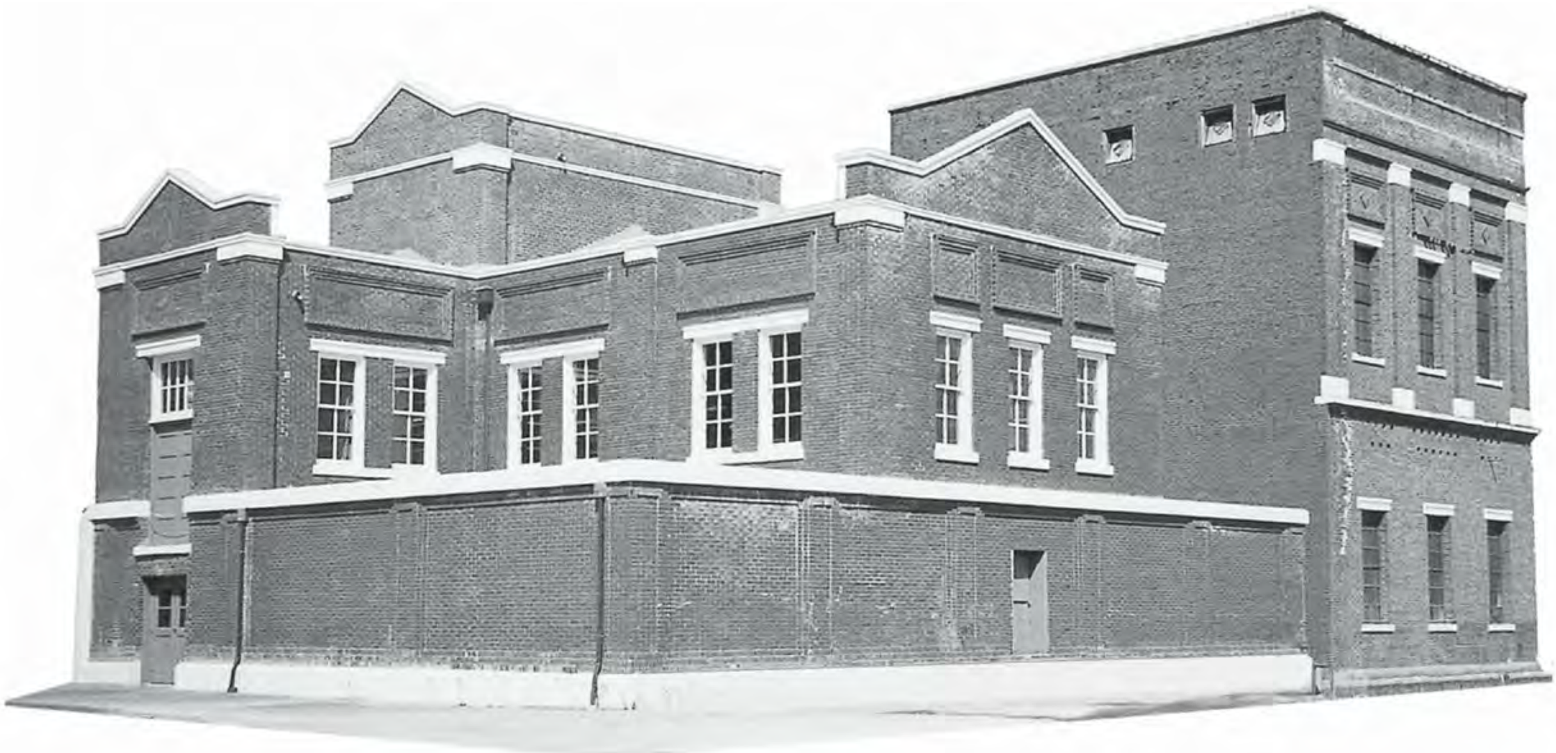

City of Albany

Water Facility Plan

December 2003

MWH

CITY OF ALBANY

Water Facility Plan

REGISTERED PROFESSIONAL
ENGINEER
58908PE
OREGON
JAN. 12, 1999
DAVID L. McPHERSON

Expires: 12/04

CITY OF Albany OREGON

MWH

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CHAPTER 1 – INTRODUCTION

This water facility plan summarizes work performed by the consulting firm of Montgomery Watson Harza (MWH) and the City of Albany to update and consolidate two water facility plan documents. Prior to development of this plan, the *Albany and Millersburg Water System Facility Plan*[1] and the *North Albany Water Facility Plan*[2] guided improvements to the water system. This plan evaluates the City's water system including, the Santiam-Albany Canal, the Vine Street Water Treatment Plant (WTP) and the distribution system including water lines, pump stations and storage facilities. The plan provides recommendations to correct existing deficiencies, meet future growth requirements, and ensure compliance with the federal Safe Drinking Water Act (SDWA) and other regulatory requirements. The plan has been prepared both to serve the needs of the City's water customers and to conform with state drinking water planning requirements (Oregon Administrative Rule (OAR) 333-061-0060 (5)) governed by the Oregon Department of Human Services Drinking Water Program (ODWP).

