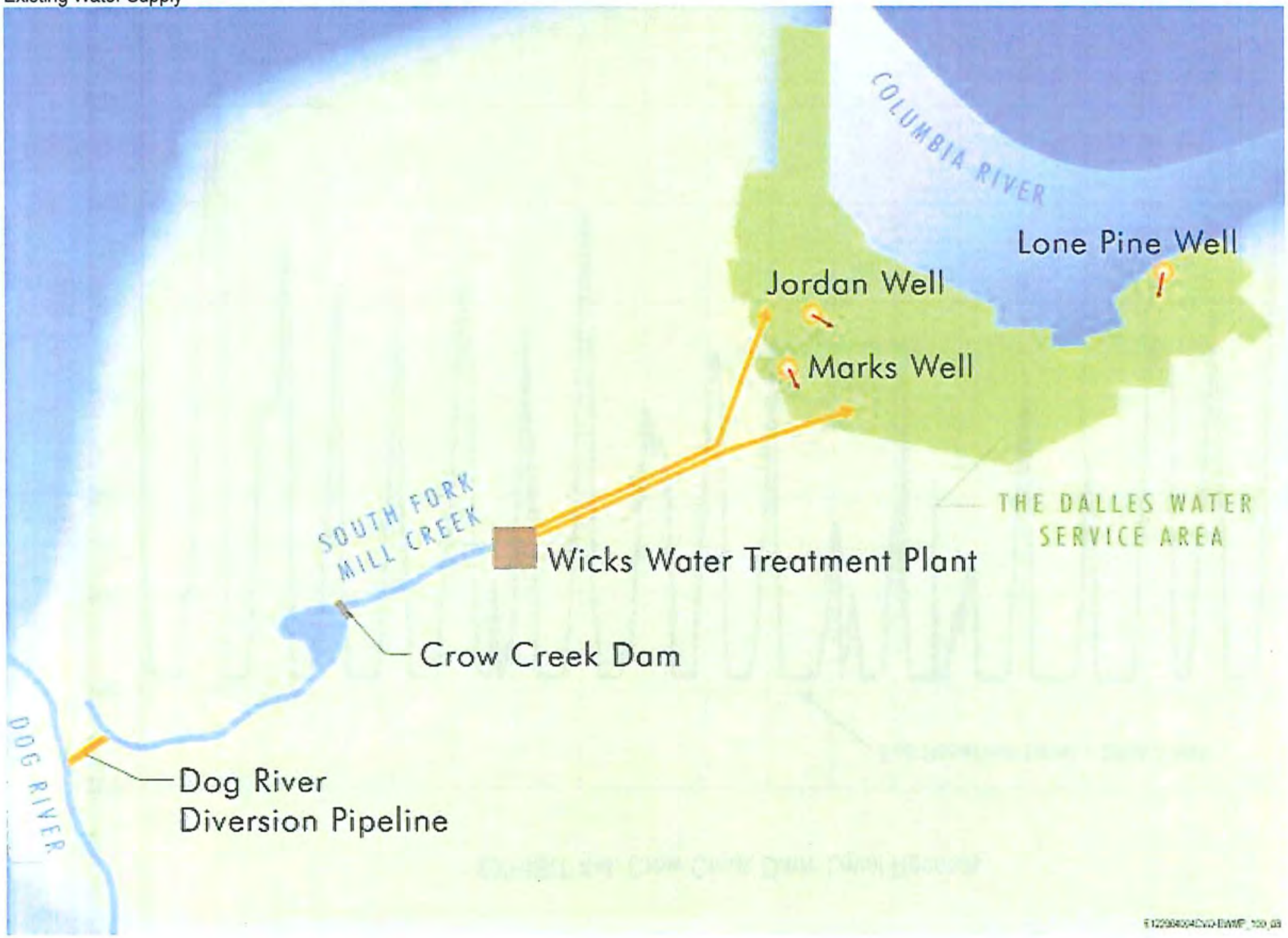


EXHIBIT 4-4. Crow Creek Dam: Level Records

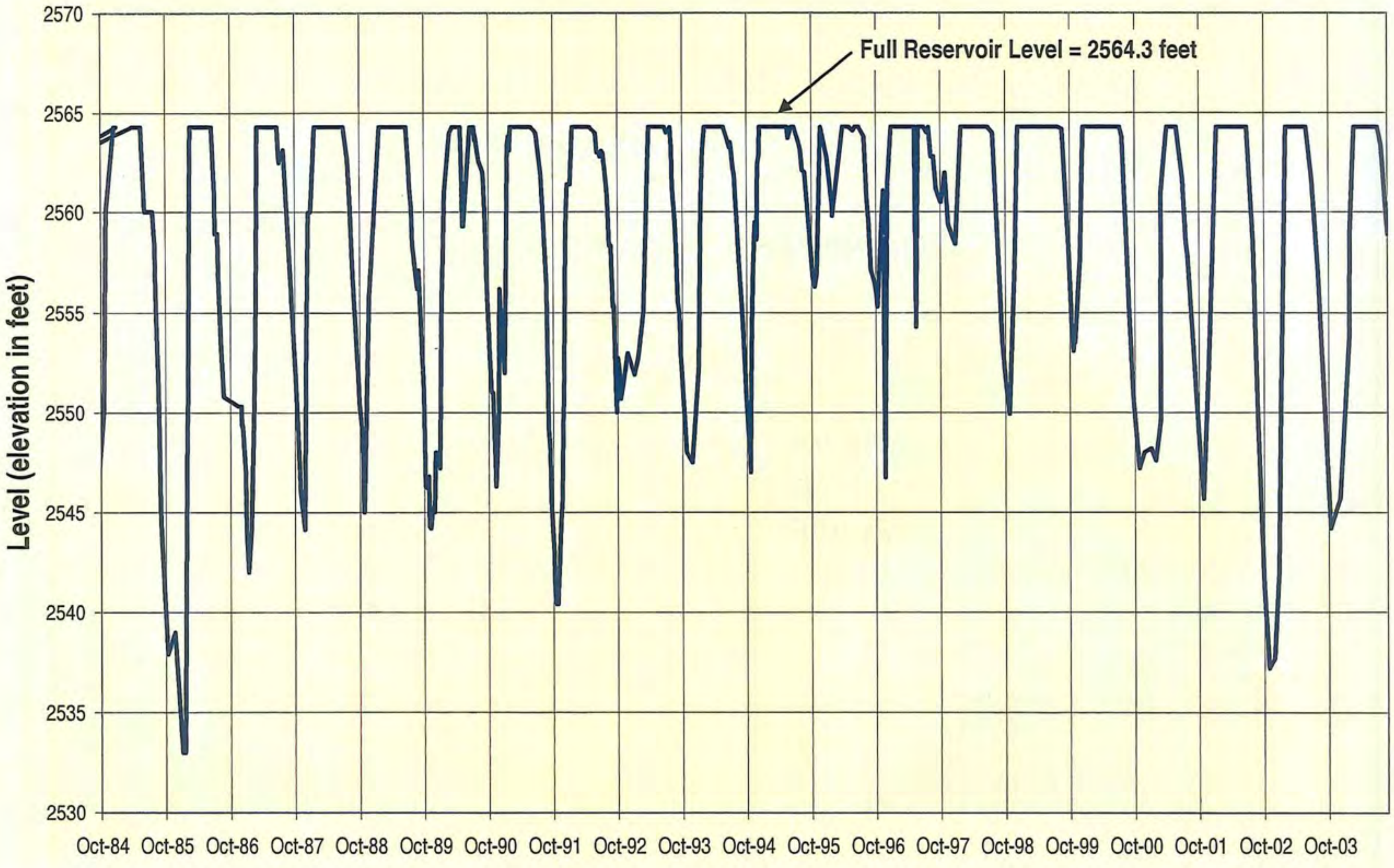


EXHIBIT 4-6. Supply versus 2025 Demands for 90% Exceedence Flows

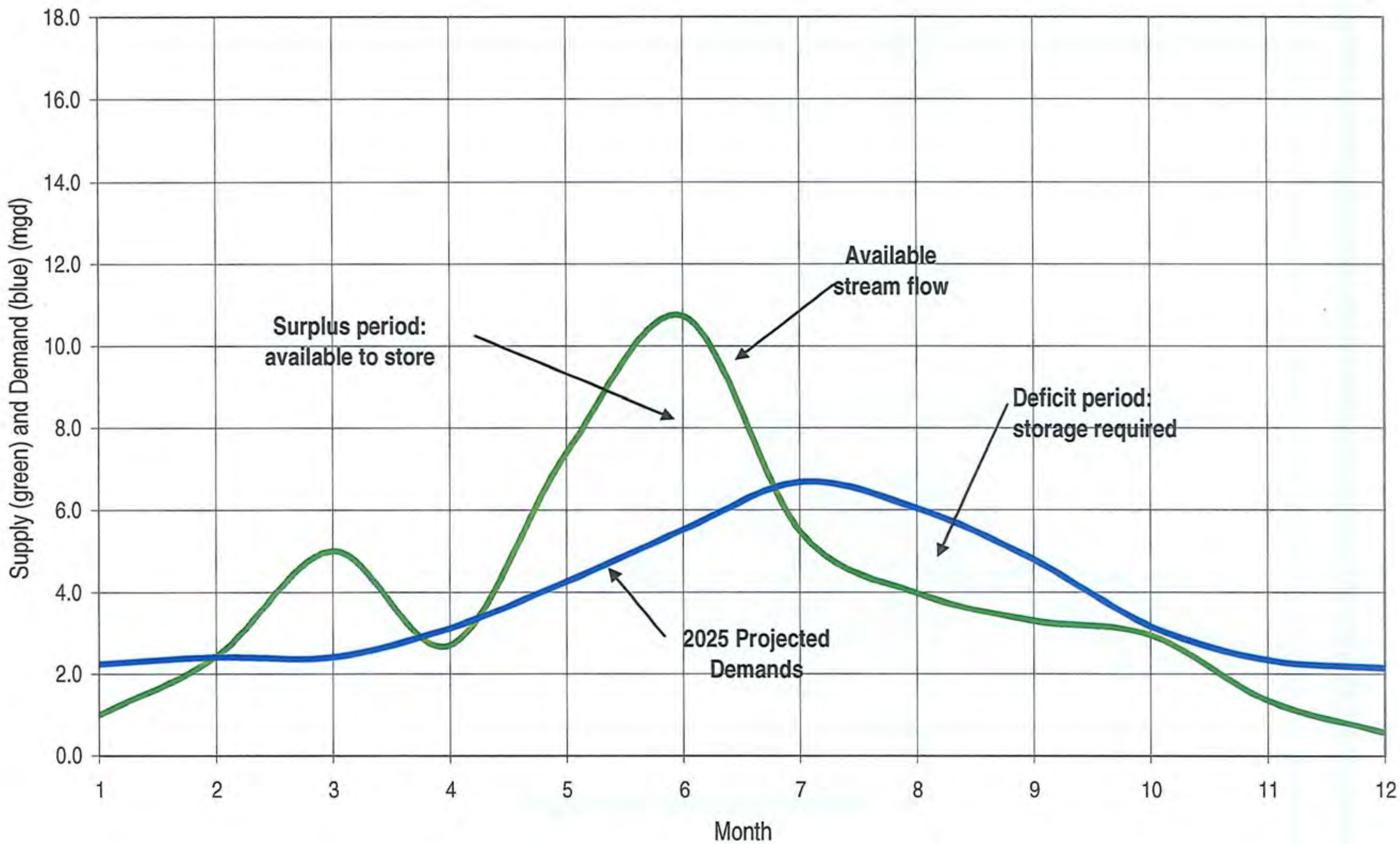


EXHIBIT 4-10. Jordan Well Iron Levels

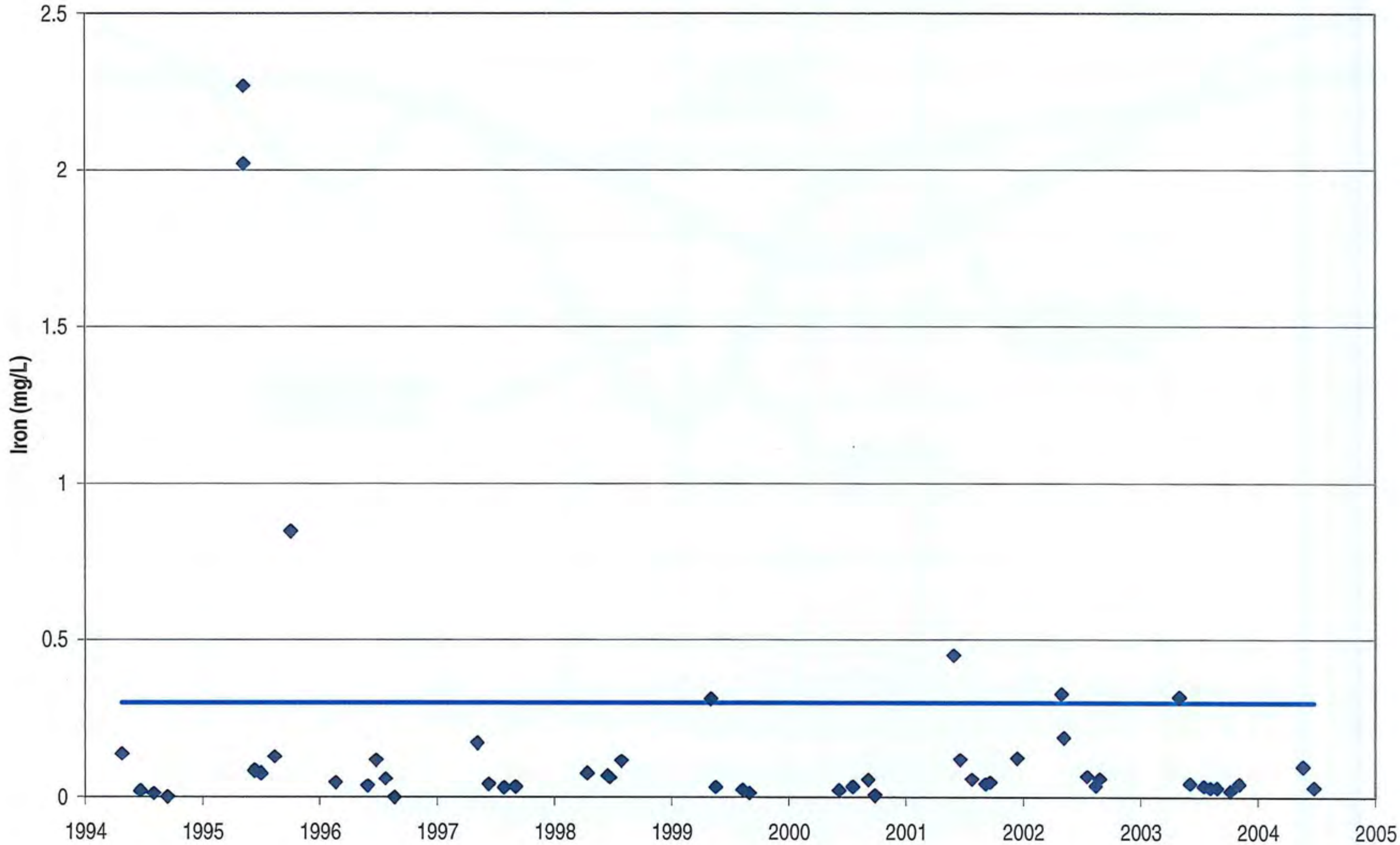


EXHIBIT 4-11. Jordan Well Manganese Levels

EXHIBIT 4-12. Marks Well Iron Levels

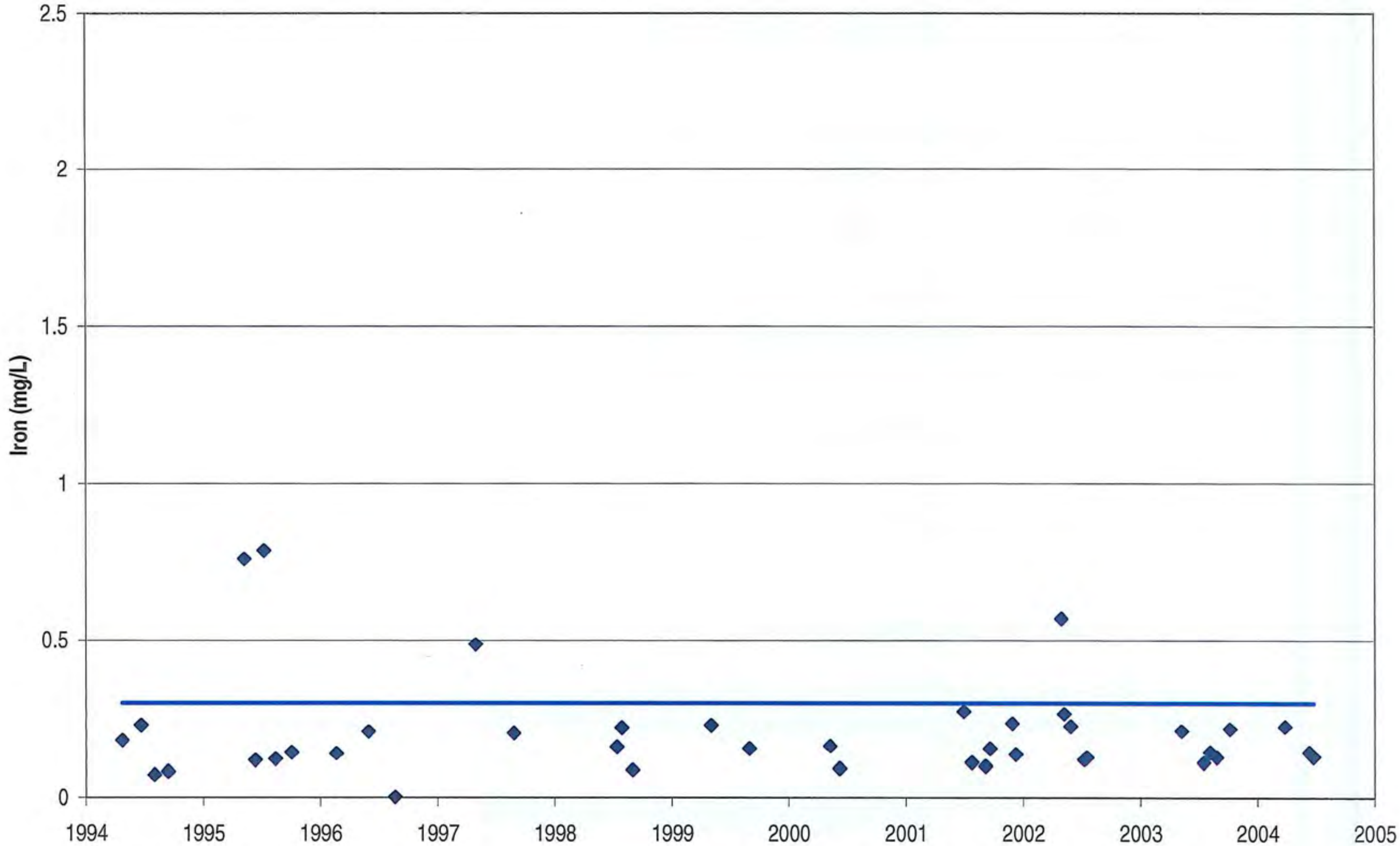


EXHIBIT 4-13. Marks Well Manganese Levels

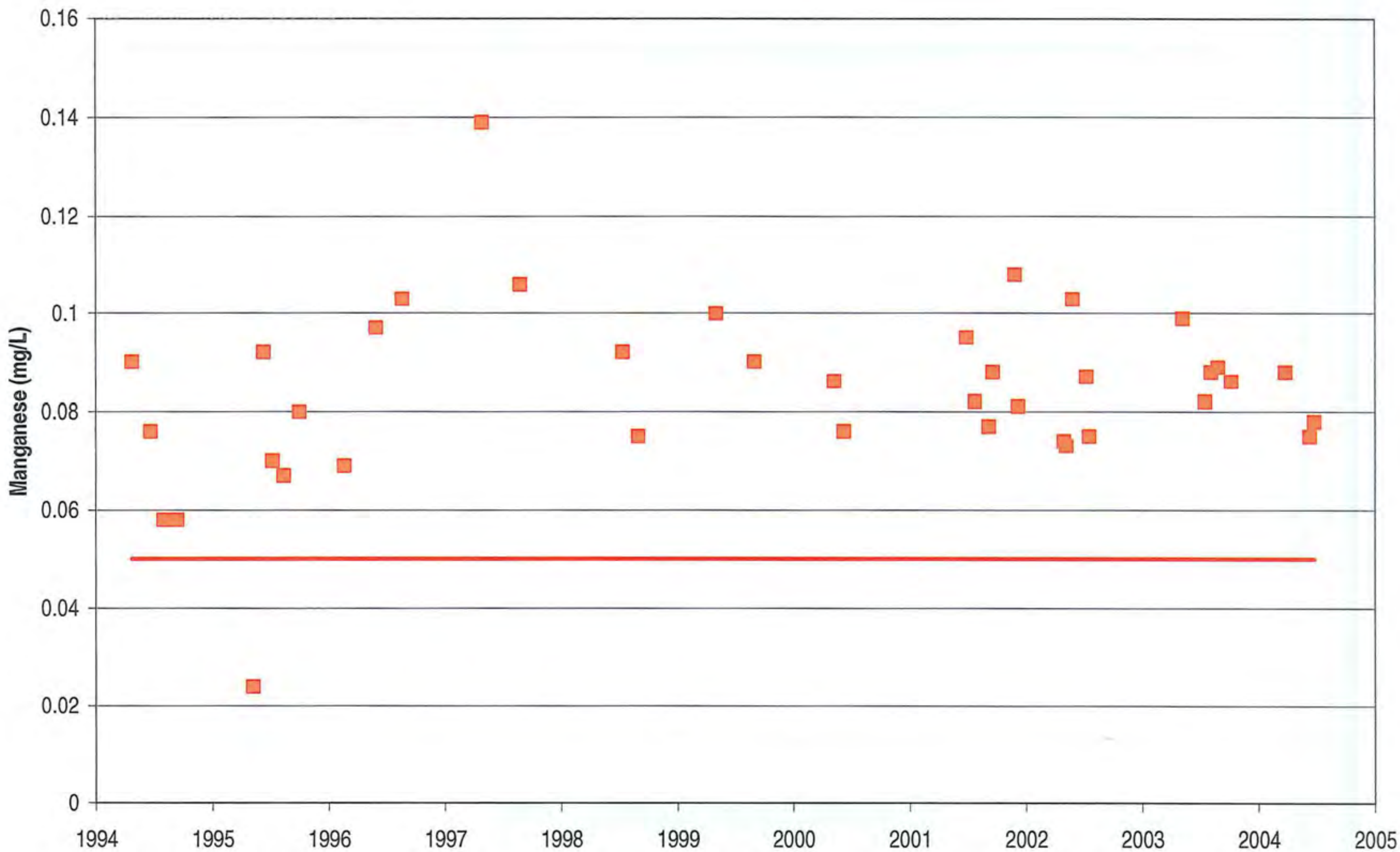


EXHIBIT 4-14. Lone Pine Well Iron Levels

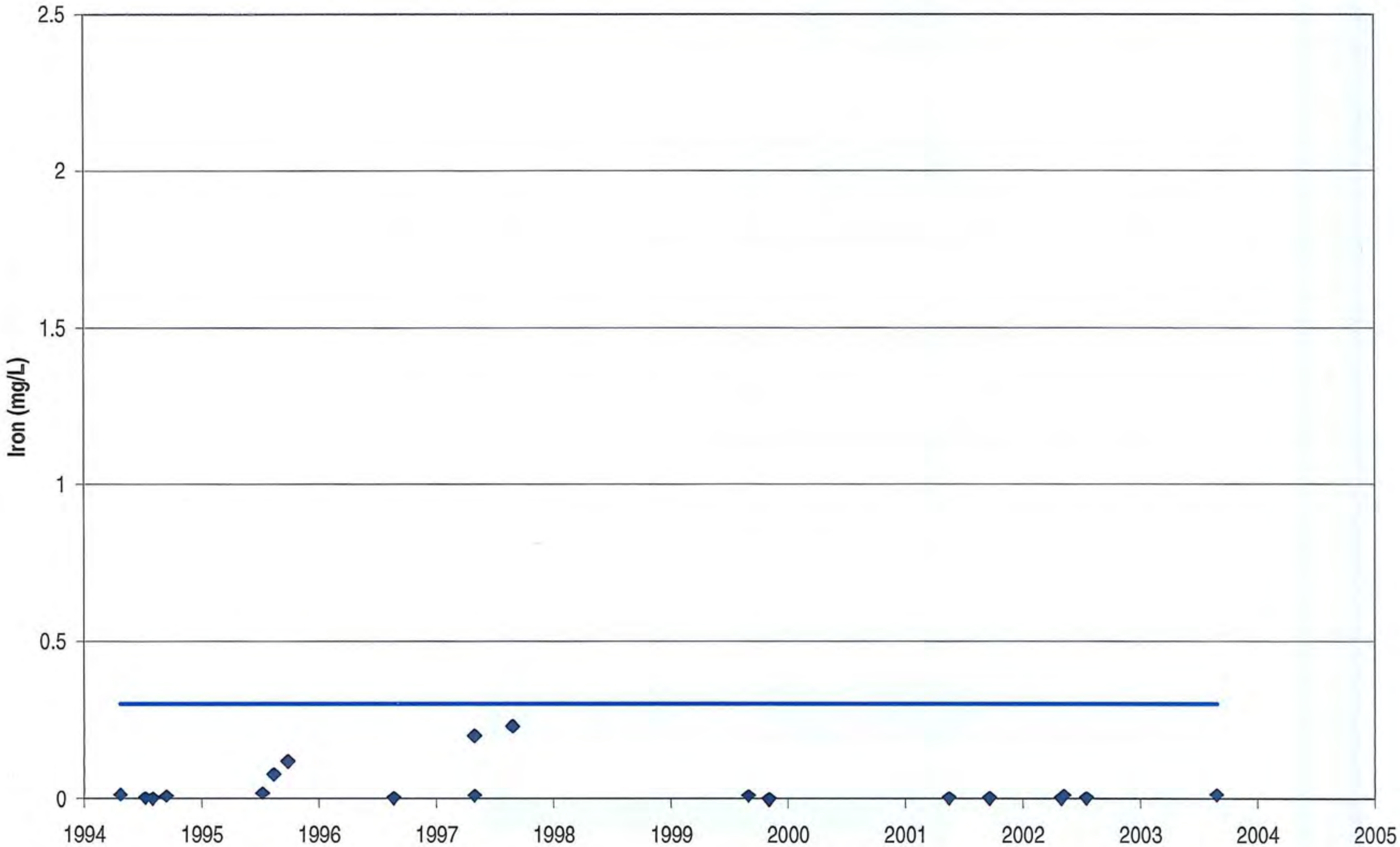


EXHIBIT 4-15. Lone Pine Well Manganese Levels

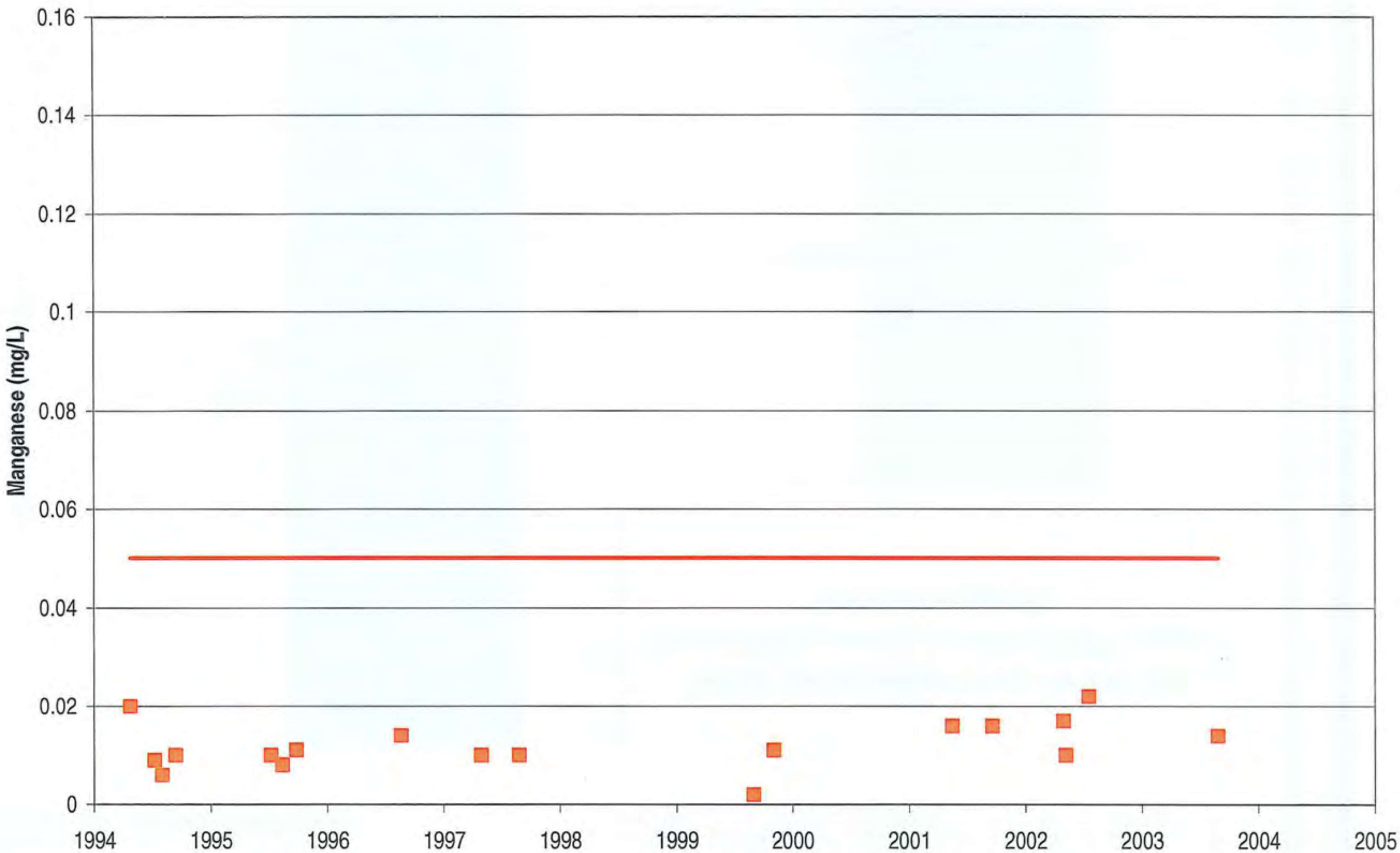


EXHIBIT 4-16
Available Supply Compared to 2005 Demand

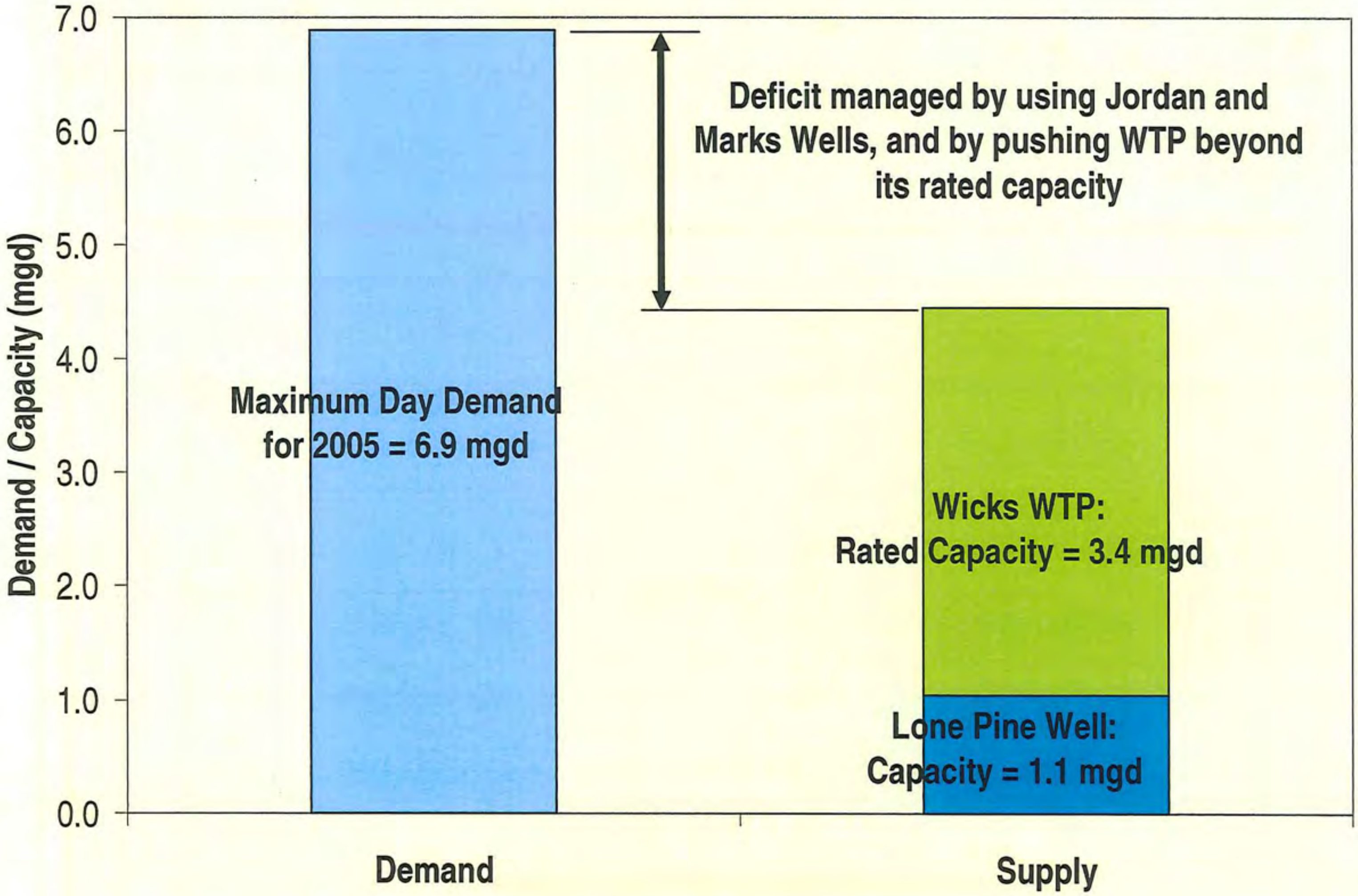
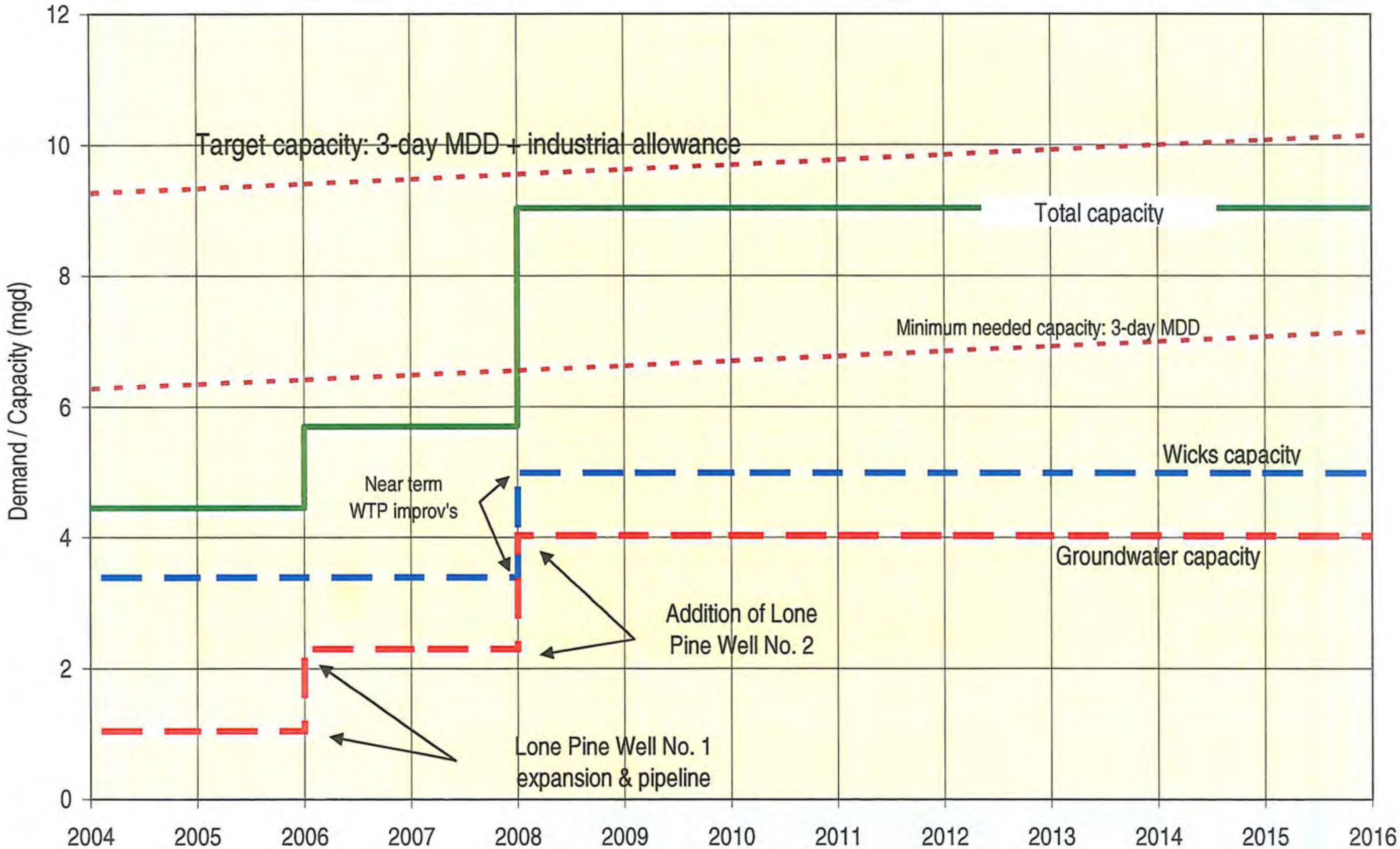


EXHIBIT 4-17 The Dalles Supply Expansion Plan



CHAPTER 5

Water Treatment Plant Analysis

This chapter presents recommendations for near-term and long-term expansions and improvements to the Wicks WTP.

The capacity of the Wicks watershed has been analyzed in a previous section. The watershed, once the dam raise is implemented, can supply up to a 12-mgd maximum day demand about 8 out of 10 years. A 12-mgd maximum day demand means that the 3-day maximum demand would equal about 11 mgd and the maximum month for the summer would equal approximately 10 mgd. Therefore, it is desirable to expand the plant to produce 10 mgd on a sustained basis and, if possible, plan the future facilities to produce up to 12 mgd.

Background

The Wicks WTP, located about 7 miles south of the city, was constructed in 1947. It treats water from South Fork Mill Creek, which is fed by the Crow Creek Dam Reservoir and the Dog River diversion. The plant has provided 94 percent of the city's water in recent years, with the remainder supplied by wells located within the city.

The maximum gross production to date has been approximately 6.4 mgd. When in-plant uses are subtracted—water used for filter backwashing, filter-to-waste, potable water for the operators, sample streams, and other uses—the maximum net production has been approximately 5.4 mgd. The net production is the water supplied to the city's customers. In-plant uses have therefore consumed 16 percent of the gross production at these high flows.

The Wicks WTP is credited with 2.5-log Giardia removal by the Oregon Department of Human Services, Drinking Water Program, at flows up to 6.05 mgd gross production.

As described in Chapter 3, the summertime (June through September) water demands of the community are about two and one-half times the wintertime (November through February) water demands. This aligns with the treatment capacity of the plant, because it is capable of producing more water when water temperatures are higher and when turbidities are lower, the conditions that generally occur during the summertime.

The Wicks WTP consistently provides a high quality drinking water to the community at a relatively low cost. The finished water quality meets all current drinking water standards. A primary reason for the low cost is that water flows by gravity to and from the plant. In addition, the plant has not required any significant rehabilitation in recent years.

The plant uses two parallel trains of flocculation, sedimentation, and tri-media filtration. It was designed to allow the addition of a third parallel train, thereby increasing the capacity by 50 percent. The plant is located in a narrow ravine, bounded by steep, rock walls on the east side and Mill Creek on the west side. These physical limitations leave room for only a third parallel train, but not a fourth train.

The raw water varies considerably in quality from one year to the next. During storm events, the turbidity can reach 200 to 400 nephelometric turbidity units (NTUs). The last big storm that produced difficult-to-treat water was the 1996 flood. The plant was temporarily removed from service when the turbidity reached 800 NTU.

The last master plan in 1991 recommended a new plant for treating Columbia River water. The city decided not to develop the Columbia River source until the Wicks source is developed to its full potential, primarily because of the unprotected nature of the Columbia River's watershed.

Evaluation of Existing Facilities

The information in this section is based on site visits to the plant, review of operating records and design drawings, and discussions with city staff.

Exhibit 5-1 describes the existing processes at the Wicks WTP. **Appendix C** provides a detailed description of the plant and its processes.

EXHIBIT 5-1
Summary of Water Treatment Processes

Process	Description	Dimensions	Total Volume (gal)	Total Area (sf)
Grit chamber	Hydraulic (slows water velocity to allow settling of heaviest particles)	28' long; width is from about 2' (upstream) to 5' (downstream); depth 1'-3" (upstream) to 2' (downstream)	1,170	100
Rapid mix chamber	Hydraulic (with baffle walls)	8'-8" long x 6'-1" wide x 4'-5" deep (with vertical baffles that are 3'-5" high)	1,740	53
Flocculation basins	2 parallel basins, each with two chambers. Hydraulic mixing, only.	29' long, each basin 12' wide, depth = 13'	67,700	696
Sedimentation basins	2 parallel basins, sloped bottom to low point near upstream entry; tube settlers in the downstream third of the basin; Trac-vac sludge removal equipment	80' long, each tank is 25' wide, depth from 9'-6" to 11'-3"	310,400	4,000
Filters	2 filters; media depth = 2.5', 3' from top of media to top of trough	Each 20' x 15'		600

Exhibit 5-2 presents the loading rates of the primary processes at varying flow rates, with discussion and conclusions for each following the exhibit.

EXHIBIT 5-2
Detention Time or Loading Rate by Process Compared to Industry Recommendations
Flow rates and detention times are for gross production

Process	3.0 mgd	4.0 mgd	5.0 mgd	6.0 mgd	Industry Recommendation*
Grit chamber detention time (min)	0.6	0.4	0.3	0.3	No recommendation
Flash mix chamber detention time (min)	1.8	1.4	1.1	0.9	No recommendation
Flocculation basins detention time (min)	32	24	19	16	Mechanical, with 20-30 minutes of detention time (existing flocculation is hydraulic)
Sedimentation basins loading rate (gpm/sf)	0.5	0.7	0.9	1.0	1.0 gpm/sf overall (0.3 gpm/sf for open area and 2.5 gpm/sf for area covered by tubes settlers)
Estimated unit filter run volume (gal/sf)	3,500	3,200	2,800	2,500	Greater than 5,000 gal/sf per filter run
Filter loading rate (gpm/sf)	3.5	4.6	5.8	6.9	No recommendation, but 3 to 5 gpm/sf is typical for these filters

*Based on *Ten States Standards* and *Water Quality and Treatment Handbook of Community Water Supplies*, published by McGraw Hill in association with American Water Works Association.

Grit Chamber

The grit chamber is no longer needed because the fish screens in the new intake remove most of the debris that used to be removed in the grit chamber. The grit chamber causes no harm, but is unnecessary.

Rapid Mix

The existing hydraulic rapid mix facilities are inefficient and would need to be upgraded if the existing facilities were to be expanded.

Flocculation

The existing hydraulic flocculation system is inadequate and would need to be upgraded to mechanical flocculation and, possibly, the volume increased to allow a minimum of 20- and preferably a 30-minute flocculation time.

Settling

The settling capacity is adequate. Although the Trac-Vac system does not cover the entire basin, the system is sized adequately.

Filtration

The filtration process is undersized, as identified in **Exhibit 5-2**. The operators corroborated this finding, indicating that the filter run time is reduced to only 5 hours at times when the flow rate increases to 5 mgd through the plant. A typical water plant will achieve run times of 24 hours at maximum loading rates. The unit filter run volume (UFRV) was calculated for this case and found to be about 2,700 gallons per square foot of filter area (gal/sf). A UFRV equal to 10,000 gal/sf is desirable and a value of 5,000 gal/sf is considered a minimum acceptable level.

The existing filters were designed using criteria of the day including a turbidity limit of 1 NTU. Using today's criteria, the filters should be limited to 4.0 gpm/sf. This rate results in a production of 1.7 mgd each, or 3.4 mgd total. These are gross production values, so the actual delivery to the city would be less. The plant typically operates at higher flow rates, but the short filter cycles and amount of wasted water are outside normally accepted industry standards.

The short filter runs are believed to be a result of three factors:

- High loading rate
- Use of filter aid polymer
- Tight media

A CH2M HILL-developed filter media model was used to determine whether a looser media could provide similar water quality performance at a higher loading rate. **Exhibit 5-3** summarizes findings from this analysis by comparing the existing media to two alternative designs. The alternative designs result in a similar clean-bed headloss and nearly equivalent particle removal performance at significantly higher loading rates. However, both alternatives require a deeper media (36 and 40 inches compared to the existing 30-inch depth).

EXHIBIT 5-3
Alternative Media Designs

Media	Depth (inches)	Recommended Loading Rate (gpm/sf)	Clean Bed Head Loss (in feet) at 5.0 gpm/sf	Log Removal Predicted by Model
Existing tri-media	30	4.0	1.2	2.5
CH2M HILL standard dual media: 24 inches of 1.0 mm anthracite over 12 inches of 0.5 mm sand	36	5.5	1.4	2.5
Deep bed dual media: 28 inches of 1.0 mm anthracite over 12 inches of 0.5 mm sand	40	6.0	1.6	2.5

Industry experience in the past 20 years has shown the benefits of deeper beds. These include:

- Provide excellent turbidity and particle removal

- Accommodate dual-media designs that accumulate head loss less rapidly
- Allow greater loading rates, resulting in treatment of greater quantities of water with less area
- Provide more solids storage
- The existing media support/underdrain uses Leopold blocks with 10 inches of supporting gravel. By replacing these underdrains with a type-S gravel-less underdrain, it is possible to gain 10 inches for additional media depth. Other, even lower-profile underdrain systems exist that may provide for even more media depth. A more detailed review of hydraulic distribution of backwash water is recommended to assess the suitability of these lower-profile options.

By making some modifications to the filter media and the flocculation process, CH2M HILL believes that the capacity of the filters can be increased to 2.6 mgd each (6.0 gpm/sf). This will yield a total gross production of 5.2 mgd. The existing plant has been operated at raw water flows of close to 5.6 mgd and produced high-quality water. However, the amount of water wasted is 7 to 11 percent, resulting in a net production of approximately 5 mgd. In addition, the short filter runs make this level of flow difficult to maintain. The filtration and flocculation modifications suggested will allow longer filter runs at the high flows and result in closer to 4 percent waste.

Increasing the media depth from 30 to 40 inches will result in increased expansion on backwash. It is recommended that the bottom of the backwash troughs be at least 18 inches above the top of the media. Further investigation will be needed at the time of design to ensure that this separation is provided.

Sludge Lagoons

There is one existing sludge lagoon, which is divided into two halves by a center wall. The overall dimensions are 145 feet long and 25 feet wide, providing an area of 3,625 square feet. The earthen basin is approximately 12 feet deep. Its volume is sufficient to store about one year's accumulation of solids removed from the raw water. It does not provide any drying. Water continuously overflows to the creek. The city contracts annually for removal of the accumulated solids. Because there is no opportunity for drying, the solids must be moved when the water content is high. As the plant capacity is increased, the sludge lagoon capacity will need to be increased or the loading will need to be decreased by recycling.

Existing Plant Capacity

Exhibit 5-4 summarizes the capacity of the current facilities. The most limiting process is filtration, having a recommended capacity of 3.4 mgd. The flocculation process limits gross production to approximately 4.9 mgd at the recommended loading rate.

The plant has exceeded these flow limits. It has treated flows up to 6.4 mgd (gross production). This has only been possible for limited periods by careful operation and at the expense of generating high waste flows (approximately 16 percent compared to a typical goal of 5 percent). At 16 percent waste flow, the net production from a gross production of 6.4 mgd has equaled approximately 5.4 mgd.

EXHIBIT 5-4
Capacity of Existing Plant Processes
Gross production

Process	Oregon Drinking Water Program CPE Findings	CH2M HILL Recommended Maximum Capacities	Comments
Rapid Mix	5.6 mgd	6.0 mgd	Improved rapid mix recommended to decrease coagulant dose
Flocculation	5.1 mgd	4.9 mgd	Mechanical flocculation recommended to achieve 4.9 mgd capacity
Sedimentation	6.7 mgd	6.0 mgd	Tube settlers have been effective in maximizing capacity of existing basins
Filtration	6.0 mgd	3.4 mgd	Filters can and have passed more water but the filter run times are as low as 4 hours and 10 percent of the water is wasted.

Near-term Improvements

The following improvements are recommended to optimize the use of the present facilities.

- Install the planned 4.3-MG clearwell. This project meets three significant needs: 1) it enables the plant to comply with the new, more stringent disinfection by-products rule by allowing the point of primary chlorine addition to be moved downstream of the filters (see Chapter 7, Regulatory Review); 2) it allows for more steady-state operation of the plant and the water quality benefits such operation provides, as opposed to adjusting plant flows more quickly in response to changing demands; and 3) the additional storage volume provides a buffer for times when it is necessary or desirable to shut off the plant, either to perform operation and maintenance activities or to allow highly turbid water to pass.
- Install mechanical flocculation to address the current limitation in flocculation capacity. The current flocculation system is rated for a maximum capacity of 4.9 mgd. Along with the filters, this limits the production capacity.
- Upgrade the filters by replacing underdrains with a gravel-less model, increasing the depth of the filter media, and revising the media selection to optimize removal efficiency and filter run times. The filters are the most limiting process in the existing plant. According to current standards, their design limits the plant to a production of 3.4 mgd. The filter changes will enable the filters to produce water at higher rates for longer periods.
- Add solids drying beds to eliminate the need for annual, contracted cleaning of the existing basins.

With these improvements, it is expected that the gross capacity of the treatment plant can be increased to 5.2 mgd on a sustained basis. The waste flow rate will be reduced from up to 16 percent for current high-rate operations, to possibly 4 or 5 percent. At a waste flow of 4 percent, a 5.2-mgd gross production rate results in a net production of 5.0 mgd.

With respect to the sequence of these improvements, it is recommended that the clearwell be the first improvement. The additional storage volume that it will provide will greatly reduce the risk of running out of water during shutdowns that will be needed to improve the flocculation and filters.

Engineered drying beds, compared to the existing lagoons, are recommended to provide drying of the solids. Preliminary criteria are as follow:

- Total depth = 7 feet
- Area = 20,000 square feet for existing plant capacity and 40,000 for buildout capacity
- Solids storage depth = 4 feet
- Water depth above solids = 1.5 feet
- Sand bottom = 6 inches
- Single underdrain pipe per basin
- Include inlet and outlet structures to control flow into basin, and overflow and underflow at outlet

The city has identified a possible location for new solids drying beds. Further investigation is necessary to determine if there is sufficient space to meet current and ultimate needs.

Buildout Expansion Alternatives

The two general alternatives for increasing capacity are to expand the existing facilities or to install new parallel or replacement facilities. The alternative of adding a new treatment plant at another location is not recommended because current infrastructure provides for supplying raw water to the plant and transmitting treated water to the city.

The existing plant facilities are generally in sound condition. If possible, it is desirable to expand these facilities instead of replacing them or adding another facility at this same location.

Many treatment processes are suitable for the treatment of the water from the South Fork of Mill Creek. Alternatives that might be considered include:

- Membrane filtration
- Upflow solids contact units
- Dissolved air flotation (DAF)
- Sand-ballasted sedimentation
- Lamella plate clarification

Rationale for further consideration of these options follows. **Appendix D** provides further description of these alternatives.

Membrane Filtration

Membrane filtration would not make good use of the infrastructure that is present and would result in two very different processes. For example, new chemical feed systems would be required for the membrane system, in addition to chemical feed for the existing plant. A membrane system would require a different operations philosophy. The operations staff would be faced with two separate treatment systems to operate. Because membrane systems are so different from those at the existing plant, it is recommended that a system that can better leverage the existing facilities be considered first.

Upflow Solids Contact

The upflow solids contact process provides the benefit of having a small footprint. However, this process responds poorly to changes in temperature, flow rate, and raw water quality. Because such changes occur during storm events, this technology is fatally flawed and is not a good choice.

Dissolved Air Flotation

DAF is designed to remove low-density solids. Algal cells represent a relatively small fraction of the solids in the water from the South Fork of Mill Creek. Furthermore, algae concentrations are only present seasonally and represent relatively short-term treatment challenges. Removal of the high-density colloids associated with storm events is the principal objective of the treatment process for the Wicks WTP. DAF is not a good fit for expansion of the Wicks WTP.

Sand-ballasted Sedimentation

Although sand-ballasted sedimentation offers good performance in a relatively small footprint, it does not appear to be a good fit for expansion of the Wicks plant. Successful operation depends more on the polymer dose than the alum dose; therefore, a different chemical feed system and operational philosophy would be required. Much of the sand ends up in the solids that must be disposed of, placing greater demand on the sludge lagoons. In addition, ozone is needed to remove excess polymer before filtration. Sand-ballasted sedimentation is not a good fit for the expansion of the Wicks WTP.

Lamella Plate Clarification

Lamella plate clarification uses a smaller footprint than the existing clarification system but its operation is similar to the existing process. The chemical requirements are essentially the same. There are multiple suppliers (GEWE, USFilter, MRI). Sludge production is similar to the existing process. Lamella plate clarification is a recommended process for expansion.

In-kind Expansion

The existing facilities produce a high-quality drinking water. The primary drawbacks are the area requirements, and the filter operation. The area available for expansion is limited and there may not be enough room for an in-kind expansion. During peak production, the filters require frequent backwashing.

The water treatment industry has moved to deeper bed filters because the extra media can treat more water with less investment in concrete walls, valves, and piping. Deeper filters should be considered for the expansion. The filter media should be a dual media with up to 4 feet of 1-mm anthracite over 1 foot of 0.5 mm sand. These recommendations should be re-evaluated during the predesign process to determine whether there is sufficient driving head available. In addition, it is recommended that two filters be added, each with surface area similar to the existing filters. Using a similar area will allow continued use of the backwash tanks used to supply backwash water at gravity flow to the existing filters.

Recommendations for Buildout Expansion

CH2M HILL recommends expanding the existing facility by using a lamella plate clarification system and adding two new filters of similar surface area to the existing filters. This approach maximizes the value of the existing facilities, including the chemical feed facilities (which require only minor changes), the control room and laboratory (which can be used as-is), and the treatment basins (which will continue in use). A replacement plant or an expansion by adding a new parallel plant would result in duplication of costs for some or all of these facilities.

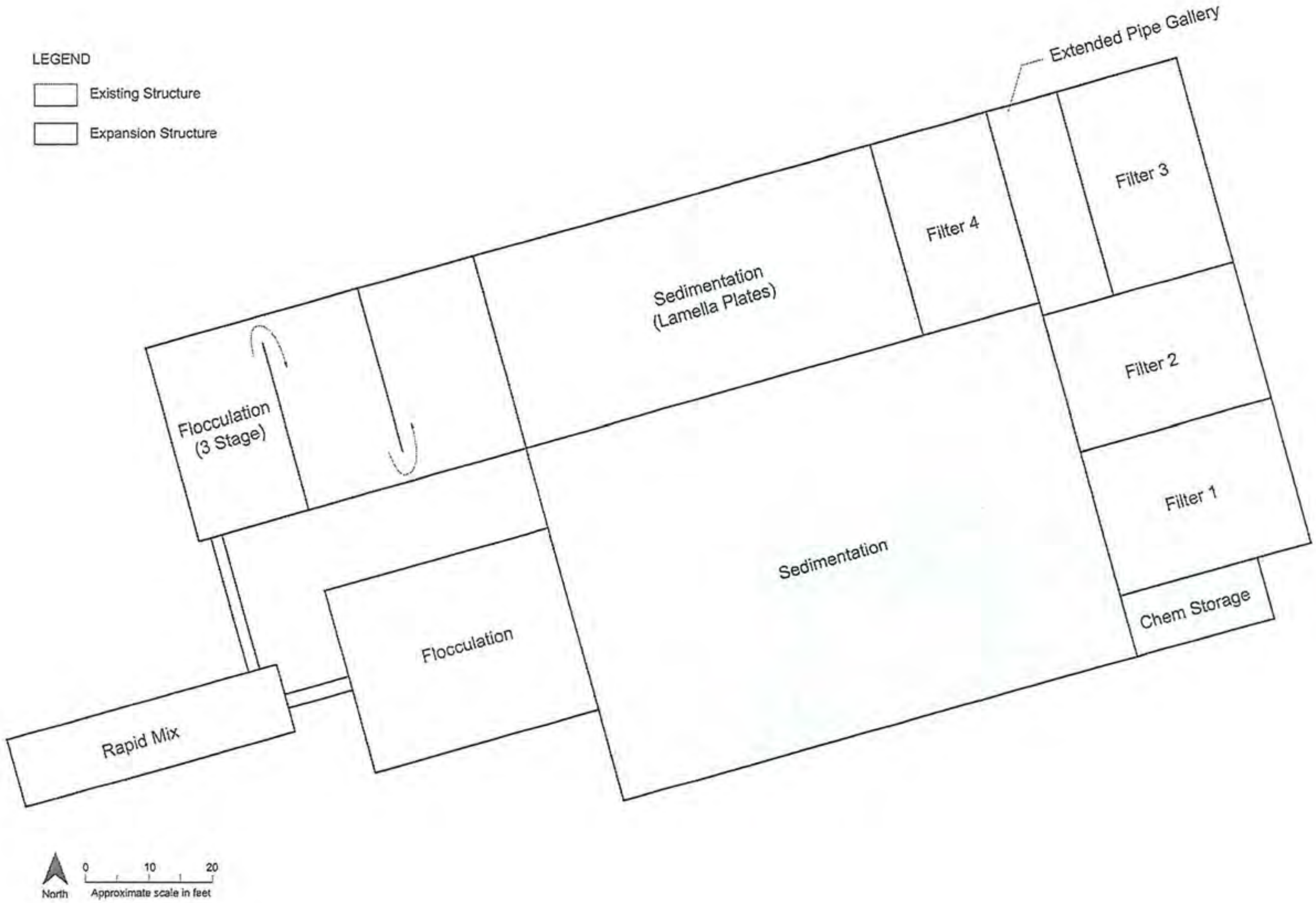
1. The existing grit removal basin should be replaced by a new rapid mix system. This system can either be an in-line static mixer or an in-line mechanical mixer, depending on the flow variations that can be provided by the in-line static mixer system, and their respective costs.
2. New flocculation facilities should provide 30 minutes of flocculation during peak flow. An arrangement that uses mechanical flocculation in a three-stage serpentine baffling pattern accomplishes this goal. This arrangement minimizes short-circuiting and assures good flocculation, even during periods of low flow.
3. Lamella plate clarification is the selected sedimentation alternative because of its compact footprint and because it is a similar process to the existing tube settlers.
4. Two new filters are proposed to match the expansion of the flocculation and sedimentation facilities. The two new filters will be of the same size as the existing filters so that filtration rates and backwash rates can be identical in all filters. The primary difference from the existing plant layout is that one filter will be located on each side of the pipe gallery. The compact size of the lamella plate clarifier provides room to locate a filter on the south side of the pipe gallery, opposite existing Filters No. 1 and 2.

The proposed 4.3-MG clearwell, for which design drawings have already been completed, is sufficient for the expected increased capacity.

Exhibit 5-5 was developed to show how these facilities would fit on the site. Although the site is limited, it appears that these facilities can be accommodated. This expansion would double the existing capacity to 10.4 mgd gross production or approximately 10.0 mgd net production. This expansion does not reach the goal of using the full supply potential of 12.0 mgd; however, it maximizes the use of the existing infrastructure and provides the most cost-effective expansion to 10 mgd. If the city finds it necessary to capture the remaining potential of 2 mgd, it may be necessary to install a second parallel plant. However, the city may also find that the expansion described herein can be operated at 12 mgd for short periods.

It is recommended that the expansion be constructed in one phase to minimize costs. The flocculation and clarification basins in particular do not lend themselves to a phased expansion. Using CH2M HILL's in-house WTP cost estimating tool (called CPES), the order of magnitude estimate for this expansion is $7.45 million in January 2005 dollars. This total includes an allowance for engineering design, engineering services during construction, and start-up services. It also includes an allowance for permitting, although this is expected to be minimal. It is recommended that the city further define the project and confirm the estimate prior to allocating funds and initiating the project. At $7.45 million and for an increase of 5.0 mgd, this yields a cost per gallon per day of increased capacity of approximately $1.50. A new plant might be expected to cost $2.0 to $2.50 per gallon per day.

EXHIBIT 5-5
Wicks WTP Expansion Concept
The Dalles Water Master Plan



CHAPTER 6

Transmission Pipelines

This section presents findings and recommendations for the system's transmission pipelines. The system has one raw water transmission pipeline, the Dog River Diversion Pipeline. The system has two finished water transmission pipelines, which deliver treated water from the Wicks WTP to the city distribution system. The transmission pipelines are in need of replacement because of their physical condition and to increase their capacity.

Dog River Diversion Transmission Pipeline

This section describes the concepts and design criteria for replacement of the Dog River Diversion Pipeline (Dog River Pipeline).

The Dog River Pipeline was constructed in the late 1800s or early 1900s to divert water from the Dog River watershed into the South Fork Mill Creek watershed. The city has an 1870 certificated water right that allows diversion of all water at the location of the diversion structure.

The city has long recognized that the water system CIP should account for replacement of this pipeline. It was originally constructed of wood stave but a portion has since been replaced with steel. Both sections, the wood stave and steel, warrant replacement, because their remaining useful life is limited and to increase the pipeline capacity.

Description of Existing Dog River Transmission Facilities

Exhibit 6-1 summarizes information for the Dog River Pipeline. It is located in the Mount Hood National Forest near Brooks Meadow, to the southwest of the Wicks WTP. It is not located within the National Gorge Scenic Area.

The upstream end of the pipeline connects to a diversion and headworks structure located on Dog River. The diversion dam is a low elevation, concrete structure. Removable stop logs are used to adjust the height of the pool held behind the dam. From the pool, water enters the pipeline diversion structure through a downward acting, manual slide gate and then through a bar screen. The concrete box behind the slide gate and upstream of the pipeline connection contains a level recording instrument (Campbell Scientific Data Recorder). This is used to monitor flows through the pipeline, as the level corresponds to the height of water flowing above a rectangular weir. The headworks facilities, except the inlet vault, are reported by city staff to be in good condition. They require only periodic maintenance, such as flow meter calibration, bar screen cleaning, and stop log replacement. City crews repaired one corner of the inlet vault that collapsed. They have noted that the concrete is beginning to break apart and has only limited rebar.

The pipeline is approximately 3.5 miles long. The original wood stave pipeline remains for approximately 2.5 miles of this length. The downstream 1 mile was replaced in 1974 with 20-inch-diameter steel pipe. Based on monitoring records, the city staff has estimated that the present carrying capacity of the pipeline is 8 mgd. A capacity of 8 mgd matches the calculated flow capacity if the following values are used: an internal diameter of 19 inches, a friction factor (Hazen-Williams C factor) of 90, and an available head of 200 feet.

Although the steel pipeline is newer, it is also recommended that it be replaced when the wood stave pipeline is replaced. The steel line is 30 years old. Its present condition and remaining useful life is unknown. It was installed where the pressures are highest because the wood stave pipe was leaking in this area. Replacement of the steel pipe is recommended so that this section does not become the problem area in the near future, which may occur if only the wood stave pipeline is replaced. Replacement of the steel pipe will also enable the city to meet its capacity goal. A third reason for replacement of both the steel and wood stave pipe sections is so that the existing diversion pipeline can remain in service during construction.

EXHIBIT 6-1
Dog River Pipeline Description

Item	Description
Pipeline materials, ages, diameters	Wood stave, 22-inch outside diameter and approximately 19-inch inside diameter (installed early 1900s): 2.5 miles; steel, 20-inch outside diameter (installed 1974): 1 mile
Total length	3.5 miles
Pipe capacity	8 mgd (12 cfs), approximately
Pipe condition	Unknown—no access or inspection ports along alignment
Pipeline inlet facilities	Entrance of river flow controlled by slide gate. Flow passes through bar screen into concrete basin. Pipeline connected to basin. Level in basin is monitored to indicate flow rate through pipeline.
Pipeline outlet facilities	Pipe discharges directly in creek bed. Large rocks cover the creek bottom and protect against erosion. Visible section of pipeline at outlet end is 18-inch-diameter clay tile. Uncertain how far the clay tile section extends upstream.
Hydraulics (elevations)	Inlet elevation is approximately 4,260 feet. Outlet elevation is approximately 4,060 feet. Pipeline does not follow uniform grade. It passes through a low point of 4,140 feet before passing over a high point at 4,240 feet. The resulting pressure at the low point is approximately 50 psi.
Access	A hiking and biking trail parallels much of the pipeline route. The trail is in the alignment of the road that was created during construction of the original pipeline.

Design Criteria for Replacement of Dog River Diversion Pipeline

The primary design criteria to consider for a replacement pipeline are location, capacity, and pipe material. **Exhibit 6-2** summarizes the suggested design criteria for the replacement pipeline. Permitting requirements are described in a subsection that follows.

It is recommended that the new pipeline be installed parallel to and within approximately 5 to 15 feet of the existing pipeline. This allows the existing pipeline to remain in service during construction and eliminates the cost of removing the existing pipeline. It is also possible that the existing wood stave pipeline would be considered an historic artifact and could not be removed without additional permitting.

EXHIBIT 6-2
Suggested Design Criteria for Dog River Replacement Pipeline

Item	Value or Description
Capacity	17 mgd
Diameter	24 inches
Material	Ductile iron
Length	3.5 miles
Pipeline inlet facilities	Replacement of the inlet vault and general maintenance is warranted, but no changes to the inlet configuration
Pipeline outlet facilities	Unchanged from existing
Alignment	Parallel to existing pipeline

The existing pipeline has a 65-foot construction right-of-way that can be reused for constructing the new pipeline. Following the existing alignment is expected to minimize property damage and provide the best opportunity for obtaining needed permits. Access to the site can be obtained on National Forest Road No. NFD 17, which crosses the pipeline alignment approximately 2,000 feet from the outlet.

The diversion of water from the Dog River watershed is to supplement the supply from the South Fork Mill Creek watershed. Based on the analysis of the two watersheds and a comparison with the city's desired peak day from the Wicks WTP of 12 mgd, it is recommended that the Dog River Pipeline provide a capacity of approximately 17 mgd (12,000 gpm). This is based on diverting nearly 100 percent of the flow every day of the month during a 50 percent exceedance year. (Fifty percent exceedance flows are by definition the flows that are equaled or exceeded in five out of ten years.)

Exhibit 6-3 summarizes the 50 percent exceedance estimates provided by the USGS from their monitoring records for Dog River. This table also includes a column showing these values multiplied by 1.5. This multiplier provides an approximate estimate of the peak day during the month, because the 50 percent exceedance values are provided as monthly averages. The 1.5 multiplier is a reasonable approximation for peak to average day for a given month based on historical records. Sizing the pipeline to divert the 50 percent exceedance values multiplied by 1.5 will enable the city to maximize the use of this source. Two factors support increasing the capacity from 8 mgd to 17 mgd: the incremental cost for a larger pipeline is relatively small and the city will have only this one opportunity to replace this pipeline for many years.

EXHIBIT 6-3
Dog River Flow Estimates and Desired Capture Values Monthly Averages in mgd

Month	USGS 50% Exceedance Estimate	1.5 x 50% Exceedance to Estimate Peak Day
January	2.7	4.1
February	4.9	7.4
March	3.7	5.6
April	3.7	5.6
May	9.0	14
June	11.0	17
July	4.6	6.9
August	2.5	3.8
September	1.7	2.6
October	3.0	4.5
November	1.5	2.3
December	1.6	2.4

The capacity goal of 17 mgd requires a pipeline diameter of 24 inches. A 24-inch-diameter pipeline, 3.5 miles long and for a head difference from inlet to outlet of 200 feet, will provide a maximum flow of approximately this amount. This sizing assumes that the pipeline can flow full, creating a vacuum to flow over the high spot of elevation 4,240 feet. A careful survey will be required to check the pipeline grade. It will be necessary to consider downstream control and evaluate vacuum conditions in the final design to ensure that the vacuum does not collapse the pipeline.

The pipeline material choices are ductile iron, concrete cylinder, high-density polyethylene (HDPE), and steel. HDPE pipe would work well in this application except for the limited access. HDPE is typically fusion-welded in long segments and then either pulled into the trench or lifted in with a side boom tractor. Either of the installation methods would require wide access roads along the open trench. Because wide access roads are not available, HDPE may not be a good choice. Concrete cylinder pipe requires field welding, requiring a high degree of installation quality control and supervision, which would be difficult to achieve in this remote application. Steel pipe requires field coating repair and, possibly, cathodic protection to minimize corrosion. The remote location does not fit well with these requirements. Pipe materials with short laying lengths, such as ductile iron, are better suited for the limited access that is available. Ductile iron provides the necessary pressure rating, a long service life, and requires no field welding or field-applied coatings. Therefore, ductile iron is the recommended material selection.