PURPOSE OF THE PLAN

The objectives of this plan are to evaluate the water system, project future demands, and establish a schedule of system improvements necessary to provide safe and reliable water service. As part of this report, a list of improvements has been developed with estimated costs and a recommended completion schedule.

This plan also serves as a resource for development of a water management plan; a separate, stand-alone document developed in compliance with OAR 690-86-120 through OAR 690-86-170. A water management plan will be submitted to the Water Resources Department (WRD) for review and approval following completion of this facility plan.

SCOPE OF WORK

The scope of work for this project was to consolidate and update the City of Albany's water facility plans into a single document that reflects existing and future needs. The scope included tasks to:

- Describe existing water facilities and service area characteristics,
- Describe historic population and water demands and develop projections of water demands with anticipated growth,
- Develop planning criteria to be used in evaluating the existing system and future system needs,
- Describe the existing and anticipated future federal and state drinking water regulatory requirements,
- Develop and calibrate a hydraulic model representing the City's water distribution system,
- Evaluate the existing system to determine required improvements,

[1] Albany and Millersburg Water Facility Plan, February 1988, Brown and Caldwell, Inc.
[2] North Albany Water Facility Plan, July 1988 (amended), Brown and Caldwell, Inc.

- Identify system improvements needed to support projected growth and development and to meet anticipated future water quality regulations,
- Assist the City in developing a water management plan meeting the requirements of the Oregon Water Resources Department (WRD), and
- Prepare a prioritized list of recommended improvement projects based on the evaluation of existing and future facilities.

This plan focuses on evaluation of existing facilities and improvements needed to meet future needs. The plan does not include recommendations for routine operating and maintenance activities such as water line flushing and cleaning, pump maintenance, and valve exercising programs. The water facility plan work products include:

- Water facility plan document,
- A hydraulic model of the water system.

The hydraulic model of the distribution system has been installed on the City's network and is available at individual workstations in the Engineering and Operations work groups.

CHAPTER 2 – STUDY AREA CHARACTERISTICS

HISTORY

The City of Albany, Oregon (the City) was founded in 1848 and incorporated in 1864. Albany is the 12th largest city in Oregon with a population of 40,852 (2000 census). The City is located approximately 24 miles south of Salem and lies adjacent to the Willamette River immediately east of its confluence with the Calapooia River.

Albany residents receive drinking water from the Vine Street Water Treatment Plant (WTP) located at Fourth and Vine. The WTP receives water from the Santiam-Albany Canal (Canal), discussed in *Chapter 7 – Canal Evaluation*. The Canal originates on the South Santiam River just east of the City of Lebanon. Construction of the Canal began in 1872 for the transportation of goods. In 1892 a hydroelectric facility was brought on line and in 1912 Albany's first water treatment plant was built. The abundance of power and water supply provided by the Canal created opportunities for manufacturing plants and mills to develop within the City. The surrounding farms also took advantage of the Canal for agricultural production and processing.