Costs were developed for both ductile iron and HDPE, because it may be possible to use HDPE based on further investigation into the right-of-way and site access. The estimates are summarized in **Appendix E**. At the time the cost estimates were developed, using a Seattle area *Engineering News-Record* (ENR) construction cost index (CCI) for November 2004, the cost estimate for HDPE pipe was approximately 10 percent higher than for ductile iron pipe. Costs for HDPE pipe are particularly volatile, depending on oil costs. The city should review the comparison between ductile iron pipe and HDPE pipe prior to project implementation to determine if there is a cost benefit to one material over the other. As an alternative, the city may wish to include both options in the design and make a final selection based on contractors' bids.

Schedule for Replacement of Dog River Diversion Pipeline

The schedule for replacement is primarily driven by the need to increase supply to the Wicks WTP, once the plant is expanded beyond 5 mgd. An outside factor that may dictate timing for replacement is the status of the proposed Mill Creek Buttes Wilderness designation. If it appears that the wilderness area will include the pipeline route and if the opportunity to replace the pipeline before the designation exists, it may be a sound investment to move forward with the project in the near term.

Permitting for Dog River Diversion Pipeline

The permitting requirements for the pipeline replacement are expected to be significant. In addition to being located in the Mount Hood National Forest, the downstream one-half mile of the pipeline route may lie within the proposed Mill Creek Buttes Wilderness. The southern edge of the proposed wilderness area, based on a review of the Oregon Natural Resources Council's web site, appears to be the trail that parallels the pipeline alignment. It is unclear if the pipeline alignment is within or outside of the proposed wilderness boundary.

Potential Permits and Approvals

Nearly all uses of National Forest lands must be authorized under a Special Use Permit. The city currently holds a Special Use Permit for the Dog River Pipeline that was issued in 1964. It has no expiration date. This permit will need to be modified to address the replacement project. The Forest Service will be required to comply with the National Environmental Policy Act (NEPA) in its determination of whether to authorize the improvements. If potential impacts are not suspected to be significant, the Forest Service will prepare an Environmental Assessment (EA) followed by a Finding of No Significant Impact (FONSI). If potential impacts are suspected to be significant, an environmental impact statement (EIS) will be required, followed by a Record of Decision (ROD). Given limited federal staff budgets and schedules, a third-party contractor is typically hired by the applicant, subject to approval by the Forest Service, to prepare the environmental documentation.

Five environmental/cultural issues have been identified based on a preliminary desktop evaluation of the project site:

1. Potential presence of spotted owl (a listed species protected by the Endangered Species Act) in the project vicinity.

2. The presence of wetlands (two areas are mapped) at Cook Meadow at the upstream segment of the pipeline corridor.

3. The potential presence of other listed plant or wildlife species in the project vicinity (particularly associated with the meadow).
4. The potential historic status of the wood-stave pipeline.
5. The potential intrusion into a proposed wilderness area.

Further project clarification, coordination with the Forest Service, and field surveys will be required to determine the magnitude of these issues. The type of NEPA documentation required depends on the findings of the studies and the ability to avoid or minimize impacts to potential sensitive resources.

Should a wetland crossing be unavoidable, a joint removal/fill permit will be required from the U.S. Army Corps of Engineers (USACE) and the Division of State Lands (DSL).

If the Forest Service determines that the project may affect a listed species, the city will be required to obtain an incidental take permit from the U.S. Fish and Wildlife Service as per the Endangered Species Act. Only 12 feet of the 65 feet construction right-of-way is currently maintained as a roadway. Construction of the replacement pipeline will likely require removal of trees within the right-of-way, resulting in possible impacts to habitat of the spotted owl. These impacts may be limited by timing the construction to occur at times other than the breeding season.

If there are plans to remove the existing pipeline, the Forest Service will need to comply with Section 106 of the Historic Preservation Act. This law requires that buildings and structures over 50 years old be assessed for their eligibility for listing on the National Historic Register. If the old pipeline is determined to be eligible, then a "Determination of Effect" is required to assess and mitigate impacts.

Intrusion into the proposed wilderness boundary is not expected to be a major environmental issue. The regulations of the Wilderness Act of 1964 allow existing reservoirs, ditches, water catchments, and related facilities for the control or use of water to be maintained or reconstructed if they meet a public need, or are part of a valid existing right. Motorized equipment and mechanical transportation for maintenance of water development structures is not allowed unless practiced before the area was designated wilderness or unless it is determined to be the minimum necessary tool or technique. Given that the pipeline is an existing use, that improvements would occur in the immediate proximity, and that the use predates any wilderness designation, obtaining approval to encroach onto wilderness lands is not expected to be a significant issue.

Permitting Schedule and Cost

The city would be prudent to initiate discussions with the Forest Service about a new Special Use Permit at least 3 years in advance of construction. The Forest Service would be able to provide early information on the project area that would help the city to avoid or minimize the extent of issues to be addressed in the Special Use Permit approval process. Any required environmental surveys should be initiated within 2 years of construction. Sensitive areas should be mapped prior to developing alternative pipeline alignments. Conceptual designs of alternative alignments should be developed with the cooperation of an environmental specialist and the Forest Service to further avoid or minimize impacts.

Depending on the extent of issues realized, the permit and approval process can take between 1 and 2 years.

The cost of environmental permitting will vary based on the findings of biological surveys, the ability to avoid sensitive resources, and the extent of the role that the Forest Service will take in addressing NEPA. The cost for permitting may range from $75,000 to $250,000. An allowance of $150,000 has been included in the CIP for this project.

Estimated Cost for Dog River Diversion Pipeline Replacement

The estimated project cost for replacing the Dog River Diversion Pipeline using ductile iron pipe is $3,450,000. This includes an allowance of $150,000 for environmental permitting and an allowance of $300,000 (10 percent of construction) for engineering design and construction services. The construction estimate includes a 20 percent contingency.

Appendix E provides the background for the construction estimate. It is an order of magnitude-level estimate, as described in the table.

Finished Water Transmission Pipelines

Two finished water transmission pipelines deliver water from the Wicks WTP to the city distribution system. This section presents an evaluation of these two pipelines and recommendations for their replacement.

Description of Existing Finished Water Transmission Pipelines

Treated water from the Wicks WTP plant flows into the existing 320,000-gallon clearwell tank, located just to the north edge of the treatment plant site. Two finished water transmission pipelines connect to the outlet of this clearwell and transmit water approximately 7 miles north to the city limits. One is called the High Line and the other the Mill Creek Line. In the future, when the new plant clearwell is constructed as described in the section on the WTP, the two lines will connect to the outlet of that tank. The new clearwell will have the same water surface elevation as the existing clearwell.

The pipelines parallel each other and are located along Mill Creek for the first approximately 4.5 miles from the plant. At this point, the High Line alignment turns northeast and runs across private and public lands on a mostly direct route to Sorosis Reservoir, which is located in Sorosis Park. The Mill Creek pipeline continues along the Mill Creek Road right-of-way to 16th Street, just west of Skyline Road, where its alignment turns east. This pipeline connects to Garrison Reservoir.

In addition to supplying water to the city's distribution system, each transmission line serves a limited number of customers on properties adjacent to the pipeline alignments. These services were generally granted many years ago in exchange for having the customers provide pipeline easements.

Exhibit 6-4 summarizes data for the existing pipelines. Their combined capacity is approximately 5,200 gpm (7.5 mgd).

Both pipelines are operated in an open-channel mode by controlling the flow rate that enters the pipelines from the upstream end. If the upstream valve was fully opened and a downstream valve was closed or partially closed so that either pipeline was allowed to flow under pressure, the pipe joints in high pressures areas would pull apart or rupture. This limitation prevents the city from using the clearwell storage as a backup to Sorosis Reservoir. The High Line cannot be operated as a pressurized line that responds to downstream demands, as would be necessary if Sorosis Reservoir was removed from service.

EXHIBIT 6-4
Description of High Line and Mill Creek Transmission Pipelines

Item	Description of High Line	Description of Mill Creek Pipeline
Age	Approximately 60 years	Approximately 60 years
Length	36,000 feet	37,000 feet
Diameter and material	Original line is 14-inch welded steel; some sections have been replaced with 20-inch ductile iron.	Original line is 12-inch welded steel; some sections have been replaced with 12-inch ductile iron.
Hydraulics	The overflow for the existing clearwell is 885.4 feet elevation. (The new clearwell will have the same overflow elevation.) Sorosis Reservoir has an overflow elevation of 660 feet. The low point on the existing High Line alignment is at an elevation of approximately 400 feet. Therefore, the maximum static pressure is 210 psi.	The overflow for the existing clearwell is 885.4 feet elevation. (The new clearwell will have the same overflow elevation.) Garrison Reservoir has an overflow elevation of approximately 460 feet. The low point on the existing Mill Creek pipeline alignment is at an elevation of approximately 400 feet. Therefore, the maximum static pressure is 210 psi.
Capacity	The maximum recorded flow to date has been 2,240 gpm (3.2 mgd). The calculated maximum capacity is approximately equal to this value.	The maximum recorded flow to date has been 1,790 gpm (2.6 mgd). The higher available head (compared to High Line) results in a calculated capacity of 3,000 gpm.
Condition	Condition of the interior is generally unknown. Pipe has no access or inspection ports. Pipe is visible on surface of ground at several locations. Corrosion pitting of exterior is minor. Pipe is not protected by a cathodic protection system, which suggests that there may be significant loss of wall thickness, especially in areas along creek that have wet, corrosive soils.	Condition of the interior is generally unknown. Pipe has no access or inspection ports. Pipe is not protected by a cathodic protection system, which suggests that there may be significant loss of wall thickness, especially in areas along creek that have wet, corrosive soils.
Operation	Pipeline is not capable of flowing under pressure. The flow rate must be controlled from the upstream end so that pipe flows in open channel mode.	Pipeline is not capable of flowing under pressure. The flow rate must be controlled from the upstream end so that pipe flows in open channel mode.
Access	Upstream 2 miles is located on hillside away from road, next 2.5 miles is near roadway, but is not directly accessible from road. Downstream 2.5 miles crosses private property, making access difficult.	Pipe alignment is near Mill Creek Road, but most of upper half is located in private fields and is difficult to access. Lower half is along road and for the most part is accessible.

Replacement Recommendations for Finished Water Transmission Pipelines

It is recommended that the city include replacement of the two finished water transmission pipelines in the 20-year CIP for the following reasons:

1. Pipeline condition. Both pipelines are steel and have been in place for about 60 years. The city has replaced some sections because of joint failures and resulting leaks. The pipeline conditions are unknown but it appears likely that corrosion-caused leaks and joint failures will occur periodically. Each pipeline crosses Mill Creek in five locations. City staff report that nine of these ten crossings were damaged during the 1996 flood.
2. Capacity. The two existing lines provide a combined capacity of approximately 7.5 mgd, if operated under pressure. The city's long-range plan is to obtain up to 12 mgd from the Wicks WTP supply.
3. Backup to Sorosis Reservoir. The existing High Line is not capable of operating under pressure, as would be necessary to provide backup to Sorosis Reservoir if it was removed for maintenance. Recent inspections have found significant corrosion in this steel reservoir; therefore, the city needs to remove it from service in the near future to repaint the tank.

Of these three needs, the most pressing is the third. City staff considers removing Sorosis Reservoir from service to repaint the tank an urgent need. However, this need may also be addressed by the addition of a 760-foot elevation service zone, as described in Chapter 8, Distribution System Analysis. Installation of the new distribution tank appears to be the preferred alternative. The condition of the pipelines will result in leaks and minor failures, but does not in itself warrant immediate replacement. Their present capacity of 7.5 mgd is sufficient until the dam and WTP are expanded.

A primary question for replacing the two pipelines is whether to replace them with two separate pipelines or with a single line. In the past, the city has considered replacing them with a single pipeline. However, during the floods of 1996, both pipelines were damaged at creek crossings, resulting in breaks that required repair. This caused the city to rethink using just a single line from the Wicks WTP to the city distribution system because it would not provide redundancy.

CH2M HILL believes that a single line will provide acceptable reliability and is favored for achieving cost savings because of the following reasons:

- Although a single failure would interrupt flow, the flood of 1996 demonstrated that under severe conditions, it is possible for both lines to be lost at one time.
- The pipelines are most vulnerable at creek crossings. Whether the pipelines are replaced with a single line or two lines, they will need to be armored to withstand flood conditions at these crossings. It is not overly expensive to design these crossings conservatively to make it unlikely that a flood will damage the pipeline. With this extra precaution, a single line will provide greater reliability than is currently provided by the two pipelines with less-armored creek crossings.
- The city's water system includes three wells located within the distribution system. These wells, with a combined capacity of 7.0 mgd, are capable of providing an

emergency backup in case of a pipeline failure. The CIP includes expansion of the groundwater capacity.

- The cost for two separate pipelines may be as much as 175 percent the cost of a single pipeline that provides equal capacity.

Design Criteria for Finished Water Transmission Pipelines

Exhibit 6-5 summarizes the recommended design criteria for the replacement pipeline. A single, 24-inch-diameter, ductile iron pipeline is recommended. This would provide a flow capacity of approximately 12.5 mgd. It would follow a similar alignment to the existing Mill Creek pipeline to avoid private property.

The pipeline material selection is limited to ductile iron pipe because of the required pressure rating and diameter. Steel pipe was not considered a favorable option because of the coatings requirement and trench widths required to perform joint welding. Concrete cylinder is not recommended because it is necessary to have all fittings custom-made. This introduces more risk during construction, because some bends may not fit actual field conditions. Ductile iron pipe provides strength, durability, and the ability to convey flows at extreme pressures. The shorter laying lengths of ductile iron pipe will allow the pipeline to be laid through curves in the roadway without the use of large elbow fittings. Each joint can be deflected several degrees to match the curves of the road right-of-way. Ductile iron pipe can be directly tapped to provide service to customers along the transmission route, as needed.

EXHIBIT 6-5
Recommended Design Criteria for Replacement Finished Water Transmission Pipeline

Item	Value
Number of pipelines	One
Diameter	24 inches
Material	Ductile iron
Pressure rating	Varies along length; high pressure sections will require 300 psi rating
Alignment	Parallel to existing Mill Creek Pipeline in public right-of-way.
Length	40,200 feet (7.61 miles)
Permitting	Wasco County Road Department for locating the new pipeline on Mill Creek Road. USACE/DSL for joint fill/removal permits for 5 stream crossings. Wetland area construction/mitigation.

City staff has indicated a desire to construct the new pipeline in public right-of-way to provide unrestricted access for maintenance and repair. Mill Creek Road is a Wasco County public right-of-way and runs almost the entire length of the proposed alignment. Reservoir Road could be used for pipe installation from the Wicks WTP to Mill Creek Road. On the north end of the pipeline, a short portion (650 feet) of Skyline Road could be used to install pipe that would then turn east and run approximately 3,000 feet along an east/west property line to the Sorosis Reservoir. There may be a possibility for power generation for the supply into Garrison Reservoir.

Pipe installation along the county road will require five creek crossings that would be constructed parallel to the roadway. Two of the crossings are along Reservoir Road and the remaining three are along Mill Creek Road.

Two 8-inch branch lines would need to be constructed to continue to provide service to customers along the old High Line route. It is proposed that these branch lines would be constructed on Orchard Road and Skyline Road. Each line would be approximately one-half mile in length and reconnect to the old High Line where it currently crosses these roads. The city could explore four options for the service line on the High Line alignment: 1) a new pipeline could be constructed parallel to the old line, 2) a portion of the old line could be kept live, 3) a new HDPE service pipeline could be inserted through the old steel line and then reconnected to the services along the route, or 4) service could possibly be eliminated to these customers because the easement would no longer be required (this option requires a legal/administrative analysis).

The Wasco County Road Department would need to approve and permit construction of the new 24-inch pipeline in the Mill Creek Road right-of-way.

Estimated Cost for Finished Water Transmission Pipelines Replacement

The estimated project cost for replacing the existing High Line and Mill Creek Finished Water Pipelines with a single new Finished Water Pipeline is $10,050,000. This includes an allowance of approximately $900,000 (10 percent of construction) for engineering design and construction services. The construction portion of the estimate includes a 20 percent contingency.

Appendix F provides the background for the construction estimate. It is an order of magnitude-level estimate, as described in the table.

CHAPTER 7

Regulatory Review

Community water systems are governed by rules developed by the U.S. Environmental Protection Agency (EPA) for implementation of the Safe Drinking Water Act Amendments. Oregon, as a primacy state, is required to implement water quality regulations at least as stringent as EPA's rules. For the most part, Oregon has adopted identical regulations to those at the federal level. Several additional Oregon rules are highlighted in this section.

The Dalles' water system complies with all current state and federal standards. Both current standards and proposed future standards are discussed within this section according to the following categories:

- Surface water treatment regulations
- Groundwater treatment regulations
- Distribution regulations

The most significant impact of proposed future standards appears to be the need to add clearwell storage to the water treatment plant to achieve compliance with the Stage 2 Disinfection By-Product (DBP) Rule.

The Dalles Water Quality Goals and Water Quality Achievements

Exhibit 7-1 lists The Dalles' water quality goals and the state and federal standards for these parameters. Through its success in meeting these goals and additional criteria, The Dalles has been granted the Director's Award for completion of the Phase III program of the Partnership for Safe Water, a voluntary quality assurance program instituted by EPA and the American Water Works Association.

The goal of the Partnership is cooperation among the regulatory agencies, professional organizations, and utilities to provide a new measure of safety by implementing prevention programs where legislation or regulation does not exist. The preventive measures are based on optimizing treatment plant performance. The Partnership's program includes the following elements:

- A commitment to continued compliance with existing surface water treatment regulations
- Completion of a self-assessment water quality report
- Identification and implementation of operational improvements to optimize treatment
- Meeting finished water turbidity goals
- Submission of an annual report outlining the utility's continued efforts and results in optimizing treatment

The Dalles' system is the only water treatment plant in Oregon to achieve the Director's Award, and one of only 48 utilities nationwide to receive the Five-Year Director's Award.

EXHIBIT 7-1
The Dalles Water Quality Goals

Parameter	State/Federal Standard	The Dalles Standard	Comments
Filtered water turbidity	< 0.3 NTU 95% of the time. Never to exceed 1.0 NTU.	< 0.1 NTU at all times	The Dalles' goal is consistent with the Partnership for Safe Water standard.
Chlorine residual: entrance to system	Not < 0.2 mg/L for > 4 hours	1.1 – 1.3 mg/L (free chlorine)	The Dalles' standard is for water leaving the plant; i.e., entering the transmission lines. State/federal standard is for entry into distribution system or first customer. If The Dalles maintains 1.1 – 1.3 mg/L leaving the plant, this will meet disinfection standards.
Distribution chlorine residual	Cannot be undetectable in > 5% of samples per month	> 0.5 mg/L free chlorine	This standard is designed to limit biological regrowth in the distribution system.
pH	> 6.8	7.0 – 7.2	State and federal standard is to ensure compliance with the Lead and Copper Rule.
Heterotrophic plate count (HPC)	< 500 cfu/mL	≤ 1 cfu/mL	City responds to a result of > 1 cfu/mL by flushing and re-sampling.
Phosphates	> 0.4 mg/L	0.45 – 0.60 mg/L	To ensure compliance with the Lead and Copper Rule; measured as PO_4.

Surface Water Treatment Regulations

Maximum contaminant levels (MCLs) have been established by EPA for more than a hundred individual drinking water contaminants. These include microbiological, inorganic, organic and radiological contaminants. The Dalles' water is in compliance with each of these standards.

- *Interim Enhanced Surface Water Treatment Rule* (IESWTR, promulgated December 16, 1998; final revisions published January 16, 2001).
- *Long-Term 1 Enhanced Surface Water Treatment Rule* (LT1ESWTR, promulgated January 14, 2002).
- *Long-Term 2 Enhanced Surface Water Treatment Rule* (LT2ESWTR, proposed rule published August 11, 2003).

Interim Enhanced Surface Water Treatment Rule

The IESWTR was promulgated on December 16, 1998. This rule builds on the provisions set forth in the Surface Water Treatment Rule (SWTR) by providing improved public health protection against *Cryptosporidium,* while addressing risk tradeoffs with DBPs. The IESWTR applies to public water systems such as The Dalles that use surface water and serve at least 10,000 people. EPA published final revisions to the IESWTR on January 16, 2001. Primacy states, such as Oregon, were to have adopted the regulation by January 1, 2002. Public water systems are required to achieve compliance within 3 years of federal promulgation.

Specific provisions of the IESWTR include:

- Maximum contaminant level goal (MCLG) of zero for *Cryptosporidium*
- 99 percent *Cryptosporidium* removal requirements for systems that filter
- Strengthened combined filter effluent turbidity performance standards for systems using conventional and direct filtration
- Individual filter turbidity monitoring provisions for systems using conventional and direct filtration

Treatment plants such as the Wicks WTP that use conventional filtration are assumed to meet the 99 percent *Cryptosporidium* removal requirement as long as they comply with the IESWTR turbidity requirements and existing provisions of the SWTR. A system's combined filter effluent turbidity is required to be less than 0.3 NTU in at least 95 percent of samples taken each month, and at no time may exceed 1 NTU. Utilities must conduct continuous monitoring of turbidity for each filter. The Dalles complies with all of these requirements.

Long-Term 1 Enhanced Surface Water Treatment Rule

The final LT1ESWTR, promulgated on January 14, 2002, extends the requirements contained in the IESWTR to small surface water systems that provide service to populations under 10,000 persons. The LT1ESWTR requires small systems to comply with the same *Cryptosporidium* removal and filter turbidity performance standards as those established by the IESWTR.

Long-Term 2 Enhanced Surface Water Treatment Rule

The purpose of the LT2ESWTR is to build on the provisions contained in the IESWTR for protection of public health against risks posed by *Cryptosporidium* and other microbial pathogens. When finalized, the LT2ESWTR will apply to all public water systems that use surface water. This rule will also require source water monitoring of *Cryptosporidium* for systems such as The Dalles that serve more than 10,000 people. The proposed LT2ESWTR was published in the Federal Register on August 11, 2003, with promulgation of the final rule on December 15, 2005.

When promulgated, the LT2ESWTR will supplement existing regulations by targeting additional *Cryptosporidium* treatment requirements to higher risk systems. Existing drinking water regulations established in the IESWTR and LT1ESWTR require water systems such as The Dalles that filter surface water to achieve at least a 2-log removal of *Cryptosporidium*. New data on *Cryptosporidium* infectivity, occurrence, and treatment indicate that current treatment requirements are adequate for the majority of systems, but there is a subset of systems with higher vulnerability to *Cryptosporidium* where additional treatment is necessary.

Under the proposed LT2ESWTR, systems must begin source water monitoring for *Cryptosporidium* within 6 months of promulgation to determine their treatment requirements. Filtered systems will be classified into one of four risk bins based on results of source water monitoring. The regulation specifies a range of treatment and management strategies collectively termed the "microbial toolbox" that systems may select from to meet any additional treatment requirements that are specified in their bin classification.

Cryptosporidium monitoring by large systems like The Dalles will begin within 6 months after the LT2ESWTR is finalized and will have a scheduled duration of 2 years. Systems must conduct a second round of monitoring beginning 6 years after the initial bin classification. A water system may grandfather equivalent previously collected data in lieu of conducting new monitoring, and will not be required to monitor if it provides the maximum level of treatment required under the rule.

Exhibit 7-2 lists the bin classifications according to *Cryptosporidium* concentrations in the source water.

EXHIBIT 7-2
Additional *Cryptosporidium* Treatment Requirements for Filtered Systems

Mean *Cryptosporidium* Source Water Concentrations	Bin Classification	Required Additional[1] Log Reduction for Conventional Filtration WTPs
Crypto < 0.075/L	Bin 1	No Additional Treatment
0.075/L ≤ *Crypto* < 1.0/L	Bin 2	1
1.0/L ≤ *Crypto* < 3.0/L	Bin 3	2
Crypto ≥ 3.0/L	Bin 4	2.5

1. Treatment in addition to filtration.
2. For 1 additional log removal/inactivation, systems may use any technology or combination of technologies from the Microbial Toolbox.
3. For additional 2 or greater log removal/inactivation, systems must achieve at least 1 log of the required treatment using ozone, chlorine dioxide, UV, membranes, bag/cartridge filters, or bank filtration.

Exhibit 7-2 indicates that no additional treatment to as much as 2.5 logs of additional *Cryptosporidium* removal/inactivation may be required at the Wicks WTP, depending on the level of *Cryptosporidium* that is detected in the source water supply.

To date, The Dalles has conducted one sample analysis for *Cryptosporidium* in the raw water. This test was conducted in December 1993. No *Cryptosporidium* organisms were found at a detection limit of 5.3 organisms per 100 mL. Because it is only a single result and the detection limits for testing have since been lowered, it is not possible to know what bin category the city's source water will be assigned. Based on limited *Cryptosporidium* monitoring at other locations in the Pacific Northwest, it is reasonable to expect that the source classification for the South Fork Mill Creek will be either Bin 1 or Bin 2.

If The Dalles' source water monitoring places them in Bin 1, no additional credit is necessary, and therefore no treatment changes are necessary. If The Dalles' monitoring places them in Bin 2, The Dalles will need to document or employ additional measures to achieve credit for another 1 log removal. One of the measures identified in the microbial toolbox is 1 log credit if each individual filter achieves < 0.1 NTU in 95 percent of daily maximum values, with no reading above 0.3 NTU. This criterion aligns with one of the city's existing water quality goals, one that the city consistently meets. Therefore, it appears that The Dalles can claim credit for its individual filter performance to achieve an additional 1 log removal if the raw water sampling places the city in Bin 2. The rule would only require minor adjustments in monitoring and reporting.

If the city's raw water sampling places it in Bin 3 or 4, additional treatment such as the use of ultraviolet (UV) disinfection will be necessary to achieve compliance with the rule. It is not expected that water from the South Fork Mill Creek will result in a Bin 3 or 4 classification.

If additional treatment measures are necessary, the changes must be implemented within 6 years following promulgation of the final LT2ESWTR. States may grant an additional 2 years for compliance for systems that are undertaking capital improvements.

Groundwater Rule

The Dalles also obtains a portion of its water supply from wells. These are subject to the requirements of EPA's proposed Groundwater Rule. The draft rule was published in May 2000. It is now projected to become final in 2006.

The rule indicates that a minimum level of disinfection will need to be provided if a well is hydrogeologically sensitive, or subject to microbiological contamination. The minimum level of disinfection, if required, will necessitate both the application of chlorine and the provision of several minutes of contact time between the chlorine application and the first customer. The city currently chlorinates at the wellheads, which is required because the water is mixed with a chlorinated surface water supply. However, the wells feed directly into the distribution pipe network and there is no provision for storage to achieve contact time.

It does not appear that this rule will impact the city's operation of its wells. The 1999 groundwater study indicated that none of the three wells is hydrogeologically sensitive. In addition, the city has performed bacteriological monitoring of the water from each well, prior to chlorination, for several years and coliform tests have indicated the absence of organisms. However, the Groundwater Rule may require additional testing to confirm that the wells are not hydrogeologically sensitive.

Distribution Regulations

The Dalles complies with current distribution regulations but may require changes to comply with the proposed Stage 2 DBP Rule.

- *Interim Enhanced Surface Water Treatment Rule*
- *Total Coliform Rule*
- *Lead and Copper Rule*
- *Disinfection By-Product Rule*

Two proposed new rules will regulate distribution water quality:

1. Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)
2. Stage 2 Disinfection By-Product Rule (Stage 2 DBP Rule)

In addition, Oregon's drinking water regulations have requirements that indirectly relate to water quality, including backflow prevention program rules, operator certification rules, and product acceptability criteria.

In general, the state's rules govern the quality of water and not the manner in which it is distributed. However, the rules do contain a limited number of standards for disinfection, and storage and piping criteria:

- Distribution piping shall be designed and installed so that the pressure measured at the property line of any user shall not be reduced below 20 psi (OAR 333-061-0050(9)(e)).
- The residual disinfectant concentration in water entering the distribution system cannot be less than 0.2 mg/L for more than 4 hours (OAR 333-061-0032(5)(b)).
- The residual disinfectant concentration in the distribution system cannot be undetectable in more than 5 percent of samples each month, for any two months (OAR 333-061-0032(5)(c)).
- Wherever possible, dead ends shall be minimized by looping. Where dead ends are installed, blow-offs of adequate size shall be provided for flushing (OAR 333-061-0050(9)(h)).
- Wherever possible, distribution pipelines shall be located on public property. Where pipelines are required to pass through private property, easements shall be obtained from the property owner and shall be recorded with the county clerk (OAR 333-061-0050(9)(a)).
- Wherever possible, booster pumps shall take suction from reservoirs to avoid the potential for negative pressures on the suction line, which could result when the pump suction is directly connected to a distribution main. Pumps that take suction from distribution mains shall be provided with a low-pressure cutoff switch on the suction side set at no less than 20 psi (OAR 333-061-0050(8)(a, b)).

The state's rules also include construction standards that must be met when new projects are designed and constructed. Construction standards are found in OAR 333-061-0050.

Surface Water Treatment Rules

The original SWTR was promulgated in June 1989. It consists of filtration requirements, primary and secondary disinfection requirements, and monitoring requirements. The secondary disinfection requirements are the one aspect that relates to distribution water quality. It requires that the residual disinfectant concentration in the water entering the distribution system not be less than 0.2 mg/L for more than 4 hours and that the residual disinfectant concentration in the distribution system cannot be undetectable in more than 5 percent of the samples each month for two consecutive months. Water in the distribution system with a heterotrophic bacteria concentration less than or equal to 500 cfu/mL is deemed to have a detectable disinfectant residual.

The Dalles is in compliance with the requirements of the SWTR. The Dalles currently chlorinates such that water being pumped from the clearwell into the finished water transmission pipelines has a free chlorine residual of 1.1 - 1.3 mg/L. This level of chlorine residual results in a range of residuals at the extreme ends of the system that is typically ≥ 0.5 mg/L. In addition, the city consistently meets its water quality goal of ≤ 1 cfu/mL for heterotrophic bacteria, which is much lower than the standard of 500 cfu/mL.

Interim Enhanced Surface Water Treatment Rule

The IESWTR was promulgated on December 16, 1998. This rule builds on the provisions set forth in the SWTR by providing improved public health protection against *Cryptosporidium,* while addressing risk tradeoffs with DBPs. The IESWTR applies to public water systems such as The Dalles that use surface water and serve at least 10,000 people. EPA published final revisions to the IESWTR on January 16, 2001. The compliance deadline was January 1, 2002. The primary impact of the rule on The Dalles was to increase monitoring and reporting.

Long-Term 2 Enhanced Surface Water Treatment Rule

The purpose of the LT2ESWTR is to build on the provisions contained in the IESWTR for protection of public health against risks posed by *Cryptosporidium* and other microbial pathogens. The proposed LT2ESWTR was published in the Federal Register on August 11, 2003, with promulgation of the final rule on December 15, 2005. This rule has only minor implications for distribution water quality and has not significantly impacted The Dalles.

Total Coliform Rule

The Total Coliform Rule (TCR) was promulgated in June 1989 with the primary goal of maintaining microbial quality in finished and distributed drinking water supplies. Total coliform includes both fecal coliform and *E. coli.* The MCLG for total coliform was set to zero. Compliance with the MCL is based on the presence or absence of total coliform in a sample (as opposed to coliform density as in previous rules). The Dalles is required to collect a minimum of 10 samples per month, based on its service population.

The Dalles has complied with the TCR since its promulgation.

Lead and Copper Rule

The Lead and Copper Rule was promulgated in June 1991 and went into effect in December 1992, with minor revisions released in April 2000. The rule applies to all community water systems. The rule developed MCLGs and action levels for both lead and copper in drinking water. The major difference between this regulation and other distribution regulations is that the water must be monitored at customers' taps, not at sampling stations. Lead and copper monitoring must initially occur every 6 months and twice each calendar year at locations with the highest risk of contamination resulting from the following:

- Piping with lead solder installed after 1982
- Lead water service lines

- Lead piping in buildings and homes

For compliance, the samples at the customers' taps must not exceed the following action levels:

- Lead concentration of 0.015 mg/L detected in the 90th percentile of all samples
- Copper concentration of 1.3 mg/L detected in the 90th percentile of all samples

The Dalles has consistently complied with the Lead and Copper Rule by using a dual-method approach: pH adjustment to 7.0 – 7.2, and the addition of a phosphate corrosion inhibitor. Lead and copper sample results for 1993 through 2003 are summarized in **Exhibit 7-3**. The 90th percentile lead values are typically non-detectable at a 0.002 mg/L detection limit. The highest 90th percentile value was 0.0024 mg/L, significantly below the action level of 0.015 mg/L. The highest 90th percentile copper value was 0.60 mg/L, significantly below the action level of 1.3 mg/L.

Because of compliance with the lead and copper action levels, The Dalles is on a reduced sampling schedule, which includes 30 houses every 3 years instead of 60 houses every 6 months.

EXHIBIT 7-3
Lead and Copper Monitoring Results

Action Levels:		**Lead = 0.015 mg/L**
		Copper = 1.3 mg/L
Monitoring Period	**90th Percentile Lead**	**90th Percentile Copper**
2nd half 1993	< 0.002	0.51
1st half 1994	< 0.002	0.47
Summer 1995	< 0.002	0.19
Summer 1996	< 0.002	0.60
Summer 1997	0.0024	0.46
Summer 2000	< 0.002	0.31
Summer 2003	< 0.005	0.28

Stage 2 Disinfection By-Product Rule

The Stage 2 Disinfection By-Product Rule (Stage 2 DBPR) was proposed by EPA on August 18, 2003 and was promulgated December 15, 2005.

The purpose of the rule is to reduce peak DBP concentrations in the distribution system and eliminate areas where customers receive excessive levels of DBPs. Levels of DBPs, which fluctuate based on changes in raw water quality, treatment changes, chlorine levels, and water age, have been found to vary geographically in distribution systems. The current rules governing DBPs determine compliance based on an average for samples collected throughout the distribution system. This averaging means that it is possible for some geographic locations to occasionally or even regularly exceed the MCLs for DBPs, and yet the system remains in compliance. The Stage 2 DBPR eliminates this possibility by requiring compliance at all geographic locations.

The rule requires the following:

1. Completion of an initial distribution system evaluation (IDSE) to determine sites with high DBPs. This evaluation report is due 2 years following promulgation of the final rule. It can be conducted by performing a Standard Monitoring Plan consisting of increased monitoring for total trihalomethanes (TTHMs) and five regulated total haloacetic acids (HAA5s), or by performing a System Specific Study that includes extended period hydraulic modeling to help determine worst-case sites for monitoring.

2. Compliance with the MCLs for TTHMs and HAA5 of 80 and 60 μg/L, respectively, based on a locational running annual average (LRAA). Average concentrations of TTHMs and HAA5s at each sampling site must comply with the MCLs. Compliance will be in two stages. Stage 2A allows for relaxed MCLs at each location. Stage 2B, which is proposed to begin 6 years following promulgation, will require compliance with the current MCLs of 80 μg/L for TTHMs and 60 μg/L for HAA5s at all locations.

Exhibits 7-4 and 7-5 summarize recent DBP levels measured in The Dalles' system for 2002 through 2004. The maximum system-wide TTHM running annual average for this period was 30 μg/L. This is well below the MCL of 80 μg/L. The highest single LRAA was 34 μg/L, also well below the MCL. Current data suggest that the existing system will comply with the more stringent LRAA MCL for TTHMs.

The maximum system-wide HAA5 running annual average was 46 μg/L. The maximum single LRAA was 56 μg/L. These values are not much below the MCL of 60 μg/L. It is uncertain that The Dalles can remain in compliance with the proposed Stage 2 DBP rule for HAA5, which requires that the MCL of 60 μg/L be met as a LRAA at the worst-case sites within the distribution system. The city's IDSE will include monitoring of eight sites, which may identify locations within the distribution system that experience higher levels of HAA5 than currently measured. Following the IDSE, the city must continue to monitor four sites within the system including the sites identified as worst-case.

As described in Chapter 5, one of the WTP near-term improvements is the addition of a larger clearwell. This will enable the city to change from pre- to post-chlorination while still maintaining compliance with the disinfection contact time requirements. This change is expected to reduce DBPs. Without a new clearwell, developing additional storage within the distribution system or using all existing storage during low-flow months, as proposed in this master plan, will increase water age and associated DBP potential, thereby adding to Stage 2B compliance challenges. It is assumed within this master plan that the change in chlorination location because of the new clearwell will enable the city to comply with the Stage 2 DBP Rule. However, this will not be known until the city makes this improvement, completes the IDSE, and determines HAA5 levels based on these changes.

Possible Future Regulations of Interest

Although planning for future system improvements is based on ensuring compliance with current and pending regulations, the regulatory climate is ever-changing and uncertain. The following potential regulatory changes could impact The Dalles within the 20-year planning period:

- Distribution System Rule, promulgation date unknown, is expected to revise the TCR and affect distribution system operations, including reservoir operation and mixing. It may require capital investments to modify reservoirs for better mixing.
- Lead and Copper Rule revisions may impact monitoring frequency or reduce action levels.
- Clean Water Act-related regulations may set limits on chlorine concentration and temperature of backwash discharges, which may force The Dalles to recycle backwash water to nearly eliminate backwash discharge.

EXHIBIT 7-4
Total Trihalomethane Data for 2002-2004, μg/L

Distribution Site ID No.	Feb 2002	May 2002	Aug 2002	Nov 2002	Feb 2003	May 2003	Aug 2003	Nov 2003	Feb 2004	May 2004	Aug 2004	Oct 2004	Maximum System-Wide Running Annual Average	Maximum Location Running Annual Average
G-6	27	26	19	17	43	30	17	21	32	25	6	22	30	28
S-3	20	24	22	19	37	32	24	19	30	24	18	25		28
C-5	26	26	20	22	42	36	27	20	34	22	22	27		32
E-3	34	34	30	20	35	36	35	27	38	35	28	33		34
Monthly Average	**27**	**28**	**23**	**20**	**39**	**34**	**26**	**22**	**34**	**27**	**19**	**27**		

EXHIBIT 7-5
Haloacetic Acids (5) Sampling Data for 2002-2004, μg/L

Distribution Site ID No.	Feb 2002	May 2002	Aug 2002	Nov 2002	Feb 2003	May 2003	Aug 2003	Nov 2003	Feb 2004	May 2004	Aug 2004	Oct 2004	Maximum System-Wide Running Annual Average	Maximum Location Running Annual Average
G-6	42	44	28	49	62	38	15	38	51	38	1	34	46	46
S-3	35	42	24	47	60	37	33	39	46	35	26	39		44
C-5	41	43	27	18	69	41	35	41	53	35	29	42		47
E-3	58	60	36	57	62	56	45	60	59	48	41	50		56
Monthly Average	**44**	**47**	**29**	**43**	**63**	**43**	**32**	**45**	**52**	**39**	**24**	**41**		

CHAPTER 8

Distribution System Analysis

This chapter presents the analysis and recommendations for the distribution system. It also describes the city's pipe inventory and recommendations related to managing those assets.

Description of the Distribution System

Exhibits 8-1 and 8-2 provide a map and schematic diagram of the existing distribution system.

Service Zones

The Dalles' distribution system is currently divided into 13 service zones. Service zones represent the physical separation of the piping network into discrete areas based on ground elevation. They have been developed to provide acceptable pressures to customers. The zones are labeled numerically, the label generally reflecting the hydraulic grade line for service within the zone. This value is equal to the reservoir overflow elevation if the zone is fed directly from a storage tank.

Most water used within the city's system is supplied from the Wicks WTP, located to the south of the city. Water enters The Dalles' system by gravity from this source through two transmission lines: one feeds Garrison Reservoir, and one feeds Sorosis Reservoir. The higher elevation zones, which are fed from these two reservoirs or directly from one of the transmission pipelines, feed the lower zones through PRVs.

A smaller portion of water is supplied from the city's three wells. Marks and Jordan Wells pump directly into the distribution system, and the Lone Pine Well has a dedicated pump line to the Intermediate Reservoir, as shown in Exhibit 8-2.

Exhibit 8-3 lists the existing services zones, elevation ranges for customer connections, minimum and maximum static pressures, and the source from which service is provided.

EXHIBIT 8-3
Existing System Service Zone Summary

Service Zone Label	Direct Service Provided From	Storage Provided From (includes all upstream reservoirs that can serve zone)	Lower Customer Elevation (Maximum Static Pressure)	Upper Customer Elevation (Minimum Static Pressure)
310	From 395W through PRV #5 (alternatively, from 507 and 395 through PRVs 1, 2, 3, and 10)	Garrison Reservoir (also Sorosis Reservoir if alternative supply is used)	80 feet (100 psi)	193 feet (51 psi)
352	Intermediate Reservoir and Lone Pine Well	Intermediate, Columbia View, and Sorosis Reservoirs	80 feet (118 psi)	241 feet (48 psi)

EXHIBIT 8-3
Existing System Service Zone Summary

Service Zone Label	Direct Service Provided From	Storage Provided From (includes all upstream reservoirs that can serve zone)	Lower Customer Elevation (Maximum Static Pressure)	Upper Customer Elevation (Minimum Static Pressure)
395	From 460 through PRV#30 (alternatively, from 507 through PRVs 1, 2, 3, and 10)	Garrison Reservoir (also Sorosis Reservoir if alternative supply is used)	154 feet (104 psi)	300 feet (41 psi)
395W	From 460 through PRV #31	Garrison Reservoir	160 feet (102 psi)	311 feet (37 psi)
460	Garrison Reservoir, Jordan and Marks Wells	Garrison Reservoir	190 feet (117 psi)	355 feet (45 psi)
475	From 660 through PRV #19	Sorosis Reservoir	285 feet (82 psi)	385 feet (39 psi)
507	19th Street Reservoir, from 660 through PRV #7024	19th Street and Sorosis Reservoirs	243 feet (114 psi)	411 feet (42 psi)
513	From 632CV through PRVs #24 and 25	Sorosis and Columbia View Reservoirs	311 feet (87 psi)	416 feet (42 psi)
560	From 660 through PRVs #6 and 18	Sorosis Reservoir	335 feet (97 psi)	415 feet (63 psi)
632	660 through PRV #22	Sorosis Reservoir	335 feet (97 psi)	415 feet (63 psi)
632CV	660 through piping, Columbia View Reservoir	Sorosis and Columbia View Reservoirs	302 feet (143 psi)	511 feet (52 psi)
660	Sorosis Reservoir	Sorosis Reservoir	348 feet (135 psi)	615 feet (19 psi)
880	Mill Creek Transmission Pipeline (assumed HGL of 475 feet)	None (clearwell at Wicks WTP)	220 feet (110 psi)	407 feet (29 psi)

The largest demand area is the 310 zone. This encompasses the downtown area as well as the Port. Hence, it includes the majority of commercial and industrial customers. During peak summertime demand periods, this zone accounts for approximately one-third of the total system demand.

The gorge on the east side of the city, through which Highway 197 is located, restricts water movement in the east and west directions. This physical barrier does not completely eliminate movement of water into or out of the 632CV, 513, and 352 zones, but it does limit the transfer of water. This results in two system conditions. One is that the city is unable to make full use of the Lone Pine Well because the demand in 632CV, 513, and 352 zones is insufficient to fully use the 2 mgd pumping capacity of this well. The second, related condition is that there is insufficient turnover in the Columbia View Reservoir. The city has removed this reservoir from service in recent years when the Lone Pine Well was offline because of water quality concerns.

Pressure Reducing Valves and Closed Valves

The city's system relies on PRVs to supply water to most service zones. PRVs, while effective for a gravity flow system such as the city's, result in a complicated system—both to operate and to simulate using a hydraulic model.

The city's system uses approximately 25 PRVs, although only 8 of these delivered flow during peak hour conditions. Ninety percent of the PRV flow transfer occurred through three of these: No. 5 (from zone 395 to 310), 30 (from zone 460 to 395), and 31 (from zone 460 to 395W). In addition, the system has approximately 25 closed valves, meaning there are 25 locations where separate service zones are interconnected by pipe, but valves are closed on these pipelines to isolate zones.

Storage

Distribution storage is provided in five reservoirs. **Exhibit 8-4** lists the reservoirs, including their overflow elevations, material type, and volume.

EXHIBIT 8-4
Reservoirs

No.	Name	Volume (million gallons)	Overflow Elevation (feet)	Material Type	Comments
1	Sorosis	3.0	660	Steel	Fed directly from High Line. In need of repainting; however, existing system does not allow it to be removed from service
2	Garrison	6.0	460	Steel	Fed directly from Mill Creek Line.
3	19th Street (also called Hospital)	3.0	507	Steel	Fed from Sorosis Reservoir.
4	Columbia View Reservoir	3.0	632	Steel	Fed from Sorosis Reservoir or from Intermediate Pump Station. Since about year 2000, has only been used when the Lone Pine Well is operated, such as the summer of 2005.
5	Intermediate	1.0	352	Steel	Fed from Lone Pine Well or from Columbia View Reservoir.

Pump Stations

The Dalles system includes two booster pump stations, the Intermediate Pump Station and the Garrison Pump Station.

The Intermediate Pump Station is located next to the Intermediate Reservoir. This pump station lifts water from the 352 zone, which is fed by Lone Pine Well, to the 632CV zone. It fills the Columbia View Reservoir. The Intermediate Pump Station has an approximate capacity of 3,500 gpm if both pumps are operating.

The Garrison Pump Station transfers water from the Garrison Reservoir (at an overflow elevation of 460 feet) into the Sorosis Reservoir (at an overflow elevation of 660 feet). The transfer pipe line is a dedicated line, meaning there are no customer services on this line. The Garrison Pump Station is used only infrequently, during times when the Sorosis Reservoir is draining more quickly than it can be filled through the High Line. It is controlled by the level in Sorosis Reservoir. If the level drops to 20 feet, one pump comes on. If the level continues to fall, the second pump is turned on. The pump station has an approximate capacity of 1,200 gpm if both pumps are operating.

Concerns with Distribution System

The following distribution system needs and goals were identified at the outset of the project:

1. Provide an interconnection between the two northern zones (352 and 632) to increase the use of the Lone Pine Well. There is insufficient demand in the 352 and 632 zones to use the 1,600 gpm capacity of the Lone Pine Well, and the system does not currently provide a means to transfer this water to other zones. As described in Chapter 4, it is beneficial to increase the use of this well.

2. Modify the system so that water from the Columbia View Reservoir can service other than just the 632CV and 352 zones. The city removed the Columbia View Reservoir from service from 1999-2004 because the demands in the 632CV and 352 zones were inadequate to prevent stagnant water conditions in the tank. This problem is related to the problem identified in item 1 above. If storage in the Columbia View Reservoir can service areas to the west, this provides more use for water pumped from Lone Pine Well.

3. Reduce the number of service zones from the current total of 13. Thirteen is a large number for a city of this size. It makes for a complicated distribution system, requiring many PRVs and closed valves. The implications of this large number of zones include the following: a) it may not be clear in which zone a customer connection is located; b) the many closed valves and PRVs may result in poor water quality in dead end pipelines; c) the PRVs require regular maintenance. If they function improperly or have the wrong setting, they will cause high or low pressures in the system. d) The separated zones reduce the effectiveness of the storage reservoirs for providing peaking, fire, and emergency storage throughout the system.

4. Evaluate the location and size for a backup reservoir for Sorosis Reservoir so that Sorosis Reservoir can be removed from service for maintenance. The city believes that this tank has experienced significant corrosion and it is important to repair and repaint this tank as soon as possible.

5. Evaluate service to the Cherry Heights area. There are currently eleven homes in this area, which is located in the southwest corner of the system in the 460 zone. There is the potential for nine additional homes to be added, bringing the total to twenty. The homes are currently supplied by a single 6-inch line that crosses a creek. The city believes this is

vulnerable to service interruptions and asked for an evaluation of serving this area by means of a pipeline located on Cherry Heights Drive as a replacement or supplemental service to the existing line.

6. Consider a replacement for the 12-inch pipeline from Sorosis Reservoir north to Scenic Drive. The alignment for this existing pipeline is down a sheer, unstable cliff, rendering it vulnerable to failure.
7. Evaluate the storage needs for the system to determine if additional storage tanks should be included in the CIP. This specific goal is related to the backup reservoir analysis for Sorosis Reservoir.

In addition to these particular needs and goals, the distribution analysis included an evaluation of the fire flow capability of the system for current and buildout demands, and an analysis of the system needs to meet areas of projected demand growth.

Distribution Evaluation Criteria

Chapter 9, Design and Operating Criteria, provides a full listing of the system criteria that were adopted as part of this master plan. The following criteria relate specifically to the evaluation of the distribution system:

- Residential fire flow: 1,000 gpm for 2 hours
- School fire flow: 3,500 gpm for 3 hours
- Commercial/industrial fire flow: 4,000 gpm for 4 hours
- Pipe sizes: 12-inch-diameter outer loops, 8-inch-diameter internal grid
- Operating pressures: 40-100 pounds per square inch (psi) (note that above 80 psi may require a PRV to meet the Oregon Plumbing Code)
- Storage volume: sum of equalization (20-25 percent of MDD), emergency (100 percent of MDD) and fire (see above)

Model Development

The Dalles provided CH2M HILL with AutoCAD drawings showing pipe locations, pipe diameters, and pipe connections. Using customized GIS model-building software (H2OMAP), CH2M HILL traced over the pipe locations to create a digital file of the system. The Dalles also provided individual, detailed maps showing the locations of approximately 25 PRVs and approximately 25 closed valves. It was determined as the model was developed and tested that many of these valve locations or settings were incorrect. The Dalles' engineering and operations staff reviewed and corrected the information during the development phase.

The Dalles also provided a digital terrain model. This file provides spot elevations and coordinates throughout the system. Elevation contours and other procedures, such as triangulated irregular network files, were developed to assign elevations to all pipe connections.

The draft model was further reviewed by city and CH2M HILL staff during two multi-day workshops. These workshops were helpful in furthering the model accuracy for representing pipes, pipe connections, valve locations, and isolation valve status (open/ closed) and PRV settings. Field measurements of pressures were made at three locations during the second workshop. The model predictions for these locations were within 5 psi of the field measurements, providing confidence that the model accurately represents the system.

Demands were assigned to the model based on metered consumption records and the demand analysis presented in Chapter 3. These included both existing (2004) demands and projected demands for buildout. The 1 mgd weather allowance and 3 mgd industrial allowance discussed in Chapter 3 were not included in the modeled demands. The relatively small weather allowance was not large enough to affect the model, and the location of a potential new industrial customer is not yet known. If a new industrial consumer is identified and a specific site located, the city will need to revisit distribution capacity to the identified site. Since available industrial sites are located in the lower service zones, providing adequate water service is not likely to be problematic.

Exhibit 8-5 provides the overall summary of the maximum day and peak hour demands. The 2004 MDD equals 7.0 mgd and the buildout MDD equals 9.2 mgd.

EXHIBIT 8-5
Summary of Modeling Demands

Year	Maximum Day Demand (mgd)	Peak Hour Demand (mgd)
2004 (existing)	7.0	9.2
Buildout	9.2	12.1

Exhibit 8-6 summarizes the demands by service zones. This table includes peak hour as well as maximum day demands. The peak hour demands were used to simulate high demand conditions. Peak hour demands represent those that are expected to occur during the highest hour of use during the maximum summer day. For all but the 310 service zone, peak hour demands were estimated as one and one-half times the MDD for each zone. In the 310 zone, the peak hour demand was estimated as 1.1 times the MDD. The short-term peak demands within the 310 zone are mitigated by the industrial and commercial customers, which tend to use water at a more even rate throughout the day than residential customers. Therefore, the multiplier for the 310 zone, which has a significant number of industrial and commercial customers, was set lower than for other zones. The peak hour demands are estimates based on comparable systems because The Dalles has not monitored and compiled data to determine peak hour demands.

EXHIBIT 8-6
Modeling Demands by Service Zone

Service Zone	ADD (mgd)		MDD (mgd)		Peak Hour (mgd)	
	Current	Buildout	Current	Buildout	Current	Buildout
310	1.30	1.70	3.02	3.97	3.32	4.36
352	0.13	0.18	0.29	0.41	0.44	0.62
395	0.39	0.39	0.92	0.92	1.38	1.38
460	0.14	0.16	0.32	0.36	0.48	0.54
475	0.20	0.21	0.47	0.49	0.71	0.74
507	0.13	0.14	0.31	0.33	0.43	0.44
513	0.12	0.28	0.29	0.65	0.44	0.98
560	0.02	0.02	0.06	0.06	0.09	0.09
632	0.21	0.34	0.49	0.78	0.74	1.17
660	0.33	0.49	0.77	1.15	1.08	1.63
880	0.04	0.05	0.10	0.12	0.15	0.18
Total	**3.03**	**3.97**	**7.04**	**9.24**	**9.24**	**12.12**

System Evaluation Overview

Hydraulic modeling scenarios were developed for the existing system (year 2004) and for the projected system to serve buildout demands. It is common for master planning to check the performance of the system for projected conditions at the end of a 20-year period. As explained in the demand projection section of this report (Chapter 3), The Dalles' demand in 20 years is projected to be only slightly less than the demand when the system is fully built out to the urban growth boundary (UGB). Therefore, the future modeling was performed using the buildout demand.

The scenarios evaluated the system's capability to satisfactorily operate under maximum day demands, peak hour demands, and maximum day demands plus fire flows. These represent the stress conditions. Modeling was performed with and without proposed modifications to the system to test 'what if' conditions.

Findings

The following sections describe the findings from the network and storage evaluation of the distribution system. **Exhibits 8-7 and 8-8** provide a map and schematic drawing of the proposed future distribution system. These display recommended improvements that are presented within this chapter.

Performance During High Demand

The models for the existing system and for the proposed, future system were used to evaluate pressures throughout the distribution system during peak hour demand periods. In general, the system operates at acceptable pressures during 2004 and buildout peak hour demand conditions. Pressures range between 30 and 90 psi.

One small area experienced lower pressures: 16th Street between Lincoln and Liberty Streets. This area is near the zone boundaries between the 460, 560, and 475 service zones. The 460 zone is fed from Garrison Reservoir, with the maximum hydraulic grade set by the Garrison Reservoir overflow elevation of 460 feet. The 560 and 475 zones are fed via PRVs from the 660 zone at higher hydraulic grade lines. The current system configuration supplies a few pipelines from the 460 zone that should be served at a higher hydraulic grade line. City staff is aware of this problem area. They plan to investigate further following completion of the master plan to determine valving changes and possibly the addition of small sections of pipe to increase pressures.

The system has a small number of isolated low pressures areas in addition to these on 16th Street. In general, the low pressure areas are located near zone boundaries. The city can continue to investigate adjusting the closed valves or PRVs to increase pressures in these areas without creating high pressure problems in other areas.

Residential Fire Flows

The city's goal is to provide 1,000 gpm of fire flow to all residential areas. The ability of the distribution system to meet this goal was evaluated by superimposing a fire flow demand on a maximum day demand. This is a typical modeling condition. It represents the performance of the system in providing fire flows during a high demand, summertime condition. It does not represent the most severe condition, providing fire flows superimposed over peak hour demands, because the simultaneous requirement to meet peak hour and fire flow demands is highly unlikely. Buildout demands were used for all fire flow tests because they are only slightly greater than 2025 demands.

It was found that the city's system is capable of supplying the 1000 gpm fire flow to all areas except for two relatively small areas:

- West side of the 880 zone
- South edge of the 660 zone/east end of the 475 zone/southeast corner of the 507 zone

Both of these areas are served by extensive sections of 6-inch diameter pipelines. To increase the fire flows to 1,000 gpm will require replacement of many of these lines. The estimated requirements to improve fire flows in these two areas are as follows:

- West side of 880 zone: Requires approximately 3,000 feet of pipe replacement, consisting of 8- and 10-inch diameter pipelines. The total estimated cost is approximately $200,000.
- South edge of 660 zone/east end of 475 zone/southeast 507 zone: Requires approximately 24,000 feet of pipe replacement, consisting of 8-, 10-, and 12-inch-diameter pipelines. The total estimated cost is approximately $1,400,000.

It is recommended that the city consider the need for these improvements to determine if the expected benefits are commensurate with the costs. These projects are not included in the capital improvements plan developed in this master plan.

Industrial and Commercial Fire Flows

Most of the commercial and industrial customers are located within the 310 zone. Fire flows in this zone range from approximately 1,500 to 3,400 gpm. Most the locations in the central and eastern area of 310 can be supplied with 3,000 to 3,400 gpm fire flows. On the west and north sides of 310, the fire flows range from 1,500 to 2,000 gpm, rising to 3,000 gpm toward the center of the service zone. Specific values throughout the system are included in the electronic model files that were provided to the city as part of this master plan. The city staff should review the findings with the fire department and/or review building types to determine if increased fire flows are warranted. In many cases, the most appropriate improvement may be to improve the fire suppression systems of the buildings.

In addition to checking geographical areas, fire flows were analyzed at the following large industrial and commercial customers:

- Oregon Cherry Growers, 1021 Bargeway Road: 1,600 gpm
- Oregon Cherry Growers, 1st and Madison: 2,000 gpm
- Columbia Gorge Community College, 400 E. Scenic Drive: 2,000 gpm
- Mid-Columbia Medical Center, 1720 E. 19th: 3,900 gpm
- AmeriTies West, LLC: 3,900 gpm.

As stated for the 310 zone, the city may wish to follow up with these customers to determine if the existing building types and fire suppression systems require fire flows above these values. Although the city's criteria indicate a goal of providing 4,000 gpm for 4 hours for industrial and large commercial users, this may not be feasible in much of the system. If necessary, customers may need to improve their building fire suppression systems.

Storage Evaluation

Distribution storage is necessary to satisfy three uses: equalization, fire fighting, and emergency. The specific criteria used for evaluating these functions have been noted earlier in this section and are more fully explained in Chapter 9, Design and Operating Criteria. Equalization storage provides the water to compensate for the difference between the peak hour demands and the supply (which is designed to meet maximum day demands). Fire fighting storage provides a reserve for the high flows needed by the fire department to fight fires. Emergency storage provides a reserve to supply customers during times when the supply is interrupted. For The Dalles' system, which can be fed by gravity throughout, the concern is over system-wide interruptions such as a problem at the water treatment plant or failure of the transmission pipelines.

Both the equalization and emergency storage components are directly related to demands within a service level. If demands increase, these components increase. The fire component depends on land use, building types, and building spacing within the service area. For example, service areas that are strictly residential require a lower fire flow and for a shorter duration than a service area that includes commercial buildings or schools.

The configuration of the service zones within the city's system allows storage to be shared among zones. Garrison Reservoir (6 MG), which has an overflow elevation of 460 feet, cannot serve all zones. However, there is a pump station and dedicated line between Garrison and Sorosis Reservoirs, enabling this storage to effectively serve all zones.

Exhibit 8-9 summarizes the storage evaluation for The Dalles' system. As shown in this table, the system has a surplus of storage, compared to the selected design criteria. This surplus is expected to be sufficient to meet buildout demands with only a minor overall deficit. A subsection of this section discusses the need for adding a backup reservoir to Sorosis Reservoir, which is a system need despite the overall storage surplus.

EXHIBIT 8-9
Storage Needs Evaluation
All volumes in million gallons

Condition	Existing Storage Volume (MG)[1]	Required Storage Volumes (MG)[2]				Storage Surplus (+) or Deficit (-)
		Equalization	Fire	Emergency	Total	
2004	16.0	1.8	1.0	7.0	9.7	6.3
Buildout	16.0	3.1	1.0	12.2	16.2	-0.2

Notes:
1 The buildout analysis is based on projected demands that include the industrial allowance of 3.0 mgd.
2 Specific criteria for evaluating storage needs are summarized in Chapter 8.

Service Zones

The hydraulic model was used to test several potential combinations of service zones, with the goal being to reduce the number of service zones to simplify the operation and maintenance of the system. It was found that the geography and present system configuration rule out most possibilities for combining service zones. However, the number of separate service zones can be slightly reduced through the following measures. The revised service zones are illustrated in Exhibit 8-8.

1. Combine the 475 and 507 service zones. Both zones serve similar customer elevations. As illustrated in Exhibit 8-2, the 475 zone has been fed through a PRV from the 660 zone. The 507 zone has been fed from the 19th Street Reservoir. The two zones are interconnected by piping but the valves on these interconnecting lines are normally closed (valves No. 5 and 6 on the schematic). An alternative configuration is to feed both the 475 and 507 zones through PRVs from the 660 zone and/or from the 19th Street Reservoir. This can be accomplished as shown in Exhibit 8-7. In this case, the valves on the interconnecting lines would be opened.

2. Although the 660 and 632 zones are not isolated from one another, the transfer of water between the two zones is almost entirely from the 660 to the 632 zone, because the 660 zone operates at a higher hydraulic grade line than the 632 zone. The 660 zone is fed from the Sorosis Reservoir at an overflow elevation of 660 feet. The 632 zone is fed from the Columbia View Reservoir at an overflow elevation of 632 feet. The future system schematic shows the addition of a booster pump station on Morton Street to lift water from the 632 zone into the 660 zone. This will enable the two zones to operate in

combination, and will allow water stored in the Columbia View Reservoir to supply additional customers located in the 660 zone and in zones fed by the 660 zone. This improvement is described under the heading of the Lone Pine Well and Columbia View Reservoir Analysis, below.

3. An interconnection is also proposed between the 352 and 310 zones, as shown in Exhibit 8-7. This interconnection will enable the Lone Pine Well to provide service to the higher demand area of the 310 zone. It will require a pipeline, as described under the heading of the Lone Pine Well and Columbia View Reservoir Analysis.

Lone Pine Well and Columbia View Reservoir Analysis

The hydraulic model was used to determine the needed improvements to:

- Allow the Lone Pine Well to pump continuously during summer high demand periods, at flows of up to 2,000 gpm
- Withdraw water from the Columbia View Reservoir at rates that are sufficient to provide adequate turnover of the contents.

To significantly increase production from the Lone Pine Well, it is necessary to provide a pipeline connection from the 352 zone to the 310 zone (the area including the downtown and Port). Summertime demands in the 310 zone currently exceed 3 mgd. This zone contains a large portion of the city's overall demands. Even with increased use from the Columbia View Reservoir, there is insufficient demand that can be fed from the Lone Pine Well without the connection to the 310 zone.

The city plans to install a portion of this pipeline prior to the peak demands in 2006. The pipeline will initially feed a large demand in the lower portion of the 395 zone. When extended in the future, it will provide a direct connection to the 310 zone.

A delivery of 1,400 gpm (2 mgd) was targeted through the 352-310 pipeline. This provides sufficient transfer to account for the possible addition of a second well in the Lone Pine Well area bringing the total supply to 2,800 gpm (4 mgd) in this zone. (Chapter 4, Water Supply, discusses the possibility of increasing the pumping rate from Lone Pine Well or adding a second well in this vicinity.)

Additional production in the 352 zone also can be distributed into the system if the area served by the Columbia View Reservoir is expanded. Water is pumped from the 352 zone, through the Intermediate Pump Station, into the 632 zone. The most feasible approach to increasing withdrawals from this tank is to modify the interconnection between the 632 zone and the 660 zone to the west. This will enable the Columbia View Reservoir to serve part of the 660 zone. The 660 zone is currently fed only from Sorosis Reservoir.

Appendix G provides further details of the analyses, findings, and recommendations for increasing the use of the Lone Pine Well.

Transmission Pipeline from 352 Zone to 310 Zone

The effectiveness of connecting the zones with either 12-inch or 16-inch pipelines was examined. In addition, it was found that movement within the 352 zone is restricted and that a new section of pipeline is needed within this zone. This pipeline improves the delivery of water from Lone Pine Well to the 16-inch pipeline. **Exhibit 8-10** summarizes the results for the two sizes of transmission lines, with and without the new 8-inch line. These results are peak hour demands at projected buildout conditions.

EXHIBIT 8-10
Results for Increasing Use of Lone Pine Well
Demand condition: peak hour demand, buildout, reservoir level 5 feet below overflow

Transmission Pipeline Size	New 8-inch Line Included?	Result: Flow from 352 zone to 310 zone
12-inch	No	830 gpm (1.2 mgd)
12-inch	Yes	1,000 gpm (1.5 mgd)
16-inch	No	1,200 gpm (1.7 mgd)
16-inch	Yes	1,400 gpm (2.0 mgd)

The transfer of water from the 352 zone to the 310 zone will occur at lower rates than shown in Exhibit 8-10 until demand increase to the buildout levels. With both the 16-inch line and the 8-inch line in place, the model indicates that the transfer rate will be approximately 1,100 gpm (1.6 mgd) for projected 2005 demands.

It is recommended that a 16-inch-diameter pipeline be used for the new transmission line, together with the installation of the 8-inch line within the 352 zone. The combination of these two improvements enables the transfer of 2.1 mgd (1,460 gpm) from the 352 zone to the 310 zone. It is important to transfer at least this flow rate because there is the possibility of increasing the capacity of the Lone Pine Well or of adding a second well in this area. These expansion possibilities are discussed in Chapter 4, Water Supply.

Morton Street Pump Station and Dry Hollow Road PRV Added to 660 Zone

To increase the use of the Lone Pine Well, it is also necessary to pump more water into the 632CV zone and into the Columbia View Reservoir via the Intermediate Pump Station. Because demands are relatively low in the 632CV zone, this only can be done if more water can be moved through this zone to feed areas to the west. Moving water from the 632CV zone can be accomplished by placing a small booster pump station near Morton Street. The pump station should have two pumps, each using a 20-hp motor, to move approximately 1.0 mgd each from the 632CV zone to the 660 zone.

In addition to the pump station, a PRV could be added to reduce flows from the 660 zone into the 632CV zone. This could be located near the intersection of Three Mile Road and Dry Hollow Road on the existing 12-inch pipeline. The movement of water from the 632CV zone into the 660 zone works best if the 19th Street Reservoir is removed from service.

With the Morton Street Pump Station operating and a PRV setting of 60 psi for buildout peak hour demands, the flow from Columbia View Reservoir into the eastern half of the 660 zone equals approximately 1,350 gpm.

Pipeline from Sorosis Reservoir to Liberty Way

Water is delivered north from Sorosis Reservoir (660 zone) to Liberty Way (475 zone) via a 12-inch pipeline that is routed over a sheer, unstable cliff, rendering it vulnerable to failure. The hydraulic model was used to evaluate options for replacing this line. It would require approximately 2,000 feet of pipeline to circumvent the cliff.

The system was modeled without this pipeline to simulate its failure. It was found that the 6-inch pipelines that are routed around this cliff have sufficient capacity to provide acceptable peak hour and fire flow service. It is not necessary to replace the vulnerable section of this pipeline to provide reliable service.

Cherry Heights Analysis

The Cherry Heights area is a small residential area located in the 460 zone. There are currently 11 houses and the buildout capacity is estimated at 20 houses. The area is served by a single 6-inch water line that crosses a creek, making it vulnerable to loss of service. The city was interested to know if the area could also be served by a pipeline located on Cherry Heights Road, to both increase fire flow capacity and to provide a second, redundant service line to the area. This option was evaluated and it was found that a pipeline following this alignment is not a feasible option because it would deliver water at low pressures. In addition to requiring 1,700 feet of 8-inch pipeline, this option would also need a booster pump station. Based on this analysis, the preferred alternative, if the city wishes to increase fire flows and reliability to these homes, is to install a parallel 8-inch line to the one that currently serves the area. The length is about 2,000 feet. To further the reliability, the city could consider fortifying the creek crossing as well as separating the pipes at the point where they cross the creek.

760 Zone Service

There is a significant amount of developable land located in the 760 zone. It is estimated that this land could support 109 new houses south of the hospital and 328 new houses in the area east of Thompson Street and south of 12th Street, for a total of 437 houses. Using the criteria presented in the water use projections section of the report, single family dwellings are expected to have 2.4 people per dwelling on average, giving a population of 1,050 people.

Storage needs for this service zone can be calculated by first determining the MDD. At a per capita MDD use of 640 gpcd, this yields a MDD of 670,000 gpd. The criteria presented previously for storage indicate that the fire flow storage should be 120,000 gallons (1,000 gpm for 2 hours), the equalization storage should be 170,000 gallons (25 percent of the MDD), and the emergency storage should be 670,000 gallons (100 percent of the MDD). This gives a total storage need of 960,000 gallons. It would be reasonable to reduce this amount to 840,000 gallons by summing the equalization storage plus the larger of the emergency or fire storage values (in this case, the emergency storage value). This approach is taken by many water systems based on the assumption that a fire and an emergency supply need would not occur simultaneously.

This size would be adequate to provide backup for Sorosis Reservoir, so that Sorosis Reservoir can be removed from service for short periods to perform maintenance. With a maintenance schedule for non-peak periods and increased reliance on the wells, it should be sufficient to provide only about 400,000 gallons to replace Sorosis Reservoir for short periods. Because a 760 reservoir will be needed eventually, it appears that this may be a cost-effective solution to providing backup. However, the difficulty may be the timing. The city will need to invest in pipelines and a pump station to install the 760 reservoir, and at this time there is insufficient development occurring in this area to justify these expenditures.

The city has already performed a siting study for this reservoir. This study should be consulted for guiding the development of this tank.

Pipeline Material Condition Assessment

This section presents available information relating to the physical condition of the city's distribution piping.

The Dalles' transmission and distribution system consists of approximately 68 miles of pipelines. As shown in **Exhibit 8-11**, 94 percent of this pipe is either cast iron (75 percent) or ductile iron (21 percent). The system also includes small amounts of asbestos cement, galvanized iron, polyvinyl chloride (PVC), and steel pipe.