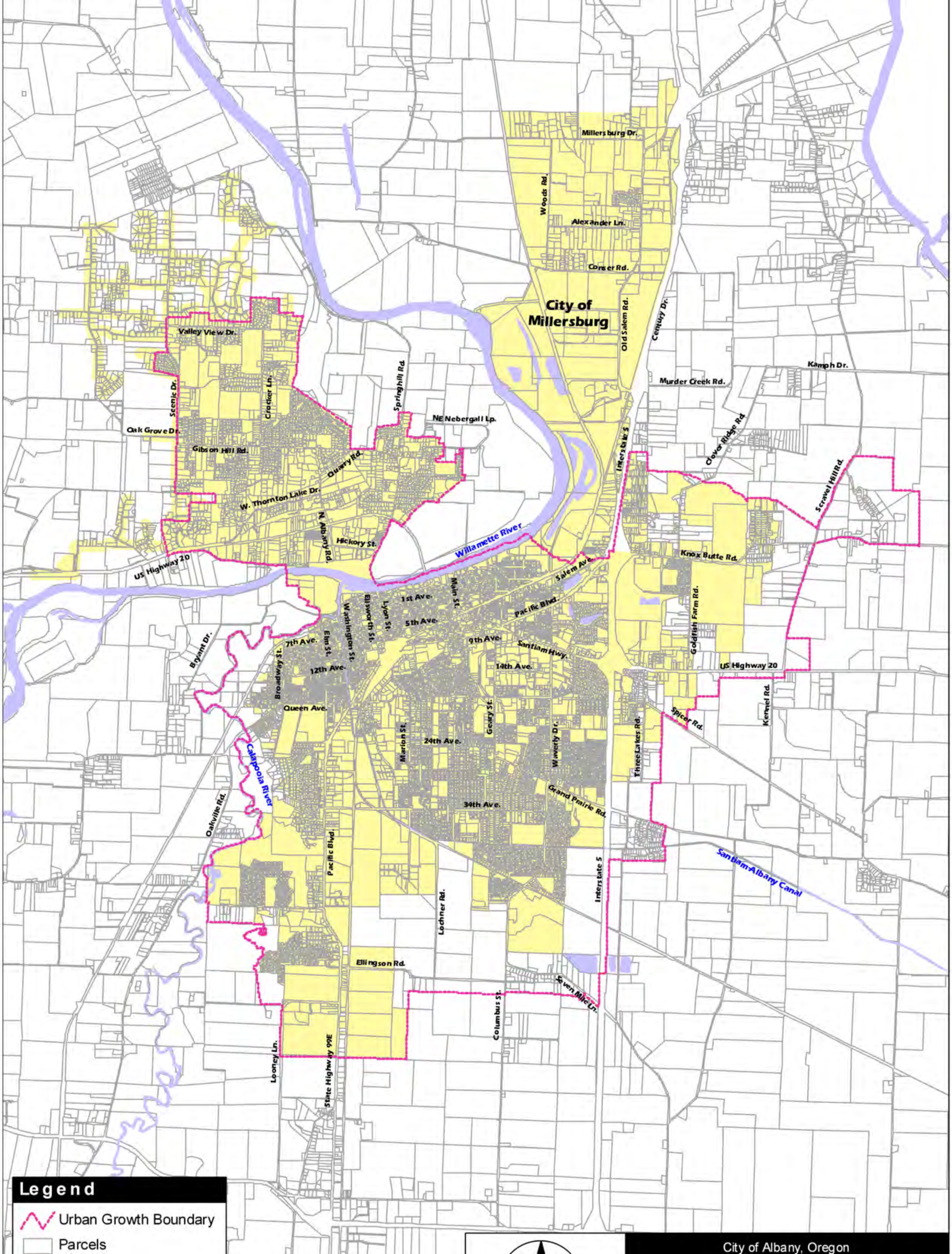
As the City's industrial and residential water demands increased, the water distribution system continued to grow. The City purchased the water system from Pacific Power and Light (PP&L) in 1984. The purchase included approximately 18 miles of canal, the Vine Street WTP, a hydroelectric facility, and a distribution system. At the time of purchase, PP&L provided water service for the City of Albany, the City of Millersburg, and the North Albany County Service District (NACSD). The City continues to provide water to these customers but took ownership of the NACSD water lines in July of 1991 through an agreement with Benton County.

SERVICE AREA

The City of Albany's water service area currently includes both the cities of Albany and Millersburg, limited service to areas outside Albany's city limits within the Urban Growth Boundary (UGB), and limited service to properties outside the UGB that were previously served by the North Albany County Service District (NACSD). Albany's water service area is shown in ***Figure 2-1***. The water system distributes drinking water to residential, commercial, industrial, and public customers as well as provides water for fire fighting.

PHYSICAL CHARACTERISTICS

Briefly described below are the physical characteristics defining Albany's service area including topography, geology and climate, as well as their implications for the development and use of the City's water system.