EXHIBIT 8-11
Distribution Pipe Inventory by Material and Diameter

Material	Diameter (inches)	Total Length (feet)	Total Length (miles)	Percent of System
Asbestos Cement	6	1,527	0.3	0.4%
	8	827	0.2	0.2%
	10	1,907	0.4	0.5%
Subtotal		**4,261**	**0.8**	**1.2%**
Cast Iron	4	2,365	0.4	0.7%
	6	159,505	30.2	44.2%
	8	44,374	8.4	12.3%
	10	14,002	2.7	3.9%
	12	42,414	8.0	11.7%
	14	272	0.1	0.1%
	16	6,936	1.3	1.9%
Subtotal		**269,868**	**51.1**	**74.7%**
Ductile Iron	6	27,673	5.2	7.7%
	8	5,443	1.0	1.5%
	10	11,920	2.3	3.3%
	12	22,287	4.2	6.2%
	14	2,338	0.4	0.6%
	16	5,449	1.0	1.5%
Subtotal		**75,110**	**14.2**	**20.8%**
Galvanized Iron	4	619	0.1	0.2%
	6	1,418	0.3	0.4%
Subtotal		**2,037**	**0.4**	**0.6%**
Polyvinyl chloride (PVC)	6	821	0.2	0.2%
Subtotal		**821**	**0.2**	**0.2%**

EXHIBIT 8-11
Distribution Pipe Inventory by Material and Diameter

Material	Diameter (inches)	Total Length (feet)	Total Length (miles)	Percent of System
Steel	4	512	0.1	0.1%
	6	593	0.1	0.2%
	12	1,189	0.2	0.3%
	14	6,785	1.3	1.9%
Subtotal		**9,079**	**1.7**	**2.5%**
Total		**361,177**	**68.4**	**100.0%**

The age of the pipes are not included in the inventory. Cast iron pipe was generally installed in the United States through the mid-1970's, so it can be assumed that 75 percent of the city's distribution pipe is older than about 1975. The ductile iron pipe has been installed since that time.

Exhibits 8-12 to 8-14 provide graphical summaries of the pipe inventory, including inventories by diameter for cast iron (**Exhibit 8-13**) and ductile iron (**Exhibit 8-14**).

The city has not reported problems with pipe durability in the system. It does not appear that significant rehabilitation or replacement of pipe is needed in the next 20-year period. However, the city may wish to consider developing a database to track pipe failures and leaks so that future planning can consider budgeting for pipe rehabilitation and replacement. **Exhibit 8-15** provides an indication of the city's investment value in the distribution pipe network. Not including appurtenances such as customer meters, PRVs, reservoirs, hydrants, and similar facilities, the value of the pipelines exceeds $25 million in current dollars. This is an approximate value, based on using a unit cost for replacement of $8.50 per diameter-inch per foot.

EXHIBIT 8-15
Approximate Pipe Replacement Cost
(based on $8.50 per diameter-inch: cost includes design/road/contingency allowance)

Diameter	Replacement Cost
4 inches	$119,000
6 inches	$9,769,000
8 inches	$3,444,000
10 inches	$2,366,000
12 inches	$6,721,000
14 inches	$1,119,000
16 inches	$1,685,000
Total	**$25,223,000**

The following criteria impact pipe durability:

- Water conditions such as pH, alkalinity, hardness, and Langlier Index (an indicator of calcium carbonate saturation)—these parameters will vary depending on the relative use of surface and groundwater

- Pipe material and pipe age; also pipe joint type
- Soil conditions such as soil resistivity, acidity (pH), moisture content, soluble salts, and oxygen content
- Water pressure, with higher pressures resulting in greater susceptibility to leakage
- The presence of stray electrical currents
- Bedding and other construction conditions, which primarily affect susceptibility to impact failure from earth moving (earthquake, operation of heavy equipment, etc.)

Soil conditions can vary throughout the system. By evaluating pipeline condition and the level of soil corrosivity during routine operation and maintenance, the city can flag areas of particular concern (e.g., highly corrosive soil conditions, evidence of pipe corrosion) for increased monitoring, corrosion control, or pipeline replacement. Opportunities for pipe inspection and soil analysis include when new services are installed, when breaks or leaks are repaired, when connections of a new main are made to an existing one, or during excavations for other utilities.

The following procedures are recommended for collecting information about pipeline and soil conditions to guide policies regarding pipeline replacement:

1. Document pipe type, location, and condition using a standard form.
2. Photograph pipe and attach photo to form.
3. Perform soil condition measurements and document results.
4. Update database periodically with newly collected information.

Cast and ductile iron pipe should be carefully examined to determine if graphitization has occurred (a condition in which the iron dissolves from the pipe wall and leaves behind graphite). The pipe may look new (it might even have the original paint markings) but the pipe wall may have almost no remaining strength.

The Dalles may wish to consider purchasing a pipe pit gauge, a $100 tool that can be used to measure the depth of localized (pitting) corrosion.

In addition to developing a database from which to make pipeline replacement decisions, all new pipe installations in the size of 6-inch through 24-inch should be ductile iron. For larger diameter pipelines, ductile iron, steel, high-density polyethylene (HDPE), and concrete cylinder pipe materials should be considered.

Summary of Recommended Improvements

1. Add a 16-inch pipeline to connect the 352 and 310 zones. Exhibit 8-7 illustrates this line, which is approximately 6,400 feet long. The final length will depend on the actual routing. This will enable transfer of approximately 2 mgd from the 352 zone to the 310 zone, allowing increased use of Lone Pine Well.
2. Add an 8-inch pipeline within the 352 zone to facilitate moving water from Lone Pine Well to the new 16-inch line. Exhibit 8-7 illustrates this line, which is approximately

2,600 feet long. The 352-310 transmission pipeline does not provide its full benefit without this 8-inch line.

3. Install a booster pump station on Morton Street and a PRV near Dry Hollow Road, both in the 660 zone. The Morton Street Pump Station will move water from the 632CV zone into the 660 zone, enabling the Columbia View Reservoir to serve this area. The higher use from the Columbia View Reservoir allows the 632CV zone to receive more water from the Lone Pine Well while providing sufficient turnover in this tank to sustain high quality drinking water.
4. To accomplish the higher use from the Columbia View Reservoir, it is also necessary to shut down the 19th Street (Hospital) Reservoir. Eventually, when demand increases, it will be possible to use both the Columbia View and 19th Street reservoirs. The 19th Street Reservoir supplies water to the 507 zone, which is also fed through PRVs from the 660 zone. If the 19th Street Reservoir is on-line, the demand from the Columbia View Reservoir is reduced significantly.
5. The sustained pumping from Lone Pine Well is dependent on adjusting management of the Intermediate Pump Station. It appears that it would be beneficial to add a smaller pump, with a capacity of approximately 250-350 gpm, to this station. A smaller pump would help to balance the movement of water from the 352 zone into the 632 zone. The city can determine the need for such a pump after implementing the previously described improvements.
6. Combine 475 and 507 service zones by opening the valves between these two areas.
7. Consider installing a parallel pipeline to the existing line that crosses the creek to improve the reliability of service to Cherry Heights. The alternative of providing service from another location would require a booster pump station and is therefore not recommended.
8. Install an 840,000-gallon reservoir to serve the 760 zone. This will also be used to provide backup service to the Sorosis Reservoir, so that Sorosis Reservoir can be removed from service for maintenance. Therefore, even though the development in the 760 zone does not warrant constructing the reservoir at this time, it is a project that should move ahead in the next few years because of the severe need for maintenance to Sorosis Reservoir.
9. City staff should consider the findings relating to fire flows to determine if piping improvements are warranted. Fire flow results were provided for nodes throughout the system in the model that was delivered to the city as part of this master plan project.
10. Track pipeline replacement and rehabilitation needs by collecting data as described in this section.

The Morton Street Pump Station, the new PRV on Dry Hollow Road, and the abandonment of the 19th Street Reservoir are interrelated projects. The pump station should be completed and on line prior to installing the PRV. The 19th Street Reservoir can be taken off-line following implementation of these two improvements.

The improvements described will not enable the city to use Columbia View Reservoir when all water is being supplied from the Wicks WTP without stagnation problems in the reservoir.

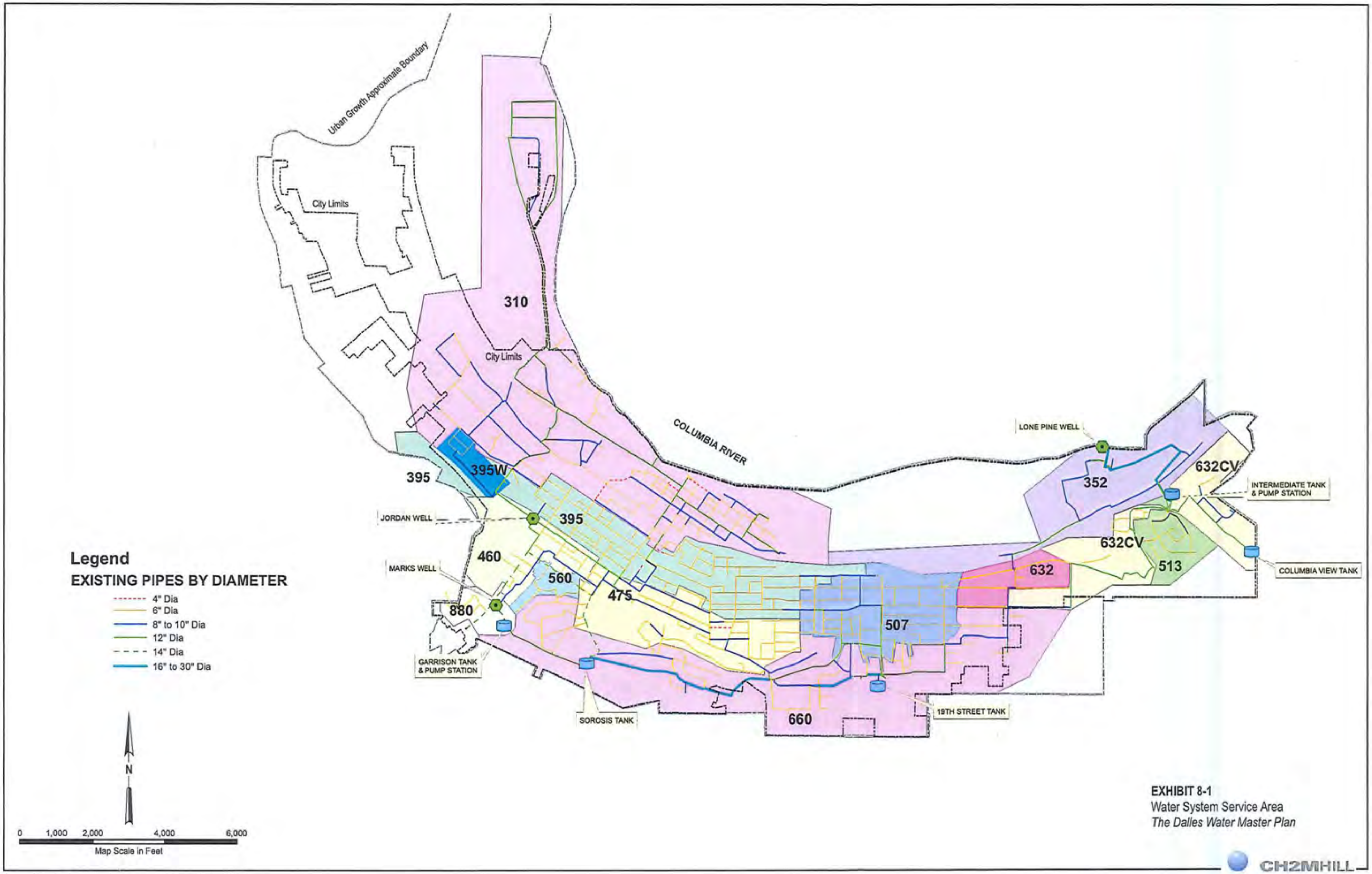
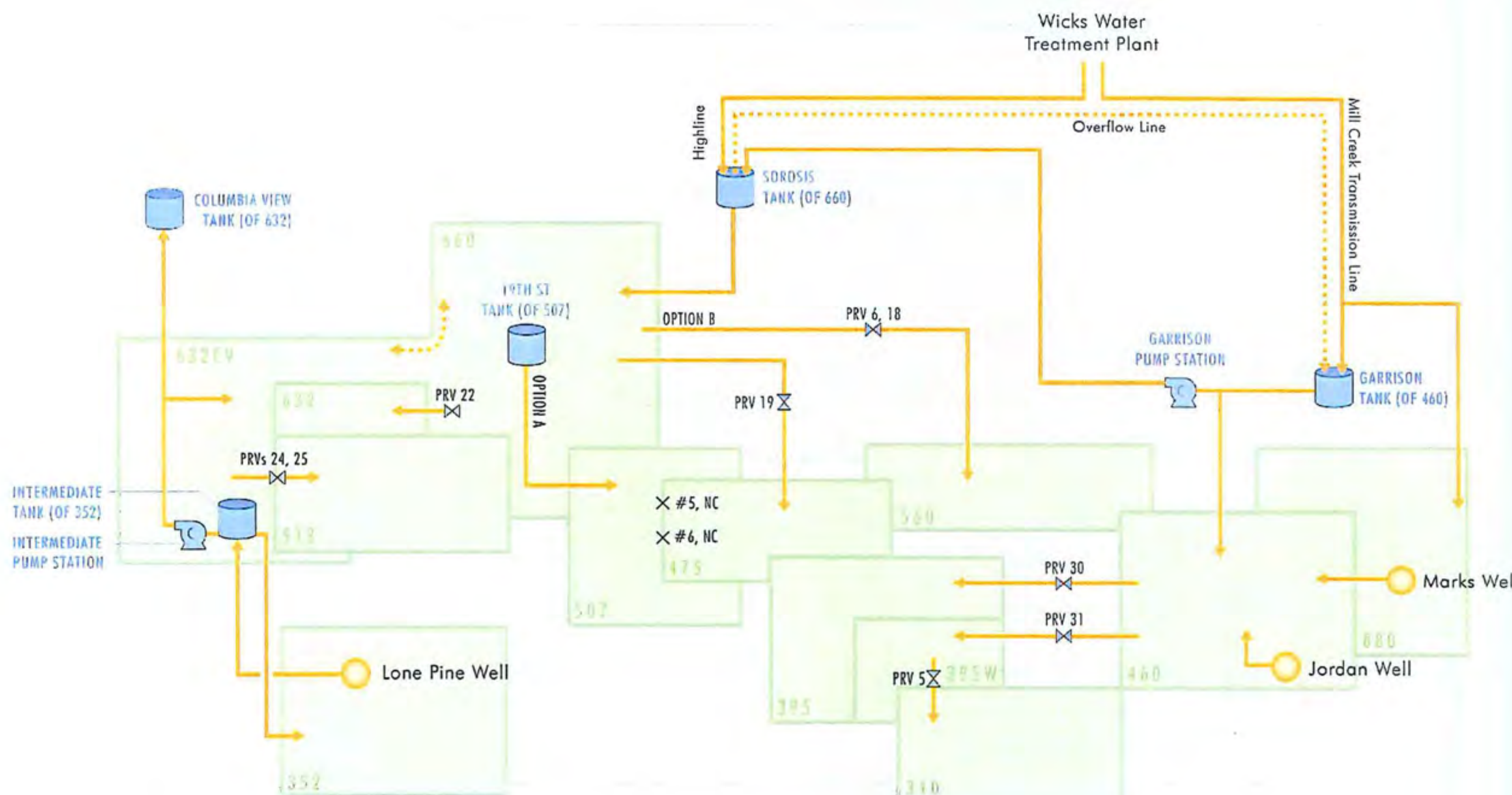


EXHIBIT 8-1
Water System Service Area
The Dalles Water Master Plan

CH2MHILL

E122004004CVO-DWMP_107_01
CVO\060400031

EXHIBIT 8-2
Existing Distribution System Schematic



Legend

PRV = Pressure Reducing Valve

NC = Normally Closed

OF = Overflow Elevation

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EXHIBIT 8-2
Existing Distribution System Schematic
The Dalles Water Master Plan

CH2MHILL

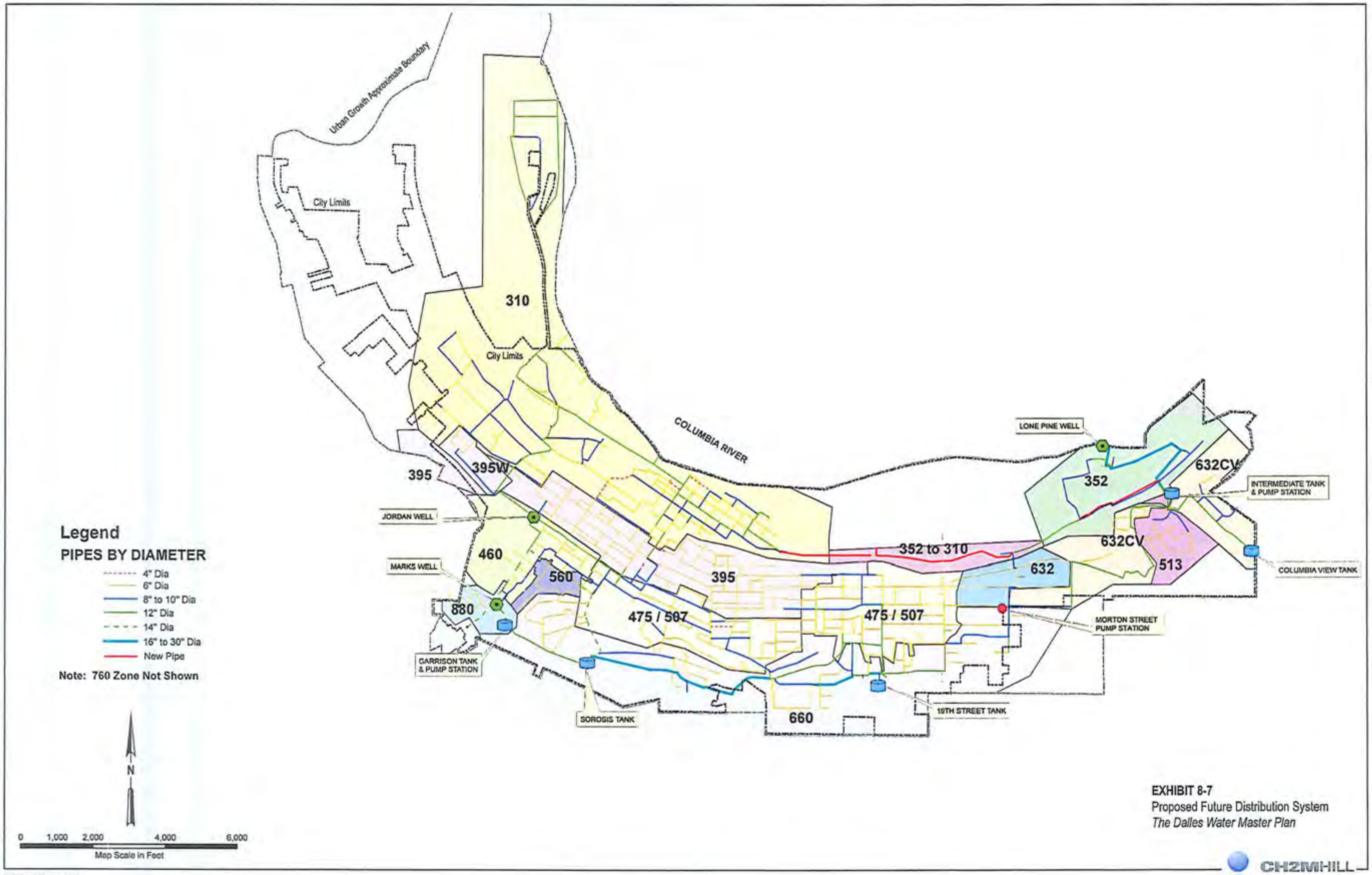

EXHIBIT 8-7
Proposed Future Distribution System
The Dalles Water Master Plan

EXHIBIT 8-8
Future Distribution System Schematic

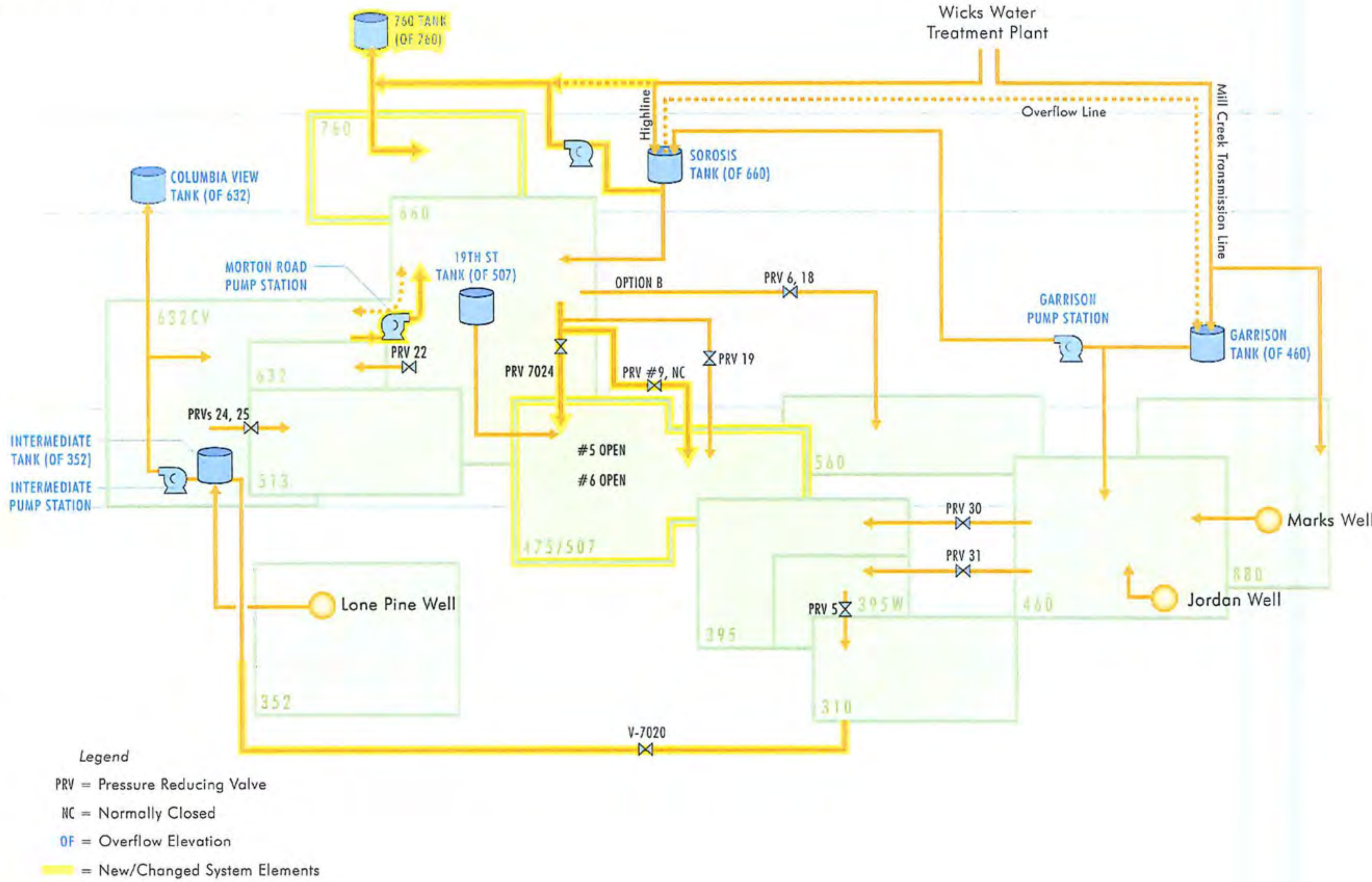

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EXHIBIT 8-12
Pipe Inventory by Material Type

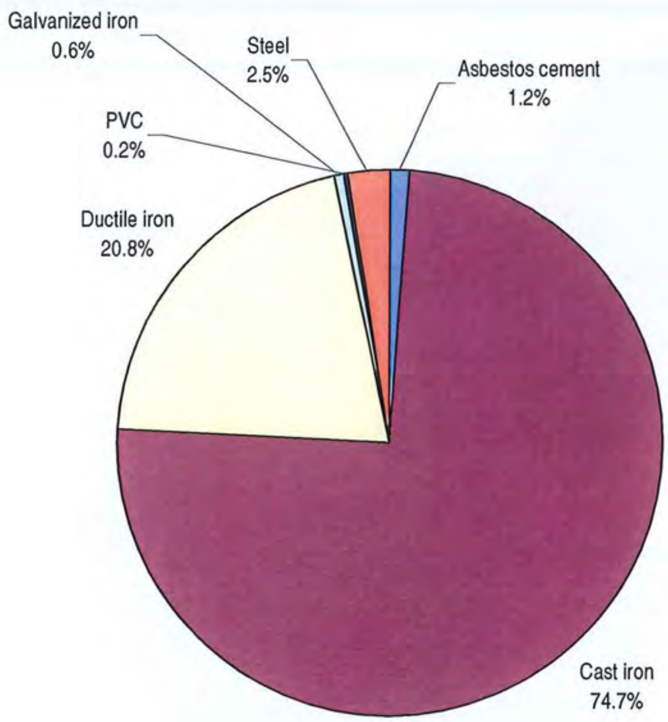


EXHIBIT 8-13
Inventory of Cast Iron Pipe by Diameter

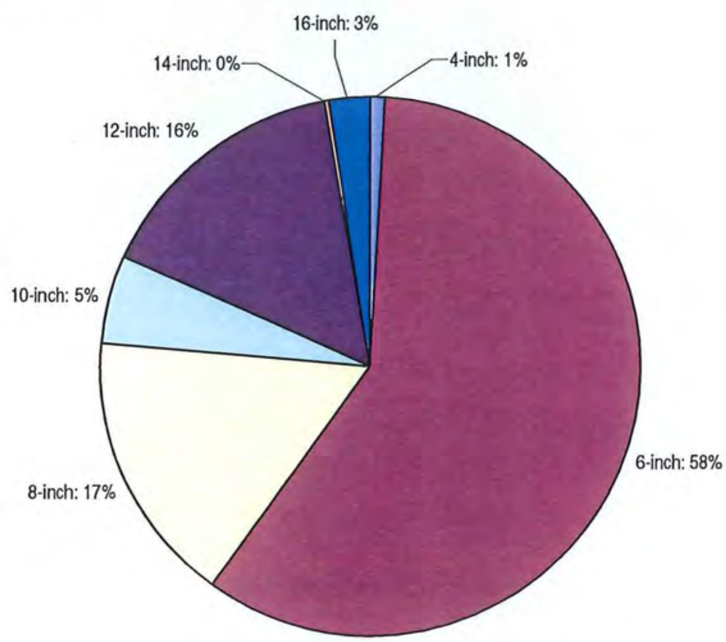
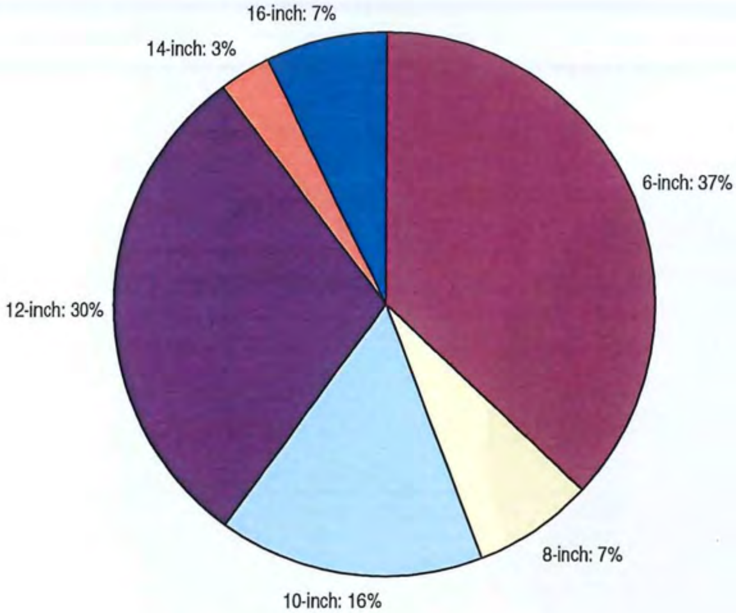


EXHIBIT 8-14
Inventory of Ductile Iron Pipe by Diameter



CHAPTER 9

Design and Operating Criteria

Exhibit 9-1 shows the recommended design and operating criteria that were updated as part of the master plan. In some cases, final decisions were not made and The Dalles will need to continue to evaluate and develop appropriate criteria.

A number of the design criteria, such as fire flows, storage requirements, and pipe sizing, were used as a basis for determining capital improvement needs for the City's system in this master plan. Other criteria are not critical for developing a master plan, but do provide guidance to the City for evaluating detailed designs of improvements. These include criteria for hydrant spacing, valve spacing, pipe materials, and emergency power connections for pump stations. The operating criteria primarily relate to maintaining and using existing facilities. Examples of operating criteria include valve exercising, record keeping, and flushing.

EXHIBIT 9-1

Design and Operating Criteria

No.	Item	Criteria Used in Previous Master Plan or Current City Values	Applicable Regulations	Recommended Value	Basis for Recommended Value	Discussion
1	Fire flows for single-family residential areas	1,500 gpm for 2 hours, storage of 180,000 gallons	ISO: 1000 gpm for 2 hrs National Fire Protection Agency: Sliding scale for single family residential 0-3600 sf: 1000 gpm/2 hrs 3601-4800 sf: 1750 gpm/2 hrs 4801-6200 sf: 2000 gpm/2 hrs 6001-7700 sf: 2250 gpm/2 hrs	Minimum of 1,000 gpm for 2 hours (120,000 gallons), at a minimum residual pressure of 20 psi, superimposed over maximum day demands	ISO, the nation's leading source for ranking fire suppression effectiveness, downgrades a community's insurance rating unless at least 1,000 gpm is available for 2 hours for houses situated such that the spacing between houses is 11 to 30 feet.	*Recommended Standards for Water Works* ('Ten States Standards') indicates that fire flows shall meet ISO standards. California Administrative Code requires 750 gpm minimum for residential one story, single family dwellings on average sized lots, and 2,000 gpm for more densely built areas, apartments, and light commercial. Oregon has no flow requirements, but does require 20 psi at all times. ISO standards also call for residual pressure of 20 psi.
2	Fire flows for schools and other habitational buildings	5,000 gpm for 5 hours, storage of 1,500,000 gallons	ISO: 3500 gpm for 3 hours (630,000 gallons)	Minimum of 3500 gpm for 3 hours (630,000 gallons)	ISO downgrades a community's insurance rating unless at least 3,500 gpm is available for 3 hours for habitational buildings such as schools. This category also includes care centers and light commercial.	See discussion for residential fire flows. No Oregon requirements.
3	Fire flows for multi-family residential areas	3,000 gpm for 3 hours, storage of 540,000 gallons		3,000 gpm for 3 hours, 540,000 gallons	City's present standard is reasonable	See discussion for residential fire flows. No Oregon requirements.
4	Fire flows for commercial and industrial areas	4,000 gpm for 4 hours, 960,000 gallons	ISO: 4,000 gpm for 4 hours (960,000 gallons)	4,000 gpm for 4 hours, 960,000 gallons	ISO sets commercial and industrial fire flow requirements based on building material type and other variable factors, and may require up to 12,000 gpm for full insurance credit. It is recommended that The Dalles supply up to 4,000 gpm, and for buildings needing more than this amount, require sprinklers.	No guidance from other states or Ten States Standards for commercial/industrial areas
5	Hydrant spacing	500 feet between hydrants		600 feet maximum spacing between hydrants so that distance to a house is <=300 ft. This is needed to meet ISO credit for 1500 gpm residential fire flows	ISO credits hydrants for up to 1,000 gpm if located within 300 feet of structure, for 670 gpm if located 301 to 600 feet from structure, and for 250 gpm if located from 601 to 1000 feet from structure. A spacing of 1,000 feet maximum would ensure at least 1,000 gpm is available to each house.	
6	Hydrant type			Provide at least one large pumper outlet (typically a 4-inch port)	ISO downgrades fire hydrants that do not have at least one large pumper outlet.	
7	Residential piping: sizes and looping			12" dia outer loops (for <= 1-mile sq) 8" dia internal grid 6" diameter in cul-de-sacs (for <250 ft length). Limit velocities to approximately <=6 fps for peak hour demands. (Higher velocities are acceptable for meeting fire flow demands.)	Follows Washington Administrative Code. Meets OARs (minimize dead ends) and Ten States Standards (minimum of 6-inch diameter mains)	Several states require a minimum of 6-inch diameter mains, and indicate that dead end lines shall be minimized. Proliferation of cul-de-sacs means that the criterion of allowing 6-inch diameter dead end mains up to 250 feet in length may result in a system that is not well-looped. Therefore, it is critical to confirm acceptability of dead end lines using hydraulic model.
8	Transmission mains: sizing			Evaluate on a case-by-case basis, based on allowable head loss. Velocities up to 8-10 fps are acceptable for peak hour demands.	Peak hour demands are uncommon, and sizing a transmission main for velocities of 8-10 fps will result in lower velocities a large percentage of the time.	No guidance from other states or Ten States Standards.

EXHIBIT 9-1
Design and Operating Criteria

No.	Item	Criteria Used in Previous Master Plan or Current City Values	Applicable Regulations	Recommended Value	Basis for Recommended Value	Discussion
9	Operating pressures		Oregon: minimum is 20 psi	Normal (any time except during fire flows): 40 - 100 psi. Minimum for fire flows: 20 psi. Pressures measured at service connection (meter).	Oregon requires a minimum of 20 psi at all times, as do most states. The 40-100 psi normal range is a reasonable target, recognizing that it may be acceptable in some cases for the minimum to drop below 40 psi and still provide acceptable service.	Oregon is silent on pressure except for the 20 psi minimum. Washington requires 30-110 psi, California 25-125 psi, Texas >35 psi, and Pennsylvania 25-125 psi. Ten States Standards indicates that normal working pressures should be 60-80 psi, and not less than 35 psi.
10	Pressure reducing valves on customer services		Plumbing code PRVs for pressures > 80 psi.	Customers to provide their own PRVs when pressures > 80 psi. City provides if system change results in pressures > 80 psi. Customer responsible in any case for O&M and city has no liability.	Typical for water utilities	
11	Equalization storage volumes: residential only	20% of MDD		25% of maximum day demand	Lacking actual diurnal demand curves, 25% is a more conservative value than used in city's last master plan and is typical for water utilities	Only general guidance is provided by states, indicating that equalization storage should consider daily use patterns.
12	Equalization storage volumes: residential plus schools/commercial			20% of maximum day demand	Accounts for lower diurnal peaks from commercial customers and schools than from strictly residential areas.	Only general guidance is provided by states, indicating that equalization storage should consider daily use patterns.
13	Emergency storage volumes	1 x MDD		1 x MDD	Assumes that failure of system occurs on a maximum day demand, and that customers continue water use at MDD rate for 12 hours, then reduce use to ADD rate for 24 hours, and that emergency condition is fixed in 36 hours.	Washington regulations indicate that emergency storage may be reduced when there is a second independent supply, such as groundwater in The Dalles' case. Therefore, The Dalles' could consider less than 1 x MDD. The criteria for emerg. storage may need to consider longer durations for specialized customers--hospitals or industrial users--where the consequences of a shortage are severe.
14	Total storage	Sum of fire, equalization, and emergency storage volumes. (There is currently no water quality consideration in this equation.)		Sum of fire, equalization, and emergency storage volumes --or-- equalization plus fire or plus emergency, whichever is larger	Need to balance distribution storage between meeting storage needs and water quality considerations	Washington codes allow a system to provide the total of the equalization storage plus the larger of the emergency or fire volumes. This approach assumes that a fire will not occur concurrently with an emergency failure.
15	Valve exercising			Exercise all valves at least once every 4 years. Consider more frequent exercising for older valves and large diameter (>= 12")	Annual valve exercising is commonly recommended for all valves, however, this is probably not practical. Some systems exercise older valves (gate valves with expanding seats) annually and resilient seat valves at least once every 4 years.	States do not provide guidance on valve exercising.
16	Water age/chlorine residual/HPC			At all distribution system locations: measurable free chlorine residual; HPC counts < 1 cfu/mL	The critical water age is system-specific. EPA has a value for HPC as a non-regulated surrogate of 500 cfu/mL. A value of 100 cfu/mL is conservative in protecting water quality. Together with maintaining a measurable chlorine residual, these are the best available practices for ensuring safe drinking water in the distribution system. (The City of The Dalles has a HPC goal of < 1 cfu/mL)	One further criterion that may be considered is to limit the maximum water age in the system, particularly if a long water age can be associated with low chlorine residuals or high HPC counts. May need separate summer and winter management policies.

EXHIBIT 9-1
Design and Operating Criteria

No.	Item	Criteria Used in Previous Master Plan or Current City Values	Applicable Regulations	Recommended Value	Basis for Recommended Value	Discussion
17	Pump station sizing			Provide maximum day demand over 24 hours, with largest pump out of service	A typical approach for pump station sizing.	
18	Number of pumps in booster pump stations			Two or more; 4 for isolated, closed-end systems	A typical approach for pump stations	
19	Pipe materials	Ductile Iron, HDPE and other materials where appropriate		Use ductile iron pipe as standard. Consider HDPE or steel or large transmission lines.	Ductile iron pipe is less prone to leaks than other pipe materials, and is the industry standard.	
20	Backflow prevention standards	Fulfill Oregon's rules		Fulfill Oregon's rules	Oregon's backflow rules are comprehensive and defensible	
21	Water quality monitoring in distribution system			Monitor for chlorine residual using on-line instruments at locations prone to low residuals or high water age. Consider also additional instruments for flows out of reservoirs.	More comprehensive sampling in distribution system helps to ensure that high quality water is delivered to all customers. In addition, it provides value from a water system security standpoint.	Selection of sites can be evaluated using hydraulic model and by reviewing system maps
22	Water use record keeping		DHS has some record-keeping requirements	Track average day, maximum day, and monthly total demands and record annually. Track within individual service levels to extent possible. Develop monthly and annual numbers for unaccounted water.	These data are very helpful for planning purposes, and are time-consuming or impossible to generate if not recorded on a regular basis.	
23	Main Flushing			Every 6 months for dead end and problem areas; goal for entire system is once every 4 years	Typical water system practice	
24	Reservoir inspection/cleaning			Inspections every 5 years using divers; cleaned only as inspection shows need		
25	Reservoir turnover			Depends on water quality. Many systems do not experience problems even though the water age is longer than AWWA recommendations	AWWA recommends complete turnover every 3-5 days	
26	Use of closed-end pumping systems in place of reservoir storage			15 or less homes preferred on a dead-end, 30 homes max on a dead-end	Although it is ideal to serve all customers with gravity storage, it may be an unacceptable cost to serve small groups of homes with a reservoir and may lead to water quality problems	
27	Isolation valving			Maximum of 4 valves to close in order to isolate segment	Typical water system practice	
28	Number of services on an isolation segment			Not more than 30 homes max	Typical water system practice	
29	Installation of flush ends on dead end mains in cul-de-sacs.			Install flushing ends for all dead-end mains	Typical water system practice	
30	Provision of emergency generators for pump stations			Provide for all closed-end systems		
31	Pump stations: backup power connections			Standard for all distribution pump stations		

EXHIBIT 9-1
Design and Operating Criteria

No.	Item	Criteria Used in Previous Master Plan or Current City Values	Applicable Regulations	Recommended Value	Basis for Recommended Value	Discussion
32	Reservoir design: inlet/outlet piping		DHS: "When a single inlet/outlet pipe is installed and the reservoir floats on the system, provisions shall be made to insure an adequate exchange of water to prevent degradation of the water quality..." (OAR 333-061-0050 (7))	Provide separate inlet/outlet piping for all new reservoirs; include inlet riser pipe (keep top below normal operating level so as not introduce extra pumping head)	Follows DHS regulations	
33	Master plan: update schedule			Annual minor updates; more significant review every 5 years; comprehensive review every 10 years		
34	5-Year capital improvements plans (CIPs)			Proposed: Annual updates; ensure that 5-year plans follow general guidelines of the master plan. Plan shall be within financial guidelines of water division, and shall be balanced and prioritized so that rate increases are justified		
35	Annual capital budgeting			Shall reflect 5-year CIP. Modifications shall be justified and documented.		

CHAPTER 10

Capital Improvements Plan

This section summarizes the improvements discussed in the preceding sections, and presents a CIP update for The Dalles' water system. All exhibits are located at the end of this section.

The review of water rates and funding alternatives, as required by the state's Drinking Water Program master planning rules, are provided in a separate report, which will be completed and made available in parallel to the master plan report.

Capital Improvements Plan

Exhibit 10-1 presents the proposed CIP update for The Dalles. The individual projects include those that have been described in the technical sections of this report, and in some cases, projects that The Dalles has previously identified as needed.

The project selection, prioritization, and timing were evaluated during preparation of the master plan through a series of meetings with The Dalles staff and members of the Citizen Advisory Committee that was formed for this project. The attached CIP represents the agreed-upon plan. The project dates shown in the CIP should be considered approximate. The Dalles will evaluate the proposed projects each year and make adjustments as appropriate. This is particularly the case for projects in later years. These dates will be refined and estimated costs updated as their proposed implementation dates become nearer.

As discussed in Chapter 8, hydraulic modeling of fireflow conditions identified two areas within the city with fireflow capacities below the city's residential fireflow criterion of 1000 gpm. To improve fireflow capacity, the replacement of existing pipelines with larger diameter pipelines was recommended. These pipeline replacement projects are not included in the CIP, as the city has yet to evaluate their relative merit.

Exhibits 10-2, 10-3, and 10-4 are cash flow charts. The first provides a summary for all projects. The second and third display cash flow for projects that address growth, and for the portion of the projects that support existing customers (maintenance projects).

Project Cost Background

The project cost estimates are considered rough order of magnitude estimates. Actual costs will vary by plus 50 percent to minus 30 percent, depending on the final project scope, the bidding climate, and other variable factors.

The project cost estimates are given in June 2005 dollars at an approximate *Engineering News-Record* Construction Cost Index for Seattle Area value of 8208. Prior to finalizing the funding for a project, it will be necessary to update the cost estimate to current costs and to develop a preliminary design to further define the project.

EXHIBIT 10-1

Capital Improvements Plan

	Start Date	End Date	Project Title	Description	Project Allocation: Capacity	Project Allocation: Regulatory & Maintenance	Construction Cost Estimate	Construction Contingency	Total Construction Estimate	Allowances for Eng, Admin, Permitting	Total Capital Cost	Notes
#4	2006	2006	Lone Pine Well transmission pipe	First section of 16" pipeline to transmit water from Lone Pine Well zone to 310 zone; 1500' (to Kerr McGee) of 16"	80%	20%	$150,000	35%	$200,000	$40,000	$240,000	City intends to implement this improvement in 2006
#3-	2006	2006	Land cost for Crow Creek Dam raise	Allowance for city's land exchange with the US Forest Service for raising Crow Creek Dam	100%	0%	$0	0%	$0	$0	$250,000	Allowance amount provided by city
#4	2006	2006	Lone Pine Well expansion	Increase capacity from existing 1600 gpm to 2000+ gpm by installing new pump and motor. Includes allowance for new MCC	100%	0%	$120,000	35%	$160,000	$30,000	$190,000	Planned for 2006 to obtain near-term increase in supply. Assume 2000 gpm at dynamic head of 450 feet (from Golder report) for hp = 303. Existing is 300 hp, so need to increase to 350 hp.
#4 PHASE II	2006	2006	Lone Pine Well transmission pipe	Second section of 16" pipeline to transmit water from Lone Pine Well zone to 310 zone; 5000' of 16"	100%	0%	$480,000	35%	$650,000	$70,000	$720,000	Planned for 2006 to obtain near-term increase in supply
	2006	2006	New well siting study	Evaluate potential sites, permitting, contaminant sources, and hydrogeology for a new well in Lone Pine area	100%	0%	$0	0%	$0	$30,000	$30,000	To determine feasibility of adding a new well near Lone Pine Well and, if feasible, to identify a favorable property for city to purchase. Cost of land purchase is not included.
	2006	2008	760 Zone: Booster pump station	Provide pump station to fill 760 zone tank from Sorosis Reservoir: 3 pumps at 350 gpm each. Each approx. 15 hp. Use unit cost of $5,000/hp.	100%	0%	$225,000	40%	$320,000	$70,000	$390,000	Planned improvements to finished water transmission pipeline will eliminate the need for the pump station. Packaged pump station planned.
	2006	2008	760 Zone: Supply reservoir tank	Add steel reservoir tank to serve 760 zone and to provide backup for Sorosis Reservoir. Volume = 840,000 gallons. Includes allowance for 500' of 16" pipe connections.	100%	0%	$900,000	35%	$1,220,000	$250,000	$1,470,000	760 Zone will eventually require this tank for gravity supply. It is needed in near-term for a backup to Sorosis Reservoir so that it cab be removed from service for badly needed repainting.

EXHIBIT 10-1
Capital Improvements Plan

	Start Date	End Date	Project Title	Description	Project Allocation: Capacity	Project Allocation: Regulatory & Maintenance	Construction Cost Estimate	Construction Contingency	Total Construction Estimate	Allowances for Eng, Admin, Permitting	Total Capital Cost	Notes
#1	2006	2008	WTP near term improvements	4.3 MG steel clearwell, as designed in Nov. 2003. Estimate at that time was $2.72 mil. Allowing for 14% inflation since, estimate has been increased to $3.1 mil.	50%	50%	$3,100,000	15%	$3,570,000	$180,000	$3,750,000	Engineering is for bidding and construction services, only. Design is complete. Low project contingency is based on having completed design.
	2006	2008	WTP near term improvements	Add mechanical flocculators to existing basins. 4 units total. Each unit $15,000 for equipment and $10,000 for installation. Plus $40,000 allowance for electrical and controls.	33%	67%	$140,000	40%	$200,000	$40,000	$240,000	Provides near-term increase in capacity
#2	2006	2008	WTP near term improvements	Upgrade filters by replacing underdrains with gravel-less type and replacing media. Two filters, each 20' x 15' (300 sf). Install 40" depth of dual media.	33%	67%	$80,000	40%	$120,000	$30,000	$150,000	Provides near-term increase in capacity
	2006	2008	WTP near term improvements	Add 20,000 sf solids drying bed, total depth of approximately 7', with inlet and outlet structures.	33%	67%	$150,000	40%	$210,000	$40,000	$250,000	Provides near-term increase in capacity
	2006	2008	WTP near term improvements	Replace flash mix basin with an in-line mechanical mixer to improve coagulation and increase capacity	50%	50%	$80,000	40%	$110,000	$20,000	$130,000	
#2	2006	2008	WTP near term improvements	Allowance for rehabilitation/replacement of mechanical and electrical equipment at plant: valves, valve operators, electrical panels, painting of pipes, etc	0%	100%	$0	0%	$200,000	$40,000	$240,000	The specific projects have not been yet been defined
	2006	2008	WTP near term improvements	Replace one of the two existing wash water tanks with a new 100,000 gallon tank	50%	50%	$200,000	40%	$280,000	$30,000	$310,000	One of existing 50,000 gal tanks has experienced significant corrosion and needs replacement. Larger volume will accommodate increased plant capacity.
#2	2006	2008	WTP near term improvements	Replace gas chlorination system with on-site chlorine generation system	50%	50%	$300,000	20%	$360,000	$40,000	$400,000	Existing chlorinators are 37 and 24 years old and end of useful life. On-site system provides improved safety. Will be designed for ultimate plant flow.

EXHIBIT 10-1

Capital Improvements Plan

#3	Start Date	End Date	Project Title	Description	Project Allocation: Capacity	Project Allocation: Regulatory & Maintenance	Construction Cost Estimate	Construction Contingency	Total Construction Estimate	Allowances for Eng, Admin, Permitting	Total Capital Cost	Notes
	2006	2010	Dog River Pipeline permitting	Allowance for NEPA permitting for Dog River Pipeline replacement project.	50%	50%	$0	0%	$0	$150,000	$150,000	Requires long lead time to give time for harvesting timber along easement route.
	2006	2025	Annual pipeline replacement	Allowance for distribution pipeline replacements ($75,000 per year for 20 years)	0%	100%	$1,300,000	0%	$1,300,000	$260,000	$1,560,000	City has only small amounts of AC, steel, iron, and PVC. Replacement is primarily for cast iron, which is 75% of system. Only 10% of the cast iron expected to need replacement over 20-year period of plan. This equates to 2,700 feet per year. At an average diameter of 8-inches (unit cost of $56/ft), this is $75,000 per year.
	2007	2008	New well	New production well in the area of Lone Pine Well: 1400 gpm capacity, 400 feet deep, pump and well house, chlorination, pipe allowance of 1000' of 12".	100%	0%	$850,000	35%	$1,150,000	$230,000	$1,380,000	Combined capacity of two Lone Pine area wells is conservatively estimated as 2800 gpm (4 mgd)
	2008	2008	Morton Street Pump Station	New pump station to facilitate moving water from 632 zone into 660 zone. Located on Morton Street. Two pumps: each 50 feet of head and 700 gpm (each 15 hp; use $5,000/hp unit cost)	50%	50%	$150,000	40%	$210,000	$40,000	$250,000	Packaged booster pump station. Provides improved turnover in Columbia View Rsvr and allows for greater use of Lone Pine Well
	2008	2008	Sorosis Reservoir repainting	Sandblast and repaint interior and exterior of Sorosis Reservoir, and upgrade to current seismic standards. Allowance included for minor replacement of interior roof supports, ladders, etc.	0%	100%	$350,000	40%	$490,000	$50,000	$540,000	Use $2.50/sf for inside and outside painting for the 3.6 mg, 30 ft tall tank (D=145). Wall A=14,500 sf (double for inside and outside). Floor/roof/ceiling=16,500 sf. Total=78,500
	2009	2010	Crow Creek Dam permitting	Allowance for permitting to cover wetlands mitigation, dam safety permitting, and related concerns	100%	0%	$0	0%	$0	$150,000	$150,000	
	2011	2012	Dog River Pipeline design & construction	18,500 feet of ductile iron pipeline, placed along existing alignment	50%	50%	$2,510,000	20%	$3,010,000	$300,000	$3,310,000	Detailed conceptual estimate included with technical memo

EXHIBIT 10-1
Capital Improvements Plan

PHASE III BONDING NOT YET ACCOUNTED FOR

Start Date	End Date	Project Title	Description	Project Allocation: Capacity	Project Allocation: Regulatory & Maintenance	Construction Cost Estimate	Construction Contingency	Total Construction Estimate	Allowances for Eng, Admin, Permitting	Total Capital Cost	Notes
2011	2013	Crow Creek Dam raise	Raise dam by 35' as described in city's 1996 study report. Costs derived from that report.	100%	0%	$6,410,000	20%	$7,690,000	$1,360,000	$9,050,000	Total includes allowance for engineering, permitting, and a 20% contingency
2014	2015	Cherry Heights connection pipeline	Provides second pipe connection to this residential area: 2000 ft of 8-inch	25%	75%	$120,000	25%	$150,000	$30,000	$180,000	Provide redundant service to Cherry Heights area
2018	2020	Finished Water Pipeline replacement	Replace existing two lines with a single, 24", ductile iron pipeline (Class 250 and 350, depending on pressure in specific sections). Includes allowance for serving existing alignment customers.	50%	50%	$7,620,000	20%	$9,140,000	$910,000	$10,050,000	Detailed conceptual estimate included with technical memo.
2022	2025	WTP expansion	Add rapid mix, new flocculation basin, new plate sedimentation basin, and 2 filters	100%	0%	$5,070,000	25%	$6,340,000	$1,110,000	$7,450,000	Estimate developed using CH2M HILL's in-house WTP software. Trigger for expanding WTP is when minimum needed capacity to meet demands plus 1 mgd reaches the total system capacity.
2026	2026	Iron/manganese treatment for Jordan Well	Install iron and manganese treatment facility for the Jordan Well using the ATEK process or similar	25%	75%	$750,000	40%	$1,050,000	$210,000	$1,260,000	Occurs beyond 20-year CIP
			Total			**$31,055,000**		**$38,130,000**	**$5,710,000**	**$44,090,000**	

EXHIBIT 10-2. Cash Flow Projections for All Capital Improvement Projects

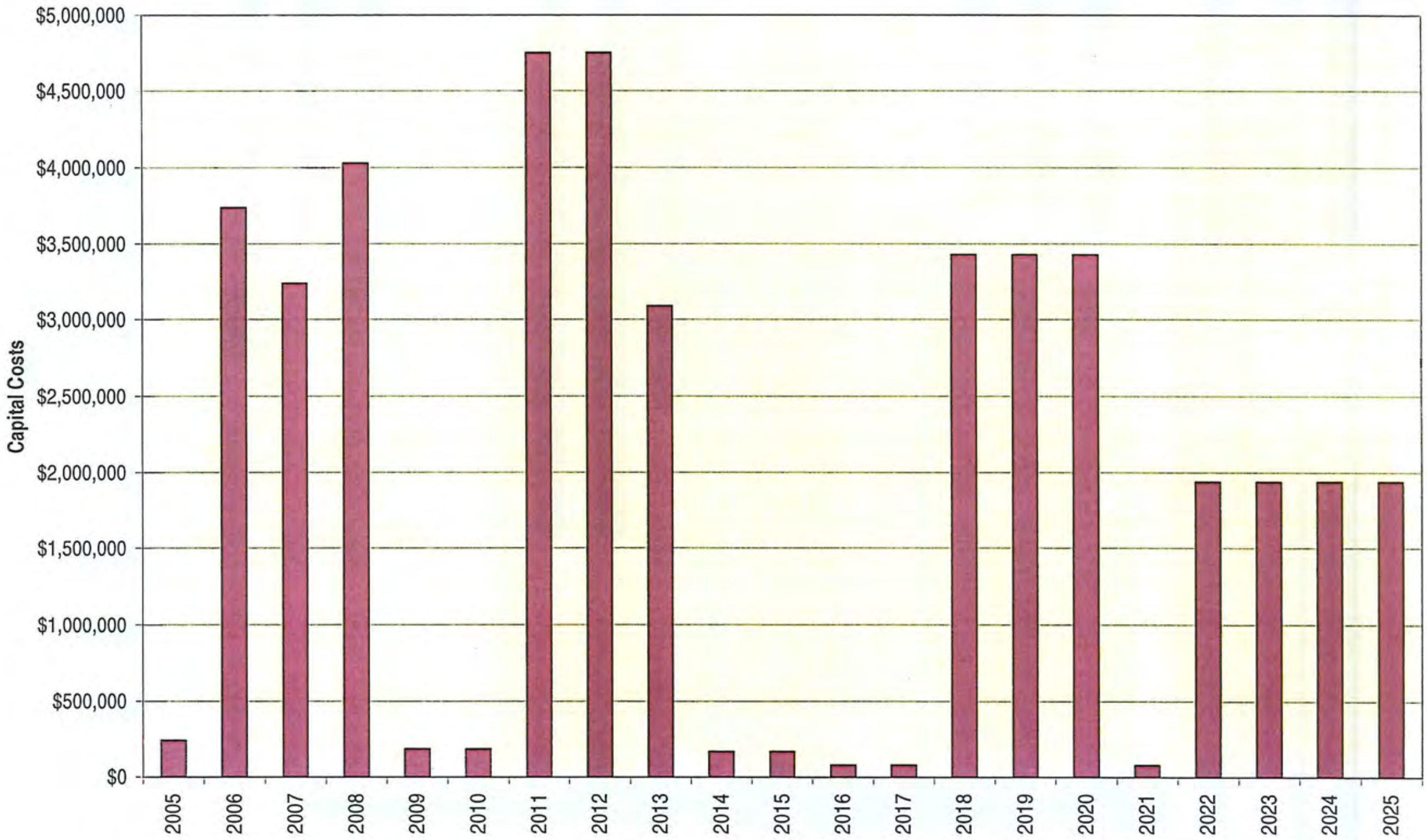


EXHIBIT 10-3. Cash Flow Projections for Capacity-Related Capital Improvement Projects

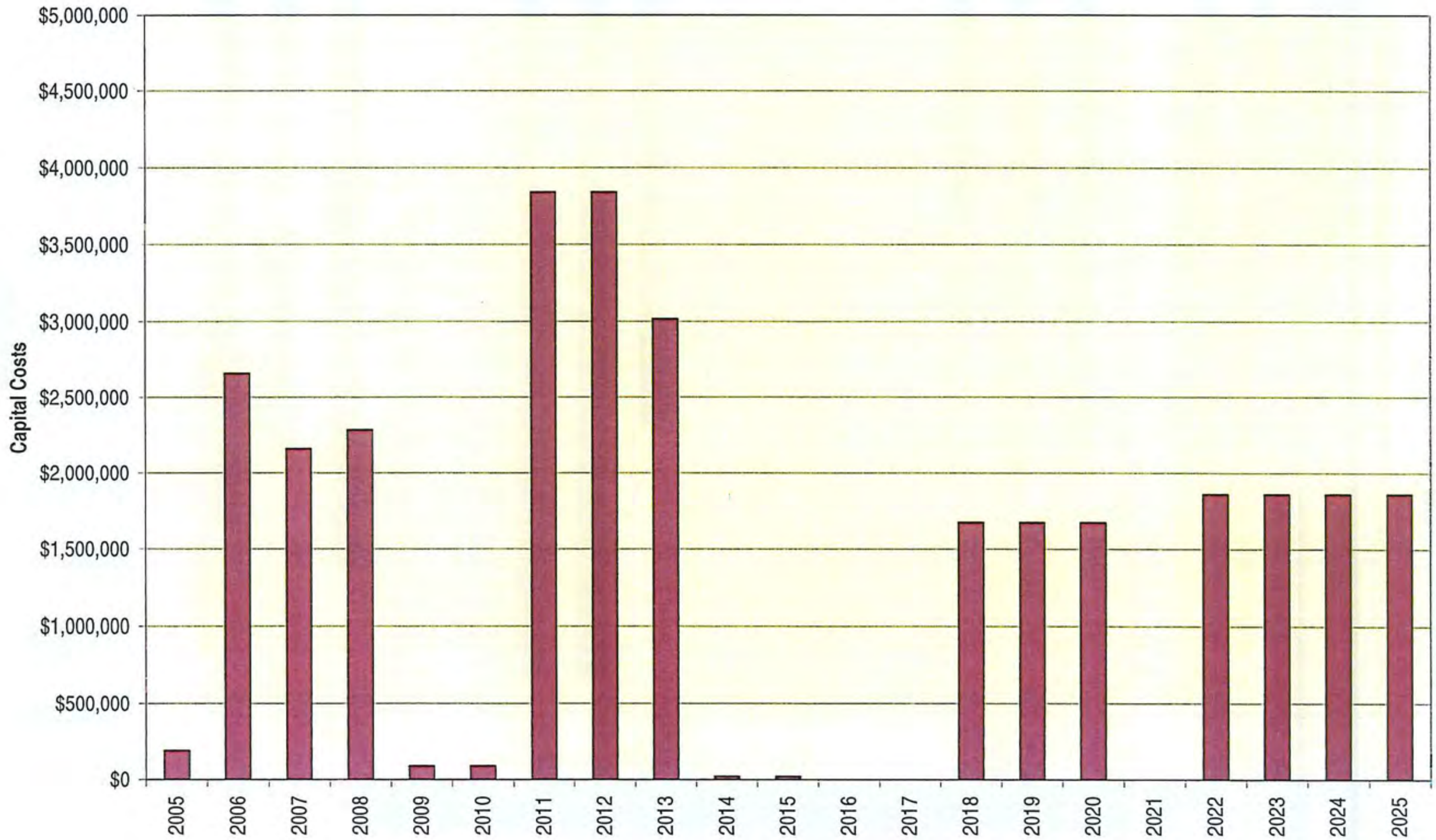
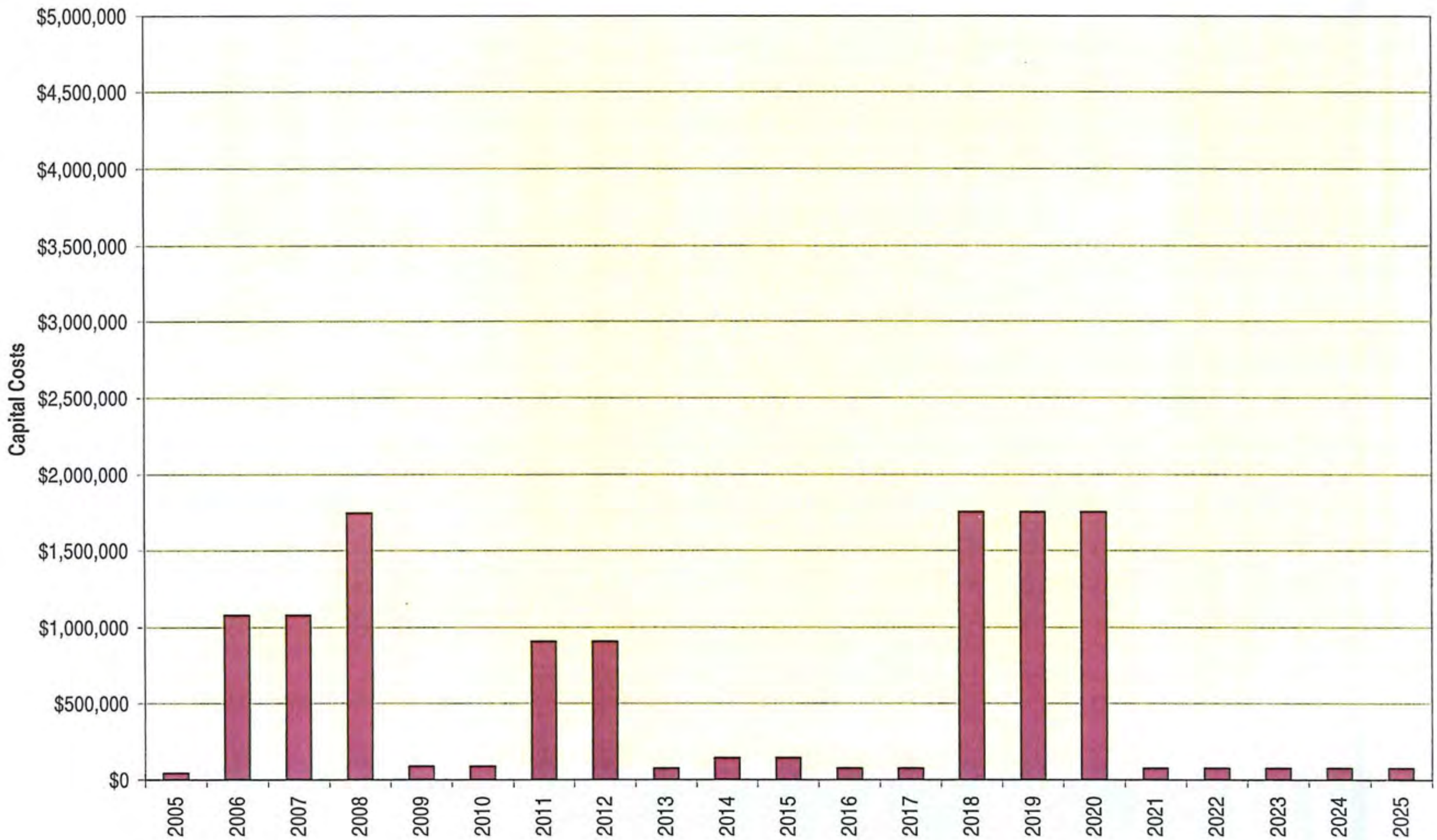


EXHIBIT 10-4. Cash Flow Projections for Regulatory/Maintenance Capital Improvement Projects



APPENDIX A

Supply Versus Demands for 2025 Demands and Ultimate 12 mgd Maximum Day Demands

Using 90% Exceedence Values for South Fork Mill Creek and Dog River

Month	Mill Creek (mgd)	Dog Creek (mgd)	Mill Creek plus Dog Creek Discharges (mgd)	Channel Losses, between dam and WTP intake (mgd)	Fish bypass flow (mgd)	Total available supply (mgd) = SFMC + DR - channel loss - fish bypass (mgd)	2025 Projected Average Daily Demands by Month (mgd)	Ultimate Daily Demands by Month for a 12 mgd Max Day (mgd)	Total Available Supply for 2025 demands (mgd)	Total Available Supply for 12 mgd Maximum Day (mgd)	2025 Conditions: Surplus available for filling dam	2025 Conditions: Storage needed in dam to meet demands	Ultimate, 12 mgd Maximum Day Conditions: Surplus available for filling dam	Ultimate, 12 mgd Maximum Day Conditions: Storage needed in dam to meet demands
Jan	2.8	1.2	4.0	0.0	3.0	1.0	2.2	3.2	-1.2	-2.2	0	37	0	67
Feb	3.7	1.7	5.4	0.0	3.0	2.4	2.4	3.5	0.0	-1.1	1	0	0	32
Mar	6.5	1.6	8.0	0.0	3.0	5.0	2.4	3.5	2.6	1.5	78	0	46	0
Apr	3.8	1.9	5.7	0.0	3.0	2.7	3.1	4.5	-0.4	-1.8	0	12	0	54
May	6.6	3.8	10.4	0.0	3.0	7.4	4.3	6.2	3.1	1.2	94	0	37	0
Jun	6.7	5.3	12.0	0.3	1.0	10.7	5.5	8.0	5.2	2.7	156	0	82	0
Jul	4.3	2.7	7.0	0.5	1.0	5.5	6.7	9.6	-1.2	-4.2	0	36	0	125
Aug	3.9	1.8	5.7	0.7	1.0	4.0	6.0	8.7	-2.1	-4.8	0	62	0	143
Sep	3.3	1.5	4.8	0.5	1.0	3.3	4.8	6.9	-1.5	-3.6	0	45	0	109
Oct	2.7	1.3	4.0	0.0	1.0	3.0	3.2	4.6	-0.2	-1.6	0	6	0	49
Nov	3.1	1.3	4.4	0.0	3.0	1.4	2.3	3.4	-1.0	-2.0	0	30	0	61
Dec	2.5	1.0	3.6	0.0	3.0	0.6	2.1	3.1	-1.6	-2.5	0	47	0	76

		2025 Conditions: Surplus	2025 Conditions: Storage	Ultimate: Surplus	Ultimate: Storage
Storage available for filling dam	Total (MG)	329		165	
	Total (AF)	1,010		500	
Storage needed to meet demand projections	Total (MG)		275		716
	Total (AF)		840		2,200

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APPENDIX A

Supply Versus Demands for 2025 Demands and Ultimate 12 mgd Maximum Day Demands

Using 90% Exceedence Value for South Fork Mill Creek and 50% Exceedence Value for Dog River

											2025 Conditions		Ultimate, 12 mgd Maximum Day Conditions	
Month	Mill Creek (mgd)	Dog Creek (mgd)	Mill Creek plus Dog Creek Discharges (mgd)	Channel Losses, between dam and WTP intake (mgd)	Fish bypass flow (mgd)	Total available supply (mgd) = SFMC + DR - channel loss - fish bypass (mgd)	2025 Projected Average Daily Demands by Month (mgd)	Ultimate Daily Demands by Month for a 12 mgd Max Day (mgd)	Total Available Supply for 2025 demands (mgd)	Total Available Supply for 12 mgd Maximum Day (mgd)	Surplus available for filling dam (mg)	Storage needed in dam to meet demands (mg)	Surplus available for filling dam (mg)	Storage needed in dam to meet demands (mg)
Jan	2.8	2.7	5.4	0.0	3.0	2.4	2.2	3.2	0.2	-0.8	6	0	0	24
Feb	3.7	4.9	8.6	0.0	3.0	5.6	2.4	3.5	3.2	2.1	96	0	63	0
Mar	6.5	3.7	10.1	0.0	3.0	7.1	2.4	3.5	4.7	3.7	142	0	110	0
Apr	3.8	3.7	7.5	0.0	3.0	4.5	3.1	4.5	1.4	0.0	42	0	0	0
May	6.6	9.0	15.6	0.0	3.0	12.6	4.3	6.2	8.4	6.5	251	0	194	0
Jun	6.7	11.0	17.7	0.3	1.0	16.4	5.5	8.0	10.9	8.4	327	0	253	0
Jul	4.3	4.6	8.9	0.5	1.0	7.4	6.7	9.6	0.8	-2.2	23	0	0	67
Aug	3.9	2.5	6.3	0.7	1.0	4.6	6.0	8.7	-1.4	-4.1	0	42	0	123
Sep	3.3	1.7	5.1	0.5	1.0	3.6	4.8	6.9	-1.2	-3.4	0	37	0	101
Oct	2.7	3.0	5.6	0.0	1.0	4.6	3.2	4.6	1.5	0.0	44	0	1	0
Nov	3.1	1.5	4.5	0.0	3.0	1.5	2.3	3.4	-0.8	-1.8	0	24	0	55
Dec	2.5	1.6	4.2	0.0	3.0	1.2	2.1	3.1	-1.0	-2.0	0	30	0	59

		2025 Surplus	2025 Storage	Ultimate Surplus	Ultimate Storage
Storage available for filling dam	Total (MG)	931		621	
	Total (AF)	2,860		1,910	
Storage needed to meet demand projections	Total (MG)		133		429
	Total (AF)		410		1,320

CVO\053560014

Recommended Water Rights Actions for The Dalles to Implement to Expand Groundwater Use

TO: City of The Dalles

FROM: Adam Sussman and Paul Berg/CH2M HILL

DATE: November 28, 2005

PROJECT NUMBER: 320724.A1.02

Introduction

This memorandum presents a preliminary evaluation of the opportunity and needed steps for the City of The Dalles to pool existing groundwater rights in order to allow installation of a new well that may be located near the existing Lone Pine Well. For purposes of this memo, the new well is named Lone Pine 2 Well, with the existing well named Lone Pine 1 Well.