City of Millersburg
Millersburg Dr.
Woods Rd.
Alexander Ln.
Conser Rd.
Old Salem Rd.
Century Dr.
Kamph Dr.
Murder Creek Rd.
Clover Ridge Rd.
Scravel Hill Rd.
Interstate 5
Valley View Dr.
Scenic Dr.
Crocker Ln.
Springhill Rd.
NE Nebergall Lp.
Oak Grove Dr.
Gibson Hill Rd.
Quarry Rd.
W. Thornton Lake Dr.
N. Albany Rd.
Hickory St.
Willamette River
US Highway 20
Knox Butte Rd.
Salem Ave.
Main St.
1st Ave.
Lyon St.
Ellsworth St.
Washington St.
5th Ave.
Pacific Blvd.
Goldfish Farm Rd.
7th Ave.
Elm St.
9th Ave.
Santiam Hwy.
Bryant Dr.
Broadway St.
12th Ave.
14th Ave.
US Highway 20
Spicer Rd.
Kennel Rd.
Queen Ave.
Geary St.
Marion St.
24th Ave.
Waverly Dr.
Three Lakes Rd.
Calapooia River
Oakville Rd.
Grand Prairie Rd.
34th Ave.
Santiam-Albany Canal
Pacific Blvd.
Lochner Rd.
Interstate 5
Ellingson Rd.
Seven Mile Ln.
Columbus St.
Looney Ln.
State Highway 99E
Legend
Urban Growth Boundary
Parcels
City of Albany, Oregon

Topography and Geology

The City of Albany is located within the Willamette River Basin. The City lies in an alluvial plain between the foothills of the Coast Range and the Cascade Range. The largest portion of Albany's water service area is located south and east of the Willamette River, in an area where the alluvial plain has very minor topographic relief. In this area the change of elevation is only 100 feet over 12,500 acres, from an elevation of 175 feet to 275 feet. Although the majority of Albany's water service area is relatively flat, there are locations of distinct topographic relief caused by deposits of erosion-resistant sedimentary and volcanic intrusive rocks that form outcroppings such as Knox Butte, Spring Hill, and an elevated ridge in North Albany with a high point at the end of Hurleywood Drive. Elevations in North Albany reach over 500 feet and Knox Butte has an elevation of over 600 feet.

Topography influences the development of water systems because different pressure zones may need to be created in order to deliver water at desired pressure ranges. ***Figure 2-2*** shows Albany's topography and detailed discussion about the different pressure zones is included in *Chapter 3 - Existing System Description.*

Climate

Climate influences water use patterns and demand. The City of Albany's climatological region is the Mid-Willamette Valley, bounded by the Coast Range to the west and the Cascade Range to the east. This area has fairly uniform climate conditions characterized by warm dry summers and cool wet winters. The variances in temperatures in the Mid-Willamette Valley are moderate. Winter temperatures rarely fall below freezing (32° F), and summer temperatures on average exceed 90° F less than fourteen days a year. ***Figure 2-3*** shows the monthly maximum and minimum temperatures from January 1995 to December 2000.

The moderate variation in temperatures correlates to long growing seasons that have lead to agricultural development in the Albany area. The growing season is defined as the period of time between the last 32° F temperature in the spring and the first 32° F temperature in the fall. Between 1990 and 2000 the growing season has ranged from 147 to 237 days with an average growing season of 191 days[3]. Water demands associated with irrigation typically peak in July and August when temperatures are high and rainfall is at a minimum.

The majority of rainfall in the Mid-Willamette Valley occurs between the months of October and March. The average annual precipitation for the last five years (1995-2000) was 55 inches per year with approximately 80 percent, or an average of 44 inches, falling between October and March. However, this time period was characterized by several high rainfall years; a more representative average annual rainfall over a long period of record is 40-inches[3]. ***Figure 2-4*** shows the total monthly precipitation from January 1995 through December of 2000 with a maximum of 17.25 inches of rainfall in December of 1996.

Albany's climate typically results in a wide variation of water demands from winter to summer. The relatively warm dry summers result in higher water usage during the summer season, primarily from irrigation of landscaping, when compared to winter water usage. Seasonal variations in water demands are discussed in *Chapter 4 - Population and Water Demands.*

[3] http://www.ocs.orst.edu Zone 2 (Willamette Valley)