In preparing this preliminary evaluation a number of assumptions were made:

- The subject wells develop water from the same source;
- The Dalles Critical Groundwater Area does not preclude water right transfers;
- Water right transfers can be made without injury to existing water rights;
- The place of use is similar for all the subject water rights;
- Water not perfected in the Mill Creek Well transfer T-7258 (to Marks Well) is available;
- All of the subject water rights are protected from forfeiture as municipal rights (ORS 540.610 (2)(a)); and
- The City Hall Well can be modified under HB 2123 (2005 legislative session).

These assumptions and water right specific information should be evaluated in greater detail at the time when the city commences with these actions.

Discussion

Table 1 summarizes the potential for Lone Pine 2 Well by pooling the city's existing groundwater rights. It appears that the existing rights allow for a new well with a capacity of up to 3,791 gpm (5.5 mgd), which is in excess of the expected yield of a new well in this area.

Well	Water Right	Water Right Capacity (gpm)	Current Pumping Capacity (gpm)	Potential Water for a New Lone Pine Area Well (gpm)
Lone Pine Well	Cert 60026	2000	1600	400
Jordan Well	Cert 48991	2468	1950	518
Marks Well	Cert 15543	1203	1300	0
	T-7258	+		
		673 from T-7258		
City Hall Well	GR 4107	2300	0	2300
Mill Creek Well	Cert 44783	0	0	573
	T-7258	Water transferred to Marks Well under T-7258		Water not perfected at Marks Well under T-7258
Total Potential for new Lone Pine area well				3791

Recommended Actions

- Lone Pine Well (Certificate No. 60026) – Submit a water right transfer to add an additional point of appropriation which would allow the city to use its full water right capacity.
- Jordan Well (Certificate No. 48991) – Submit a water right transfer to add Lone Pine 1 and 2 Wells and Marks Well as additional points of appropriation. This would allow the city to use its full water right capacity.
- Marks Well (Certificate No. 15543) – The current pumping capacity exceeds the water right by approximately 97 gpm. Submit a water right transfer to add Lone Pine and Jordan Wells as additional points of appropriation. Explore submitting proof on 97 gpm from the Mill Creek Well transfer (T-7258) with the intention of moving the remaining Mill Creek Well water (573 gpm) to the Lone Pine 2 Well.
- City Hall Well (GR 4107) – Using the process under HB 2123 (2005 legislative session), which allows modification of Ground Water Registrations, transfer the existing water right capacity of 2300 gpm to the Lone Pine Wells.
- Mill Creek Well (Certificate No. 44783 and T-7258) – Explore transferring water not perfected in the transfer to the Marks Well to other wells. This water right is for 673 gpm and only approximately 97 gpm can be perfected under T-7258 at the existing Marks Well. Therefore, the city could propose a change in point of appropriation for the remaining 573 gpm to Lone Pine 1 and 2, Jordan, and Marks Wells.

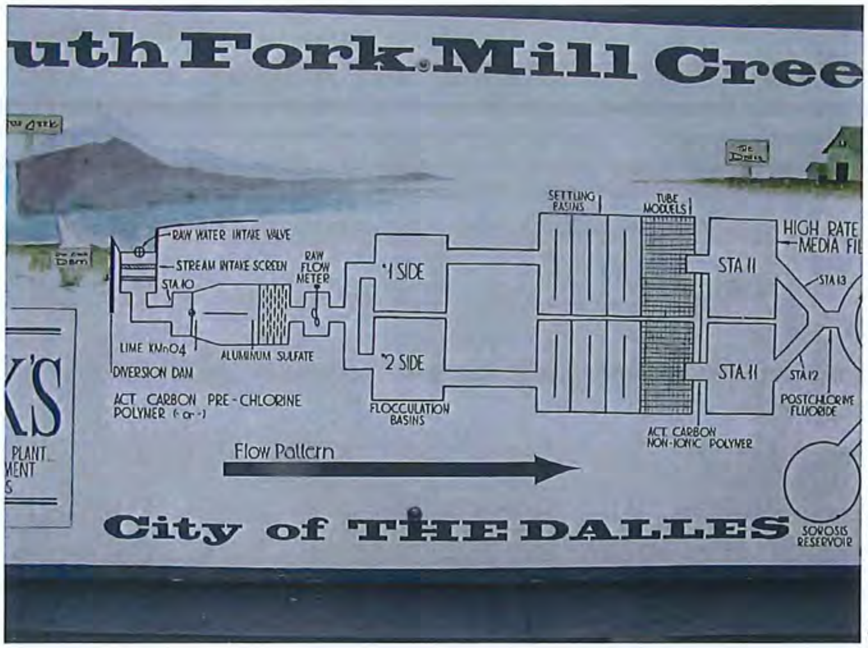
APPENDIX C

Wicks Water Treatment Plant: Process and Facility Review

Introduction

The Wicks plant was toured on November 2, 2004. Photo C-1 shows the entrance. Photo C-2 shows the process flow schematic.

C-1



C-2

Headworks

The headworks is a concrete structure that includes the spilling chamber, the grit chamber and the rapid mix (Photo C-3). There is approximately 15 feet of head available at the head of the WTP.


C-3

The water enters a spilling chamber (Photo C-4) where a manually operated slide gate is adjusted to select the desired plant flow. Excess water (the difference between the intake flow and the plant production rate) spills back into the creek.


C-4

Chlorine is added at the front of the plant unless powdered activated carbon is being used for the control of tastes and odors. The use of powdered activated carbon is infrequent.

The existing grit chamber (Photo C-5) has not really been needed since the fish screens were installed. The fish screens have 3/32-inch slots and remove most of the small debris that used to settle out in the grit chamber.


C-5

The rapid mix is currently composed of a series of baffles as shown in Photo C-6. The lower turbulence that occurs at low flows results in a need for increased alum dosages (approximately 2 mg/L higher than needed for higher flows). Field measurements showed about 18 feet of space available between the rapid mix and the flocculation to install two parallel static mixers.

C-6

The concrete appears to be in good condition. (No specific evaluation of the concrete was performed; this comment is based on general observation.)

Flocculation and Clarification

There are two flocculation/clarification basins. Flocculation is provided by hydraulic flocculators (Photo C-7). There is a concrete overlay that appeared to serve as a mount for flocculation drives at one point. The openings have now been plugged.


C-7

Sludge is removed by a Trac Vac system. The operators report that it works well. Tube packs have been installed on the last third of each basin. The Track Vac system cannot reach all areas of the settling basin. These areas must be cleaned with a fire hose every 2 to 3 months. These tube packs are covered by a concrete ceiling so the operators must climb down inside and spray the tube packs with a fire hose. It is likely that this cover limits algae growth in the summer and decreases the degradation of the plastic tubes from UV light. Photos C-8 through C-10 show various views of the clarification tanks.

C-8

C-9

C-10

Filtration

The plant has two filters containing a tri-media design. Photos C-11 and C-12 show the inside of the filter building.

C-11

C-12

The backwash procedures are as follows:

- The filters are turned off and drained
- 2 minutes of surface wash
- Backwash flow ramps up in 600 gpm increments to 2500 gpm
- At 2500 gpm the surface wash is turned off
- The backwash flow is then increased to 4800 gpm (16 gpm/sf)
- The backwash continues at this rate for about 7 minutes
- Then a second surface wash is performed for 1-½ minutes. Operations staff noted that this second surface wash releases a surprising amount of dirt.
- The backwash continues for another 5 minutes until the filter is clean
- The flow is then ramped down in three 1500 gpm steps.
- The filters are directed to waste for 10 minutes.

Backwash water is supplied from two backwash tanks holding 50,000 gallons each (Photo C-13). The BW line is 18 inches ID. These washwater tanks are filled with two pumps (lead, lag), having a capacity of 375 gpm each.

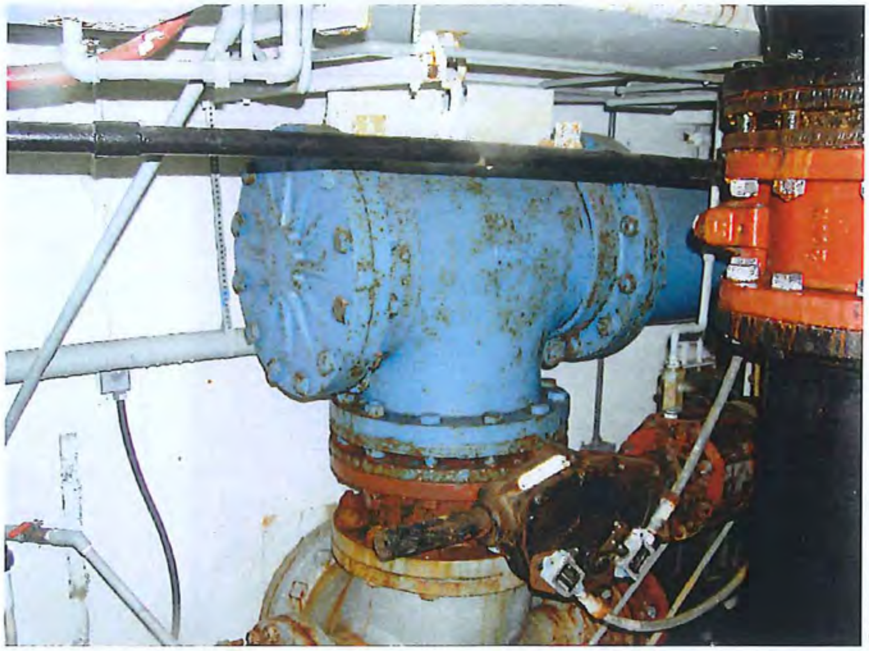
C-13

When a filter is taken out of service to be backwashed, the influent flow to the plant is decreased to avoid overflowing the clarification basins.

The filter building was constructed to allow expansion with reinforcing bar sticking out of the wall on the north side (Photo C-14).

C-14

The pipe gallery is also set up for expansion, with endcaps on tees that can be removed to allow more pipe to be added (Photo C-15)



C-15

Chemical Feed

Alum is supplied from one large tank that hold two truck loads. There are three alum feed pumps that are near their capacity at the high plant production rates.

Phosphate for corrosion control is fed from a 50-gallon drum and this technique works well (Photo C-16).

C-16

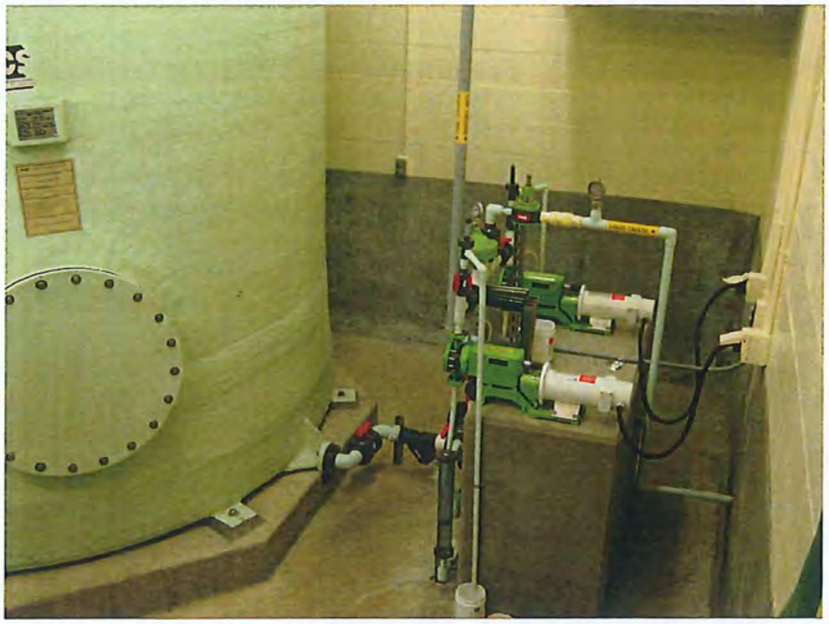
Chlorine and fluoride are fed from locations separate from the filter building. The chlorine facilities store five 1-ton cylinders (Photo C-17) and chlorinators. The chlorinators are 1960s vintage and finding replacement parts is difficult. The building with chlorine and fluoride feed also contains the washwater pumps.

C-17

The fluoride feed facilities are located in the same building as the chlorine feed facilities. Fluoride is fed in the form of sodium silicofluoride (Photo C-18).

C-18

Caustic is fed to the finished water for pH control from a separate building (Photo C-19). If an alternative coagulant such as aluminum chlorohydrate or polyaluminum chloride were used, then caustic may not be needed and the caustic feed facilities might be able to be converted to hypochlorite feed.



C-19

Sludge Lagoons

There is one lagoon that is divided into two halves. The overall dimension is a width of 35 feet, a length of 145 feet, and a depth of 11 feet (Photo C-20). The lagoon receives the underflow from the clarification basins and the backwash water. Chlorine is a problem because the decant is released to South Fork Mill Creek. The current Oregon Department of Environmental Quality (DEQ) 200J permit limits the total suspended solids (TSS) to less than 0.1 mg/L and the pH to between 6 and 9 units. The city is able to comply with these standards; however, the DEQ permit is due for reauthorization and the draft of the revised permit includes limits for chlorine and temperature. Both will be difficult to meet. If the primary point of chlorination addition were moved to downstream of the filters, it is likely that the chlorine problem would be avoided. With this change, the only chlorinated water entering the lagoon would come from the filter backwashing. The filter-to-waste and the underflow from the clarification basins would have no chlorine.

The sludge lagoons are dredged once per year.

C-20

APPENDIX D

Wicks Water Treatment Plant: Analysis of Expansion Options

This appendix presents a description of alternative technologies that were considered as feasible options:

- Membrane Filtration
- Dissolved Air Flotation (DAF)
- Upflow Solids Contact Units
- Sand-ballasted Sedimentation
- Lamella Plate Clarification

Membrane Filtration

The use of membranes for drinking water supplies is being driven by the desire for improved water quality to meet regulatory and consumer requirements, the availability of cost-competitive large-capacity membrane systems, and other factors. As advances in membrane technology continue, membrane treatment becomes increasingly cost-competitive. New membrane developments have resulted in systems with lower power requirements and longer membrane lives. Microfiltration (MF) and ultrafiltration (UF) membrane products are challenging conventional treatment for many surface water applications.

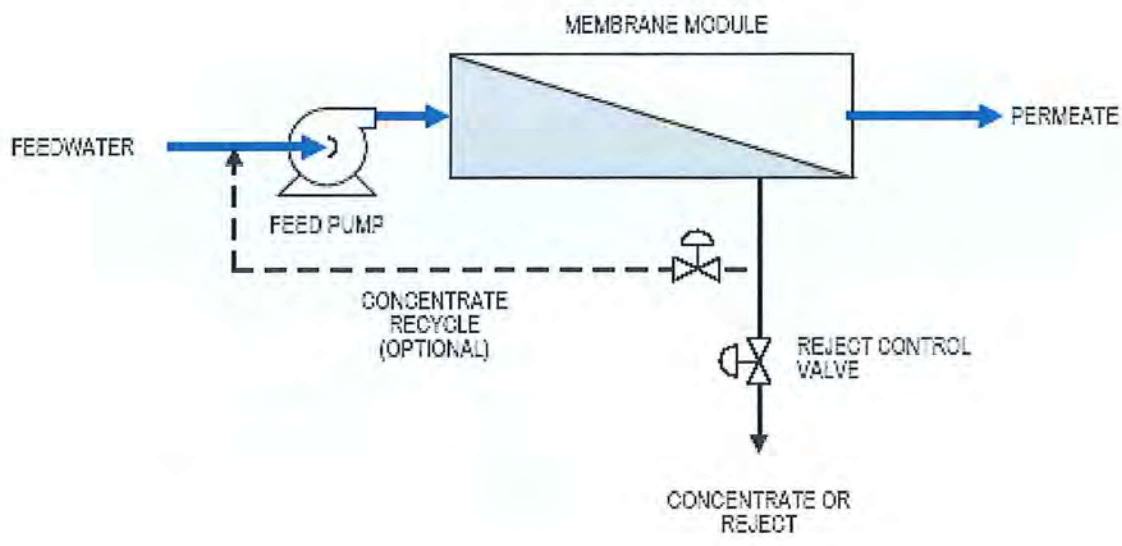


EXHIBIT D-1
Simplified Flow Schematic of Membrane Process

In membrane filtration, all particles that are more than 0.2 microns in size are physically screened out. The water is pumped at a pressure of 10 to 40 psi through the membranes. In most cases, a coagulant is not used. Exhibit D-1 shows a simple schematic of a membrane process.

Operationally, membranes are different to operate than a conventional treatment plant. Membranes do not normally require an understanding of coagulant chemistry, but do require periodic cleaning. Normally, membrane filtration is easier to operate than a conventional plant. However, expanding the Wicks Water Treatment Plant (WTP) with membranes will result in a need to operate two different plants: a conventional treatment plant, and a membrane plant. Because the operations staff is already well-versed in the operation of a conventional plant, CH2M HILL recommends that the plant be expanded using a process that uses media filtration rather than membrane filtration.

Dissolved Air Flotation

DAF was first used as a pretreatment for conventional granular media in South Africa and Scandinavia in the 1960s and became more widely used worldwide in the 1980s and 1990s. DAF is becoming more common in the U.S. because it provides a cost-effective alternative to conventional sedimentation, when applicable. DAF has also been successfully used to remove algae.

In DAF, the solids are separated out by floating the floc to the water surface, as opposed to settling to the bottom of the basin. The process introduces air bubbles at the bottom of the contactor to float the floc. The air bubbles are produced by reducing to ambient pressure a pressurized recycle water stream saturated with air. The "float" is scraped from the top of the reactor, and the clarified water is removed via laterals at the bottom of the reactor. A schematic of a typical DAF unit is provided in Exhibit D-2.

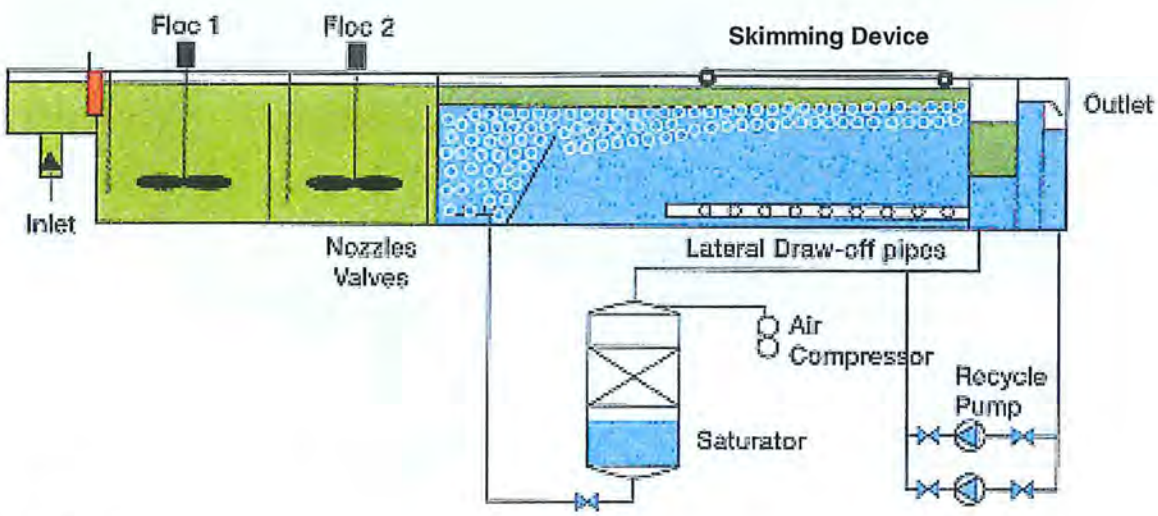


EXHIBIT D-2
DAF Schematic

DAF is particularly effective in removing solids, such as algae, which are close in density to that of water and, thus, are resistant to removal by sedimentation. DAF has been shown to be as effective as conventional processes at removing turbidity and total organic carbon (TOC) for selected waters.

DAF requires less area than conventional flocculation-sedimentation for two reasons: the flocculation section is half the size (or less) of a conventional process and the surface loading of the solids separation part of the process can be as high as 8 gpm/ft^2 (there are some unproven designs that have increased the loading rate to approximately 13 gpm/ft^2). Detention times required for both flocculation and clarification are less than in conventional treatment. This results in a much smaller reactor than is possible for a conventional process. DAF also produces a more concentrated sludge than conventional treatment, although the sludge may contain entrapped air and need to be de-aerated.

DAF is currently installed at roughly 15 plants in the U.S. for drinking water treatment. Exhibit D-3 lists the plants in the U.S. with capacity greater than 0.5 mgd.

EXHIBIT D-3
DAF Installations in the U.S. on Surface Waters

Plant	Location	Start Date	Capacity (MGD)
Millwood WTP	New Castle, NY	1993	7.5
Beaver Run WTP	Westmoreland, PA	1995	3.5
Danbury WTP	Danbury, CT	1998	5.5
Rockport WTP	Rockport, MA	1998	1.2
Tazewell RWA	Tazewell County, VA	1999	2.0
Lee Hall WTP	Newport News, VA	2000	52.0
Penn Hill WTP	West Chester, PA	1998	3.0
Table Rock WTP	Greenville, SC	1999	75.0
Fresh Pond WTP	Cambridge, MA	2000	24.0
Wangum WTP	Norfolk, CT	1996	0.5
Lakeville WTP	Lakeville, CT	1996	0.5
Hemlocks WTP (BHC)	Fairfield, CT	1997	50.0

Upflow Solids Contact Units

Solids contact units are frequently known as upflow clarifiers. They combine flocculation and sedimentation in one unit. Solid contact units are designed to maintain a large volume of flocculated solids within the unit, which enhances flocculation by encouraging interparticle collisions. The flocculated solids (solids blanket) are usually maintained at a set volume in the contactor and cohesion of the blanket is achieved through the use of a polymer in addition to the coagulant.

Upflow clarifiers are popular because of their reduced size. Consequently, they are more compact and occupy less land. Higher surface loading rates than in conventional treatment can be used to produce more water per unit area. One such unit is the Superpulsator® manufactured by ONDEO Degremont, Inc. (formerly Infilco Degremont, Inc.), Richmond, Virginia. A schematic diagram of an upflow clarifier is provided in Exhibit D-4.

Rapid mixing occurs upstream of the unit where a coagulant is added to begin the formation of floc. After rapid mixing, a polymer is added to promote sludge blanket cohesion. The coagulated water then enters the unit. The Superpulsator® uses a vacuum pump and vacuum chamber to produce a "pulsing" effect within the flocculation zone (i.e., solids blanket). The pulsing of the solids blanket expands the blanket and increases the rate of interparticle collisions. Inclined plates are used in the solids area to assist in water and solids distribution and contacting. Clarification occurs above the sludge blanket. Clarification can be assisted with the use of tubes. The clarified effluent is discharged at the top of the unit. Solids are maintained in the unit at a set height by use of a solids overflow weir. Solids are overflowed into a hopper and can be removed at a set interval. The sludge hopper is sloped to act as a sludge thickener as well. Typical solids concentrations range from 0.5 to 2 percent in the concentrated sludge, depending on the solids residence time.

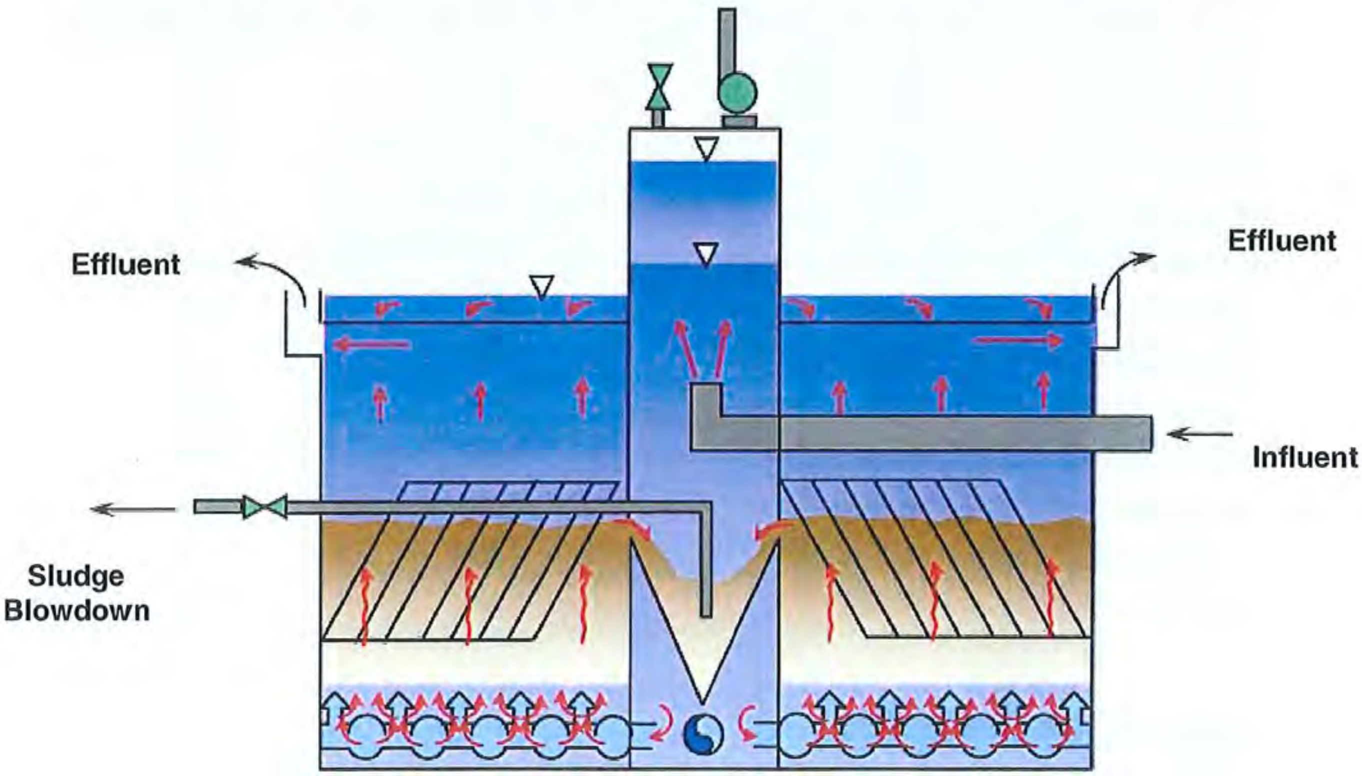


EXHIBIT D-4
Upflow Clarifier Schematic

These units do not tolerate rapid changes in raw water temperature or hydraulic loading (flow rate). Detention time is lower in this unit (1 hour or less at typical loading rates) than in a conventional process, therefore requiring more operator attention during changing raw water quality conditions. A polymer is required at doses between 0.01 to 0.4 mg/L for cohesion of the sludge blanket.

These units have no submerged moving parts or mechanisms, and the sludge blanket is self-leveling. Typical surface loading rates for the Superpulsator® can range from 1.5 to 4 gpm/ft² for water treatment, requiring much less surface area for equivalent treatment as compared to conventional processes (especially because separate flocculation is not required). These units have been shown effective at removing turbidity, TOC, and low-to-medium algae concentrations. Because of the long retention time of solids (24 hours), use of powdered activated carbon (PAC) is particularly effective at removing taste and odor (T&O) -causing compounds in these units. Along with T&O-causing compounds, disinfection by-product (DBP) precursors, algal toxins, and synthetic organics can also be adsorbed in the solids blanket.

There are about 70 municipal water treatment plants in the U.S. that are successfully using this technology, and it has become an accepted standard clarification process in many states. Exhibit D-5 lists some of the major Superpulsator® process installations in the U.S.

EXHIBIT D-5
Select Superpulsator® Installations in the U.S. on Surface Waters

Plant	Location	Start Date	Capacity (mgd)	Source Water
Naugatuck WTP	Naugatuck, CT	April 1989	6.0	Surface
Pistapaug Pond WTP	Wallingford, CT	July 1993	12.0	Surface
Augusta WTP	Augusta, ME	September 1993	5.5	Surface
Hingham WTP	Hingham, MA	January 1996	7.6	Reservoir
NJ American Tri-County WTP	Delran, NJ	April 1995	30.0	River
Middlesex Water Company	Islin, NJ	November 1998	45.0	River
Hyde Park Fire & Water District	Dutchess County, NY	December 1994	6.0	Reservoir
Hays Mine WTP	Pittsburgh, PA	May 1990	60.0	Surface
Norristown WTP	Norristown, PA	May 1997	12.0	River
Hershey WTP	Hershey, PA	April 1992	9.0	Surface

Sand-ballasted Sedimentation

Actiflo® is a proprietary process of high-rate clarification that uses microsand-enhanced flocculation and lamella settling to produce a clarified effluent. The process consists of a rapid mix where a coagulant is added, followed by an injection tank where microsand and a polymer are added in a high-energy mixing environment. Following this is a maturation zone, where a lower-energy mixing takes place to build the floc and attach it to the sand. The detention time for all these steps is about 6 minutes. The water then enters the settling tank where the microsand flocs settle out quickly, and it is further clarified with tube settling before overflow into the effluent channels. Total retention time is between 10 and 15 minutes. A schematic of this process is provided in Exhibit D-6.

The microsand sludge at the bottom of the settling tank is pumped to a hydrocyclone where it is separated from the sludge by centrifugal force. The sand is then returned to the head of the process for reintroduction in the injection tank. The separated sludge is removed at concentrations of 0.1 to 0.2 percent for further treatment.

Advantages of this process include very high loading rates (15 to 30 gpm/ft^2) that can significantly reduce surface area requirements. The process can also be retrofitted in many cases into existing tanks, thereby reducing the need for construction of additional tankage. The use and high loading of microsand allows the system to easily adjust to changing raw water quality or process flow rates. However, like other clarification systems, as the raw water characteristics change a corresponding change in the coagulant may be required. This is the same for all clarifiers.

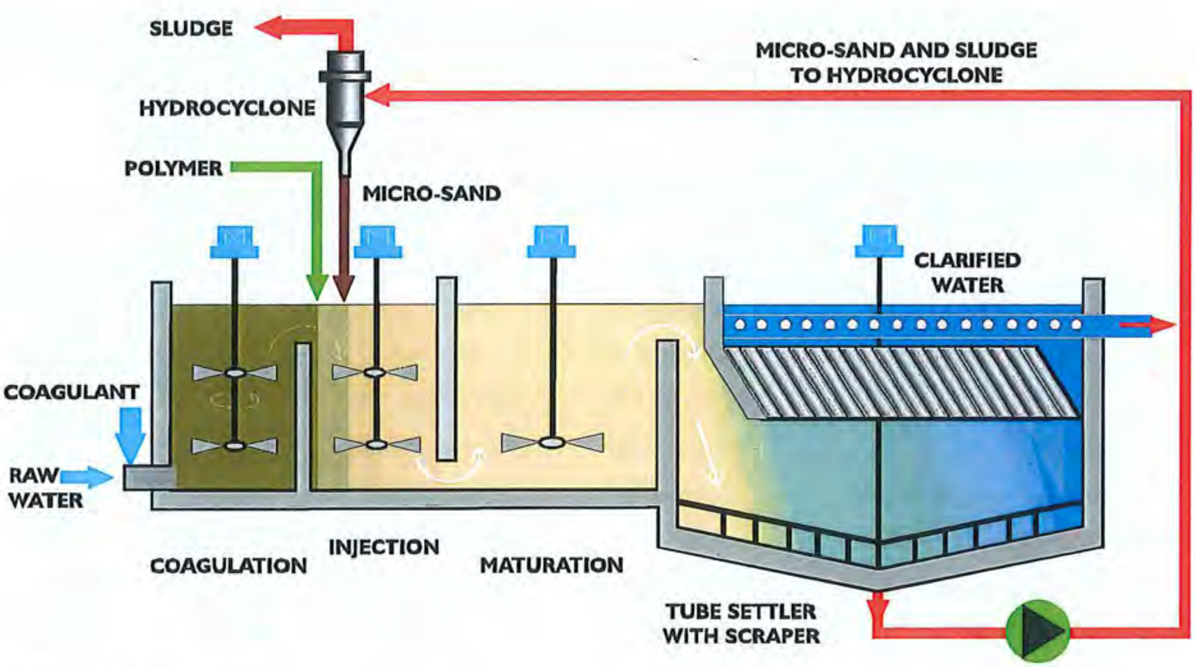


EXHIBIT D-6
Actiflo® Process Schematic

The system requires a significant amount of energy beyond other conventional processes (with the exception of DAF). The microsand must be replenished at regular intervals because of some loss in the separation process.

Originally this process began to appear in full-scale plants in Europe and Canada. In the past few years, numerous installations have been put in operation in the U.S., the largest of which is a 74-mgd installation in Tampa, Florida. Exhibit D-7 lists some of the Actiflo® installations currently in operation or under construction in the U.S. The Wilsonville plant, which treats water from the Willamette River, uses this technology.

EXHIBIT D-7
Actiflo® U.S. Installations

#	Plant/Location	Startup	Total MGD	Number of Trains	Application
1	N. Table Mountain, CO	1998	11	2	Potable
3	Casper, WY	1999	27	2	Potable
4	Newport, KY	1999	15	2	Potable
7	Sharon, PA	2000	16	3	Potable
8	Spotsylvania, VA	2000	12	4	Potable
9	Southeast Regional, UT	2000	20	2	Potable
18	Tampa, FL	2001	40	2	Potable
20	Wilsonville, OR	2001	15	2	Potable
23	Tampa Bay, FL	2001	74	2	Potable
24	Melbourne, FL	2001	20	2	Potable
29	Foothill, CA	2002	40	2	Potable
31	Atchison, KS	2002	10	2	Potable
33	Morehead, KY	2002	10	2	Potable
34	Harlingen, TX	2002	20	2	Potable
39	Fresno, CA	2002	20	2	Potable
48	Passaic Valley, NJ	2003	120	4	Potable

This process responds exceptionally well to changes in water quality, and has consistently demonstrated its ability to accommodate very high solids loading while producing a settled water turbidity of 0.2 to 0.8 nephelometric turbidity unit (NTU). In addition, its ability to operate with relatively high dosages of coagulant makes it an excellent choice for enhanced coagulation. The process has also been shown effective in treating low-to-medium algae concentrations in the raw water.

The process can be difficult to operate because the high polymer dose required to attach suspended particles to the microsand can cause rapid headloss development in the filters.