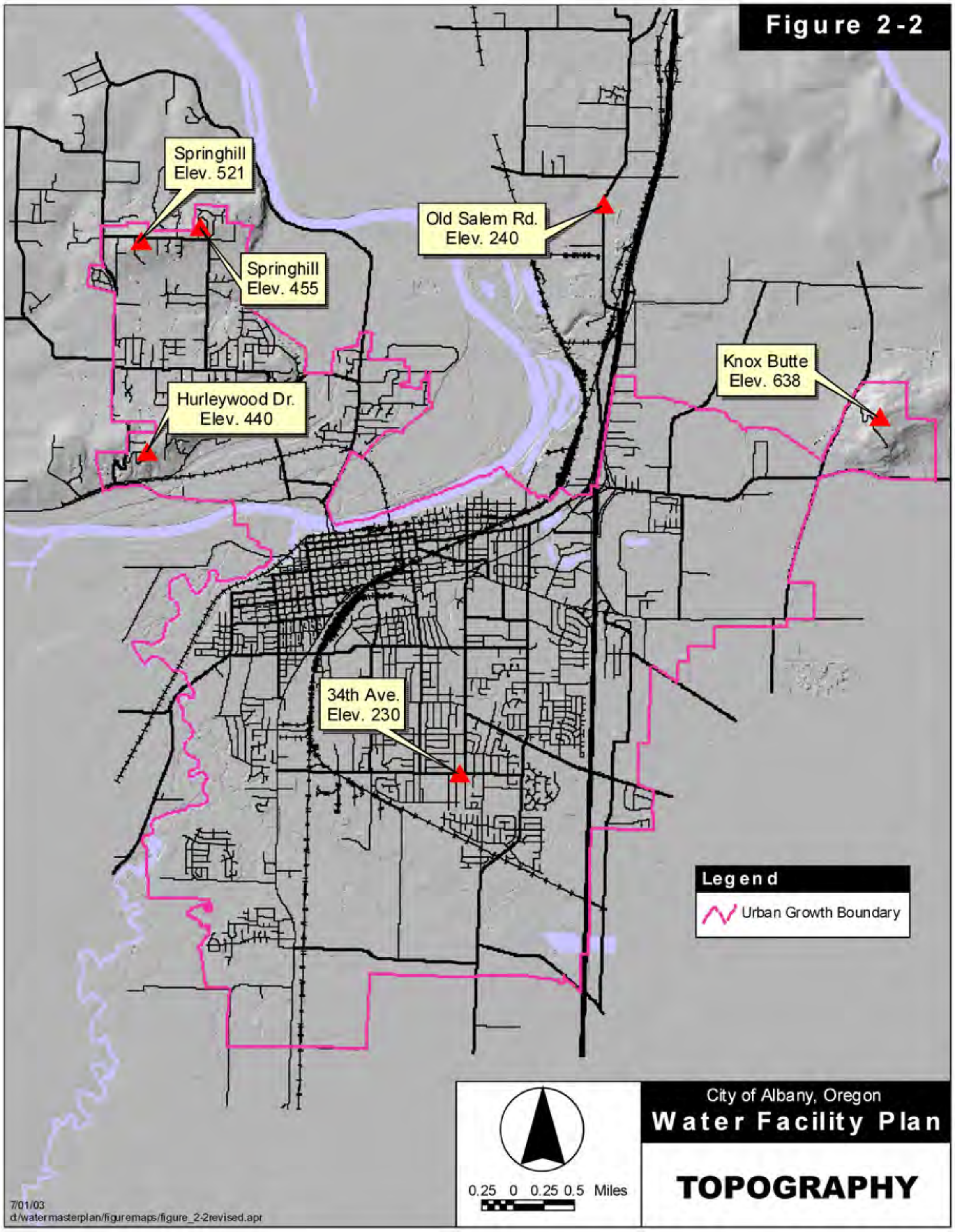


Figure 2-2
Springhill
Elev. 521
Springhill
Elev. 455
Old Salem Rd.
Elev. 240
Knox Butte
Elev. 638
Hurleywood Dr.
Elev. 440
34th Ave.
Elev. 230
Legend
Urban Growth Boundary
0.25 0 0.25 0.5 Miles
City of Albany, Oregon
Water Facility Plan
TOPOGRAPHY
7/01/03
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Figure 2-3:Maximum and Minimum Monthly Temperatures at the Vine St. WTP January 1995 through December 2000