Inclined (Plate and Tube) Settlers

Some WTPs use inclined settlers as an alternative to conventional sedimentation after flocculation. Inclined settling is accomplished using plates or tubes in a tank, where the water flow is either countercurrent, co-current, or cross-current to produce a clarified effluent.

Countercurrent inclined settlers apply the flocculated water upward through the channels formed by the inclined surfaces. Co-current settlers have the flow fed at the upper end of the inclined surface with flow down through the chamber. Solids and clarified water flow in the same direction but at different velocities. This is the least-used method of inclined settling. In cross-current settling, the flow is fed horizontally between the inclined surfaces, while the solids settle to the bottom. Most new parallel plate settlers use a combination of cross- and countercurrent flow by introducing the water at the side of the plates near the bottom.

The advantage to inclined settlers is that increased surface loading rates (4 to 6 gpm/ft^2) can be used to achieve proper settling. In addition, the plates or tubes can be retrofitted to an existing sedimentation tank. A schematic of the cross-flow/countercurrent lamella plate alternative is shown in Exhibit D-8.

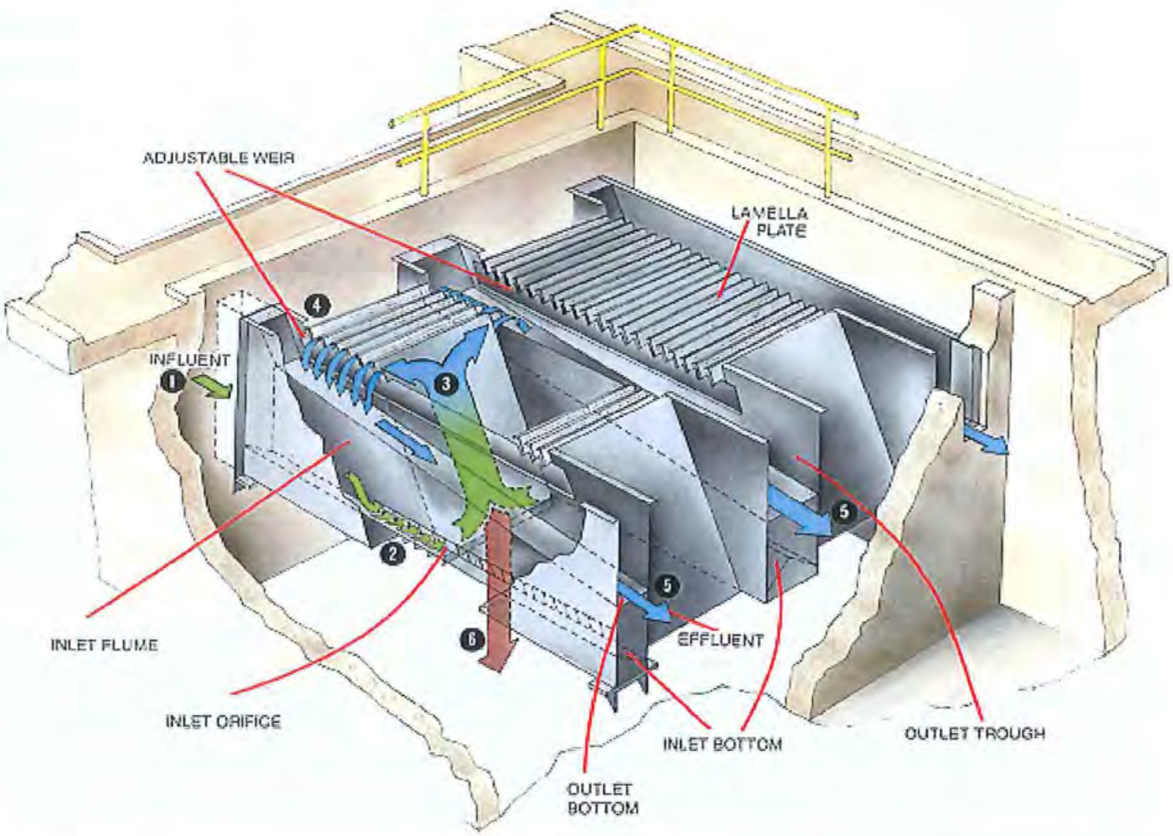


EXHIBIT D-8
Countercurrent Lamella Plate Clarifier

The material costs for the plates or tubes can vary depending on the materials required for the installation. The surface loading rates are not as high as other processes (other than conventional clarification). Solids loading on surfaces and removal of solids can be a problem in some configurations. Much of the conventional process is retained by the use of this technology, and in a new installation this then requires a large flocculation tank with a level of mechanical equipment similar to conventional coagulation and sedimentation.

Plate and tube settlers have been in use for many years in water treatment plants and are a widely accepted technology for settling flocculated solids. They are currently being used very successfully by utilities in Corvallis and Fort Collins on very similar waters.

APPENDIX E
Dog River Diversion Pipeline Replacement Cost Estimate
(November 2004 costs, ENR CCI Seattle = 7562)

Item	Quantity	Units	Material Unit Cost	Material Cost	Labor Unit Cost	Labor Cost	Total Unit Cost	Total Cost	Basis
General Requirements (insurance, mobilization, bonds, overhead, etc)	1	LS		$125,000		$76,000		$201,000	Allowance @ 8% of Construction Total
Pipeline Alignment Clearing	18,500	LF	0.00	0	3.43	63,500	3.43	63,500	Means 05 BCCD 02230 100 Assuming 30' wide
Trench Excavation--Rocky Soils	26,700	CY	0.00	0	1.80	48,200	1.81	48,200	Means 05 BCCD 02315 424 0020 & 0260 2CY Hyd Hoe
Pipe Bed & Zone--Import Matls On Site	8,130	CY	15.00	122,000	10.03	81,500	25.03	203,500	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Pipe Bed & Zone--Install/Vib Plate Compact	8,130	CY	0.00	0	3.82	31,100	3.83	31,100	Means 05 BCCD 02315 640 0500
Fill Above Pipe Zone--Native Matls	16,440	CY	0.00	0	8.36	137,400	8.36	137,400	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Ductile Iron Pipe--24" Dia Class 250	18,500	LF	70.25	1,299,600	27.25	504,100	97.50	1,803,700	Per American Matls Quote + 25% & Means 05 BCCD 02510 730
Riprap at Outfall	40	CY	40.00	1,600	8.63	300	47.50	1,900	Means 05 BCCD 02370 450 0200
Air Relief Valve in Vault	1	EA	5,000	5,000	2,500	2,500	7,500	7,500	Allowance
Low Point Blowoffs	1	EA	1,000	1,000	1,500	1,500	2,500	2,500	Allowance
Connection to Existing Concrete Vault and repair of existing inlet vault	1	EA	4,000	4,000	6,000	6,000	10,000	10,000	Allowance
						Construction Subtotal		**$2,510,000**	
						Construction Contingency at 20%		$502,000	
						Construction Total		**$3,010,000**	
						Permitting allowance		$150,000	
						Engineering design and construction services allowance (10% of construction total)		$301,000	
						Project Total		**$3,461,000**	

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

APPENDIX EP2
Dog River Diversion Pipeline Replacement Cost Estimate
(November 2004 costs, ENR CCI Seattle = 7562)

Item	Quantity	Units	Material Unit Cost	Material Cost	Labor Unit Cost	Labor Cost	Total Unit Cost	Total Cost	Basis
General Requirements--Complete	1	LS		$142,000		$82,000		$224,000	Allowance @ 8% of Project Total Cost
Pipeline Alignment Clearing	18,500	LF	0.00	0	6.87	127,100	6.87	127,100	Means 05 BCCD 02230 100 Assuming 60' wide
Trench Excavation--Rocky Soils	26,700	CY	0.00	0	1.80	48,200	1.81	48,200	Means 05 BCCD 02315 424 0020 & 0260 2CY Hyd Hoe
Pipe Bed & Zone--Import Matls On Site	8,130	CY	15.00	122,000	10.03	81,500	25.03	203,500	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Pipe Bed & Zone--Install/Vib Plate Compact	8,130	CY	0.00	0	3.82	31,100	3.83	31,100	Means 05 BCCD 02315 640 0500
Fill Above Pipe Zone--Native Matls	16,440	CY	0.00	0	8.36	137,400	8.36	137,400	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
HDPE Pipe--24" Dia SDR 13.5	18,500	LF	80.85	1,495,700	27.25	504,100	108.10	1,999,800	BCCD 02510 730
Riprap at Outfall	40	CY	40.00	1,600	8.63	300	47.50	1,900	Means 05 BCCD 02370 450 0200
Air Relief Valve in Vault	1	EA	5,000	5,000	2,500	2,500	7,500	7,500	Allowance
Low Point Blowoffs	1	EA	1,000	1,000	1,500	1,500	2,500	2,500	Allowance
Connection to Existing Concrete Vault and repair of existing inlet vault	1	EA	4,000	4,000	6,000	6,000	10,000	10,000	Allowance
						Construction Subtotal		**$2,793,000**	
						Construction Contingency at 20%		$559,000	
						Construction Total		**$3,350,000**	
						Permitting allowance		$150,000	
						Engineering design and construction services allowance (10% of construction total)		$335,000	
						Project Total		**$3,835,000**	

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

APPENDIX F

Finished Water Transmission Pipeline Replacement Cost Estimate

(November 2004 costs, ENR CCI Seattle = 7562)

Item	Quantity	Units	Material Unit Cost	Material Cost	Labor Unit Cost	Labor Cost	Total Unit Cost	Total Cost	Basis
General Requirements (insurance, mobilization, bonds, overhead)--Complete for FW and Branch Pipelines	1	LS		$370,000		$240,000		$610,000	Allowance @ 8% of Project Total Cost
Finished Water Transmission Pipeline									
Traffic Control--Flagmen @ One Way Traffic	201	DY	0.00	0	600.00	120,600	600.00	120,600	Assumed production of 200'/shift/crew
Lighted Signage/Barricades	280	DY	100.00	28,000	100.00	28,000	200.00	56,000	Allowance for subcontracted signage w/ crew maintenance
Sawcut Asphalt Pavement	160,800	LF	0.18	28,500	0.51	82,300	0.69	110,800	Means 05 BCCD 02220 380 0015
Demolish/Load AC Pavement--6' wide	241,200	SF	0.00	0	0.47	113,200	0.47	113,200	Means 05 BCCD 02220 250 5010
Truck Haul AC Spoils To Disposal Site	3,000	CY	10.00	30,000	4.82	14,500	14.83	44,500	Means 05 BCCD 02315 490 Using 12CY trucks 2 mile round trip
Trench Excavation--Rocky Soils	39,900	CY	0.00	0	1.87	74,500	1.87	74,500	Means 05 BCCD 02315 424 0020 & 0260 2CY Hyd Hoe to Trucks
Truck Haul Excavation To Disposal Site	45,900	CY			4.82	221,100	4.82	221,100	Means 05 BCCD 02315 490 Using 12CY trucks 2 mile round trip
Pipe Bed & Zone--Import Matls On Site	14,310	CY	15.00	214,700	10.03	143,500	25.03	358,200	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Pipe Bed & Zone--Install/Vib Plate Compact	14,310	CY	0.00	0	3.82	54,700	3.82	54,700	Means 05 BCCD 02315 640 0500
Fill Above Pipe Zone--Imported Matls CL "D"	20,940	CY	15.00	314,100	10.03	210,000	25.03	524,100	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Fill Above Pipe Zone--Install Imported Matls	20,940	CY	0.00	0	3.82	80,000	3.82	80,000	Means 05 BCCD 02315 640 0500
Ductile Iron Pipe--24" Dia Class 350	32,160	LF	77.81	2,502,500	27.25	876,300	105.06	3,378,800	Per American Matls Quote + 25% & Means 05 BCCD 02510 730
Ductile Iron Pipe--24" Dia Class 250	8,040	LF	70.25	564,800	27.25	219,100	97.50	783,900	Per American Matls Quote + 25% & Means 05 BCCD 02510 730
AC Pavement Replacement--4" thick x 6' wide	26,800	SY	8.84	237,000	1.84	49,300	10.68	286,300	Means 05 BCCD 02740 310 0210 & 0380
Creek Crossings	5	EA	10,000	50,000	40,000	200,000	50,000	250,000	Allowance
Isolation Valves in Vaults	16	EA	5,000	80,000	2,500	40,000	7,500	120,000	Allowance
Low Point Blowoffs	6	EA	1,000	6,000	1,500	9,000	2,500	15,000	Allowance
Air & Vacuum Relief Valves in Vaults	8	EA	2,000	16,000	1,500	12,000	3,500	28,000	Allowance
Branch (Customer) Supply Pipelines									
Traffic Control--Flagmen @ One Way Traffic	14	DY	0.00	0	600.00	8,400	600.00	8,400	Assumed production of 200'/shift/crew
Lighted Signage/Barricades	20	DY	100.00	2,000	100.00	2,000	200.00	4,000	Allowance for subcontracted signage w/ crew maintenance
Sawcut Asphalt Pavement	22,400	LF	0.18	4,000	0.51	11,500	0.69	15,500	Means 05 BCCD 02220 380 0015
Demolish/Load AC Pavement--6' wide	33,600	SF	0.00	0	0.47	15,800	0.47	15,800	Means 05 BCCD 02220 250 5010
Truck Haul AC Spoils To Disposal Site	410	CY	10.00	4,100	4.82	2,000	14.88	6,100	Means 05 BCCD 02315 490 Using 12CY trucks 2 mile round trip
Trench Excavation--Rocky Soils	3,900	CY	0.00	0	1.87	7,300	1.87	7,300	Means 05 BCCD 02315 424 0020 & 0260 2CY Hyd Hoe to Trucks
Truck Haul Excavation To Disposal Site	4,500	CY			4.82	21,700	4.82	21,700	Means 05 BCCD 02315 490 Using 12CY trucks 2 mile round trip
Pipe Bed & Zone--Import Matls On Site	1,170	CY	15.00	17,600	10.03	11,700	25.04	29,300	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Pipe Bed & Zone--Install/Vib Plate Compact	1,170	CY	0.00	0	3.82	4,500	3.85	4,500	Means 05 BCCD 02315 640 0500
Fill Above Pipe Zone--Imported Matls CL "D"	2,640	CY	15.00	39,600	10.03	26,500	25.04	66,100	Means 05 BCCD 02315 640 0050 Screened Bank Run Gravel
Fill Above Pipe Zone--Install Imported Matls	2,640	CY	0.00	0	3.82	10,100	3.83	10,100	Means 05 BCCD 02315 640 0500

APPENDIX F
Finished Water Transmission Pipeline Replacement Cost Estimate
(November 2004 costs, ENR CCI Seattle = 7562)

Item	Quantity	Units	Material Unit Cost	Material Cost	Labor Unit Cost	Labor Cost	Total Unit Cost	Total Cost	Basis
Ductile Iron Pipe--8" Dia Class 350	5,600	LF	12.40	69,400	11.97	67,100	24.38	136,500	Per American Matls Quote + 25% & Means 05 BCCD 02510 730
AC Pavement Replacement--4" thick x 6' wide	3,733	SY	8.84	33,000	1.84	6,900	10.69	39,900	Means 05 BCCD 02740 310 0210 & 0380
Isolation Valves in Vaults	2	EA	1,500	3,000	1,000	2,000	2,500	5,000	Allowance
Misc Service Connections	6	EA	1,000	6,000	1,500	9,000	2,500	15,000	Allowance

Finished Water Transmission Construction Subtotal (including prorated amount of General Requirements)	$7,196,000
Branch (Customer) Supply Pipelines Construction Subtotal (including prorated amount of General Requirements)	$419,000
Construction Subtotal	**$7,615,000**
Construction Contingency at 20%	$1,523,000
Construction Total	**$9,138,000**
Engineering design and construction services allowance (10% of construction total)	$914,000
Project Total	**$10,050,000**

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

APPENDIX G

Analyses and Recommendations for Increasing Use of Lone Pine Well

The city desires to increase production from the Lone Pine Well. Increasing production from this well is a component of the long-term supply plan (see separate Water Supply memo) and is important to meet needs this summer because of the 2005 drought.

The Lone Pine Well currently pumps at a rate of approximately 1,600 gpm (2.3 mgd). This well can potentially provide a significant portion of the city's expected maximum day demand of approximately 7 mgd if it is pumped continuously during the summer. Historically, the city has used the well only part-time during a few weeks per year. The city's limited use of the well was partly because the surface supply was adequate to meet demands and partly because of distribution limitations. The demands in the area of the well (in the 352 zone to which it pumps) are too low to make use of the 1,600 gpm capacity except for brief periods. There are inadequate piping connections from this zone to neighboring zones to make use of the well's production in other areas of the system.

The Columbia View Reservoir serves the 632 zone that is directly south of the 352 zone. Water is pumped from Lone Pine Well to the Intermediate Reservoir. The Intermediate Pump Station is fed either from the tank or directly from the 352 zone. The Intermediate Pump Station pumps water into the 632 zone and fills the Columbia View Reservoir. The city has not used Columbia View Reservoir for the past five years because distribution limitations resulted in stagnant water conditions in this tank. The Intermediate Pump Station is capable of filling the tank, but once filled, the emptying rate is too slow and the water in the tank develops a long water age.

Similar to the 352 zone, the 632 zone is mostly isolated from the zones to the west. The distribution system limits the contribution from Columbia View Reservoir in feeding zones other than the 632 zone. Because demands in the 632 zone are relatively low, the use from this tank is too low to obtain adequate turn-over. Increasing use from this tank will facilitate increased use of the Lone Pine Well. Therefore, the problems of limited production from Lone Pine Well and limited use from the Columbia View Reservoir are interrelated.

Hydraulic Analysis

The updated network model for the city's distribution system was recently completed. This model was used to determine the needed improvements to:

- Allow the Lone Pine Well to pump continuously during summer, high demand periods, at flows of up to 2,000 gpm

- Withdraw water from the Columbia View Reservoir at rates that are sufficient to provide adequate turn-over of the contents.

To significantly increase production from the Lone Pine Well, it is necessary to provide a pipeline connection from the 352 zone to the 310 zone (the area including the downtown and port). Summertime demands in the 310 zone currently exceed 3 mgd. This zone contains a large portion of the city's overall demands. Even with increased use from the Columbia View Reservoir, there is insufficient demand that can be fed from the Lone Pine Well without the connection to the 310 zone.

The city plans to install a portion of this pipeline prior to the peak demands in the summer of 2005. The pipeline will initially feed a large demand in the lower portion of the 395 zone. When extended in the future, it will provide a direct connection to the 310 zone.

A delivery of 1,400 gpm (2 mgd) was targeted through the 352-310 pipeline. This provides sufficient transfer to account for the possible addition of a second well in the Lone Pine Well area bringing the total supply to 2,800 gpm (4 mgd) in this zone. (The Water Supply memo discusses the possibility of increasing the pumping rate from Lone Pine Well or adding a second well in this vicinity.)

Additional production in the 352 zone also can be distributed in the system if the area served by the Columbia View Reservoir is expanded. Water is pumped from the 352 zone, through the Intermediate Pump Station, into the 632 zone. The most feasible approach to increasing withdrawals from this tank is to modify the interconnection between the 632 zone and the 660 zone to the west. This will enable the Columbia View Reservoir to serve part of the 660 zone. The 660 zone is currently fed only from Sorosis Reservoir.

The model was used to simulate alternative combinations of these improvements at maximum day and peak hour demands, both for current and buildout conditions.

Findings

The recommended improvements consist of the following:

1. Add a 16-inch pipeline to connect the 352 and 310 zones. **Exhibit 1** illustrates this line. It is approximately 6,400 feet long. The final length will depend on the actual routing. This will enable transfer of approximately 2 mgd from the 352 zone to the 310 zone, allowing increased use of Lone Pine Well.
2. Add an 8-inch pipeline within the 352 zone to facilitate moving water from Lone Pine Well to the new 16-inch line. **Exhibit 2** illustrates this line. It is approximately 2,600 feet long. The 352-310 transmission pipeline does not provide its full benefit without this 8-inch line.
3. Install a pressure reducing valve (PRV) in the 660 zone, with a setting of 60 psi. **Exhibit 3** shows its proposed location. This PRV drops the pressure in the eastern half of the 660 zone, which enables the Columbia View Reservoir to serve this area. The higher use from the Columbia View Reservoir allows the 632 zone to receive more water from the Lone Pine Well while providing sufficient turn-over in this tank to sustain high quality drinking water.

4. To accomplish the higher use from the Columbia View Reservoir, it is also necessary to shut down the 19th Street (Hospital) Reservoir. Eventually, when demands increase, it will be possible to use both the Columbia View and 19th Street Reservoirs. The 19th Street Reservoir supplies water to the 507 zone, which is also fed through PRVs from the 660 zone. If the 19th Street Reservoir is on-line, the demand from the Columbia View Reservoir is reduced significantly.

5. The sustained pumping from Lone Pine Well is dependent on adjusting management of the Intermediate Pump Station. It appears that it would be beneficial to add a smaller pump, with a capacity of approximately 250-350 gpm, to this station. A smaller pump would help to balance the movement of water from the 352 zone into the 632 zone. The city can determine the need for such a pump after implementing the previously described improvements.

Transmission Pipeline from 352 Zone to 310 Zone

The effectiveness of connecting the zones with either 12-inch or 16-inch pipelines was examined. In addition, it was found that movement within the 352 zone is restricted and that a new section of pipeline is needed within this zone. This pipeline improves the delivery of water from Lone Pine Well to the 16-inch pipeline. **Exhibit 4** summarizes the results for the two sizes of transmission lines, with and without the new 8-inch line. These results are peak hour demands at projected buildout conditions.

EXHIBIT 4
Results for Increasing Use of Lone Pine Well
Demand condition: peak hour demand, buildout, reservoir level 5 feet below overflow

Transmission Pipeline Size	New 8-inch Line Included?	Result: Flow from 352 Zone to 310 Zone
12-inch	No	830 gpm (1.2 mgd)
12-inch	Yes	1,000 gpm (1.5 mgd)
16-inch	No	1,200 gpm (1.7 mgd)
16-inch	Yes	1,400 gpm (2.0 mgd)

The transfer of water from the 352 zone to the 310 zone will occur at lower rates this summer because the demands in the 310 zone are lower. With both the 16-inch line and the 8-inch line in place, the model indicates that the transfer rate will be approximately 1,100 gpm (1.6 mgd) for projected 2005 demands.

It is recommended that a 16-inch diameter pipeline be used for the new transmission line together with the installation of the 8-inch line within the 352 zone. The combination of these two improvements enables the transfer of 2.1 mgd (1,460 gpm) from the 352 zone to the 310 zone. It is important to transfer at least this flow rate since there is the possibility of increasing the capacity of the Lone Pine Well or of adding a second well in this area. (These expansion possibilities are discussed in the water supply memo that has been prepared for the master plan.)

PRV Added to 660 Zone

To increase the use of the Lone Pine Well, it is also necessary to pump more water into the 632 zone and into the Columbia View Reservoir via the Intermediate Pump Station. Since demands are relatively low in the 632 zone, this can only be done if more water can be moved through this zone to feed areas to the west.

After discussions with city staff, it was determined that a possible means to accomplishing this was to install a PRV within the 660 zone and to take the 19th Street Reservoir off-line. This lowers the hydraulic grade line so that it is possible to feed a portion of the 660 zone from the Columbia View Reservoir (with an overflow elevation of 632 feet). The location that appears to work best for this PRV is near the intersection of Three Mile Road and Dry Hollow Road on the existing 12-inch pipeline.

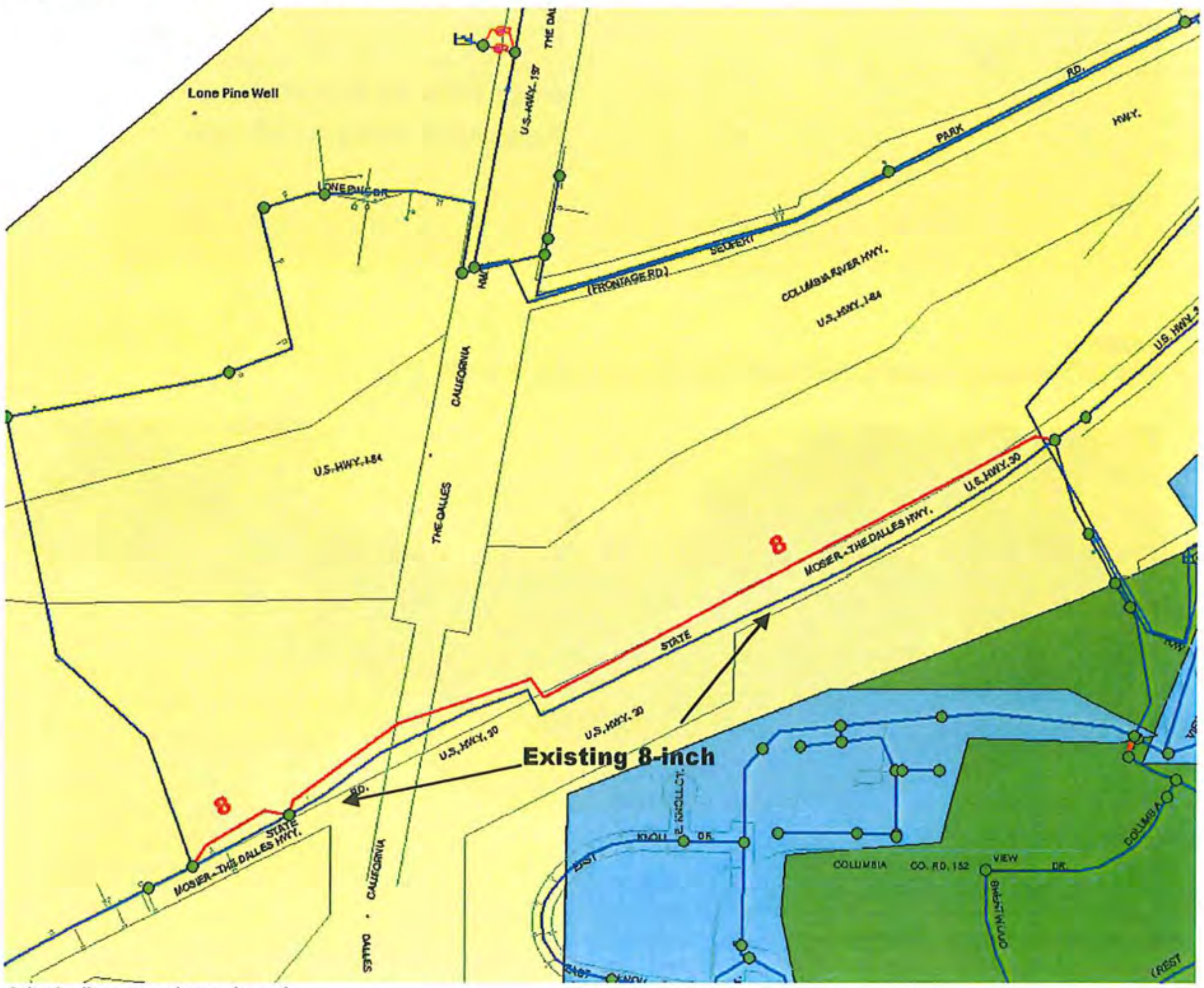
Exhibits 5-7 illustrate the resulting pressures in the system in this area when the PRV is closed (**Exhibit 5**), when the PRV is set at 40 psi (**Exhibit 6**) and when the PRV is set at 60 psi (**Exhibit 7**). When the valve is closed and the area east of this location is fed entirely from the Columbia View Reservoir, the result is low pressure in a number of areas. This is not an acceptable operating scenario. Opening the valve and setting the pressure to 40 psi improves the pressures, but still results in a number of low pressures in the system. A setting of 60 psi is recommended. This results in marginally low pressures to only three areas, but still allows Columbia View Reservoir to provide a significant level of service to this area.

For a setting of 60 psi and buildout, peak hour demands, the flow from Columbia View Reservoir into the eastern half of the 660 zone equals approximately 1,350 gpm.

EXHIBIT 1
Proposed 16-inch 352-310 Transmission Pipeline

Shown in red

EXHIBIT 2
Proposed 8-inch Pipeline in 352 Zone



8-inch diameter shown in red

EXHIBIT 3
Location of New PRV at Dry Hollow Road

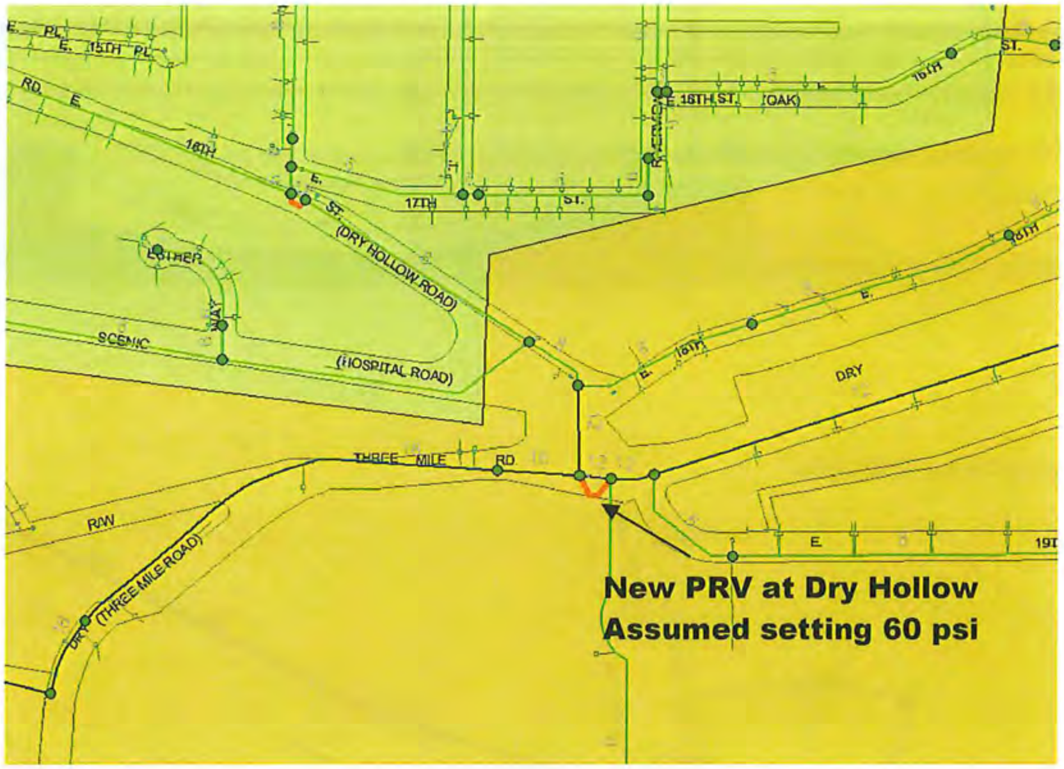


EXHIBIT 5
PRV at Dry Hollow is closed. All flow comes from Columbia View Reservoir.

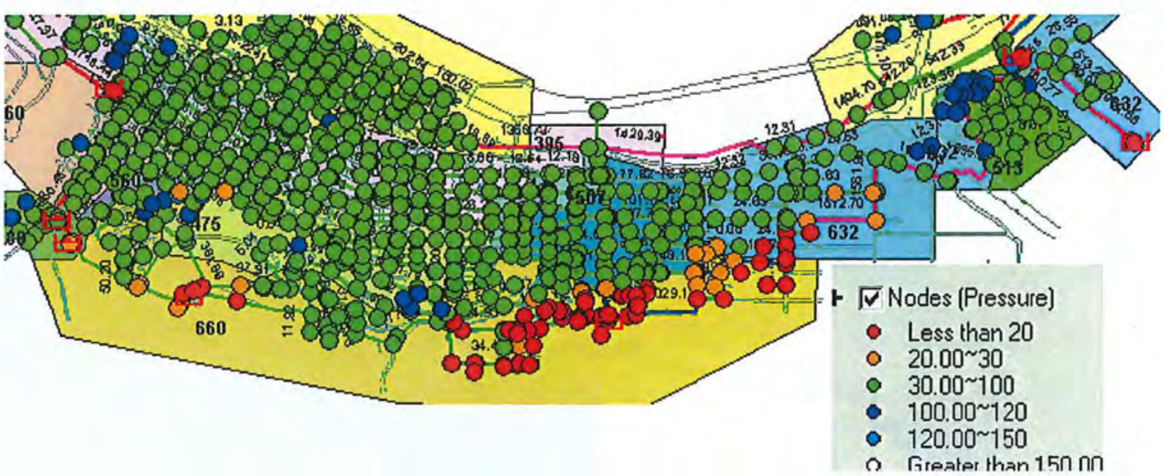


EXHIBIT 6

PRV at Dry Hollow is open. 40 psi pressure setting

Nodes shown in orange have pressures less than 30 psi and nodes shown in red have pressures less than 20 psi.

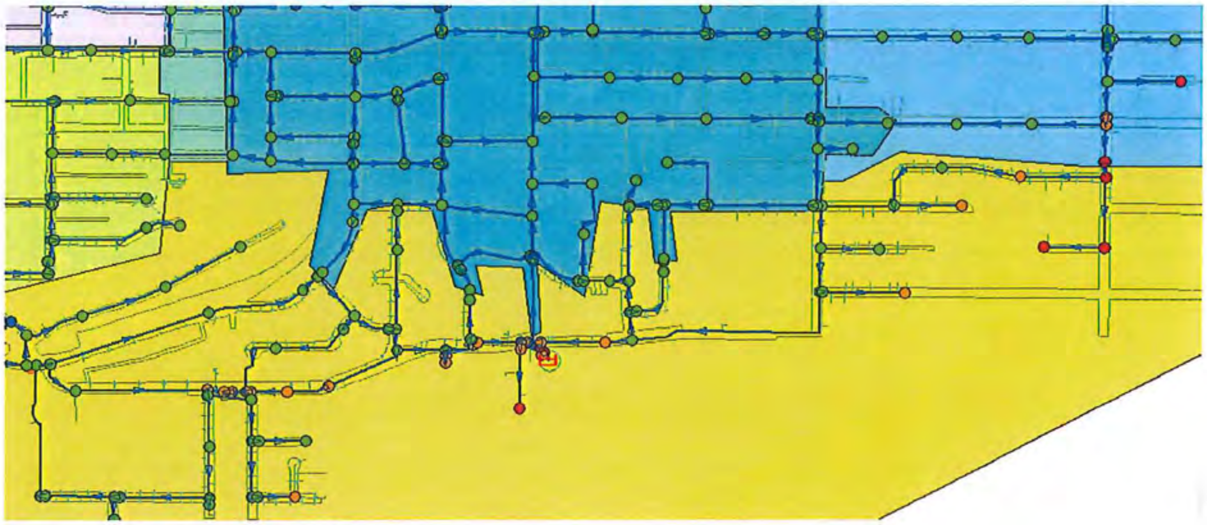


EXHIBIT 7

PRV at Dry Hollow is open. 60 psi pressure setting

Elevations above 500 feet will experience low pressures during PHD-BO demand conditions. Three locations shown.