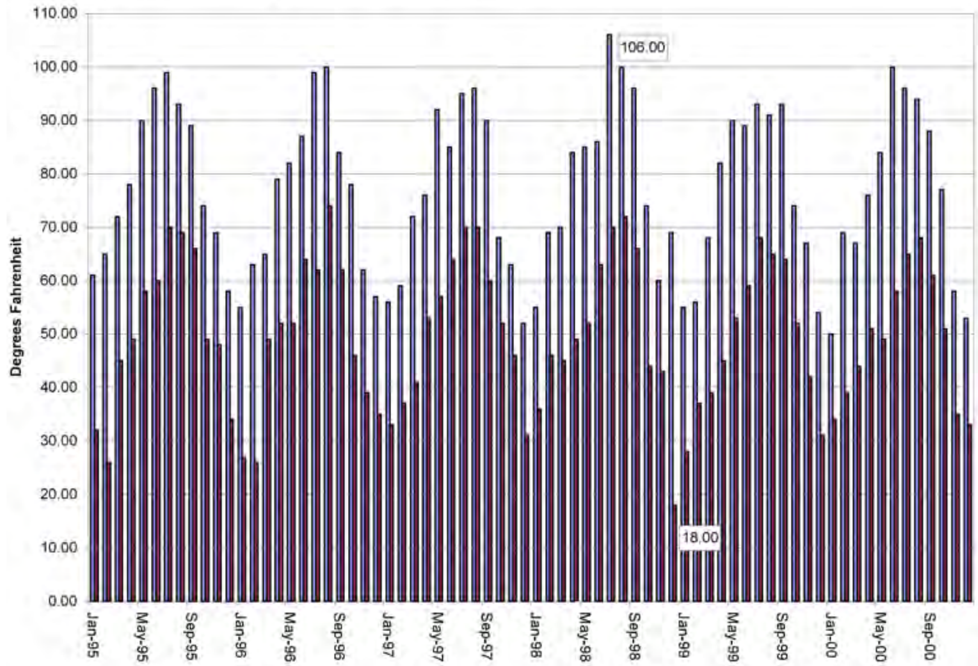
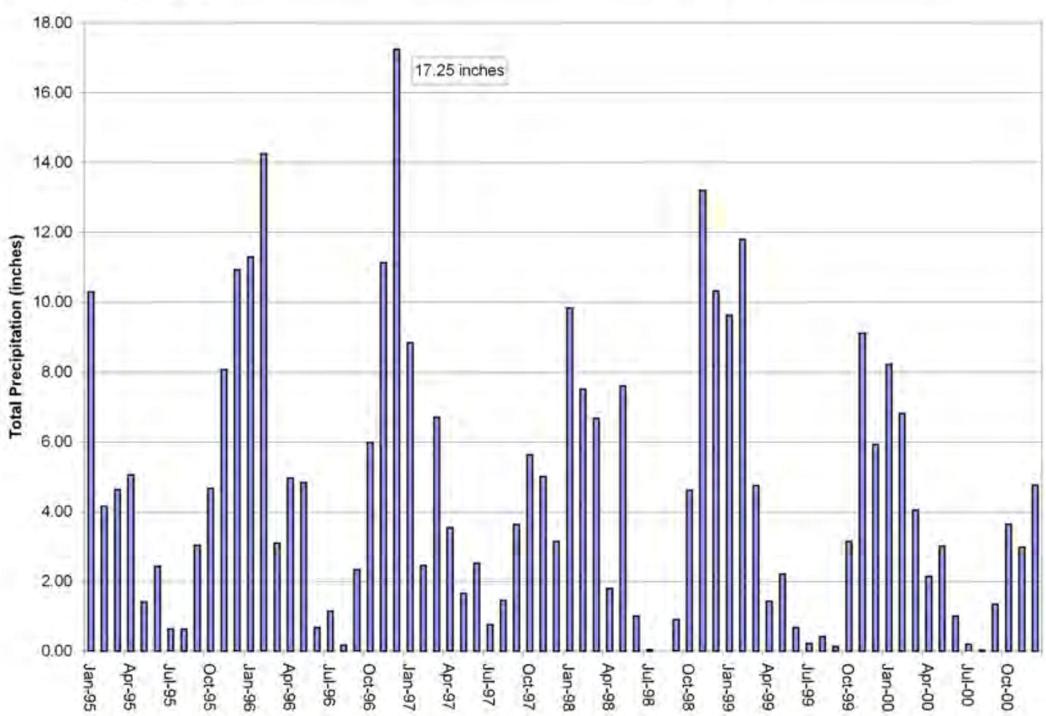


Figure 2-4: Total Monthly Precipitation at the Vine St. WTP January 1995 through December 2000



WATER SOURCES

Upon completion of the Joint Water Project (*Chapter 9 – Joint Water Project*) the City will have two sources of supply, the South Santiam River via the Santiam-Albany Canal and the Santiam River. The Joint Water Project involves construction of a raw water intake approximately one-quarter mile downstream from the confluence of the North and South Santiam Rivers. This intake will pump water to a new water treatment plant, the Scravel Hill WTP, that will supply water to Albany's and Millersburg's distribution systems.

The South Santiam River lies east of Albany and runs southeast to northwest toward its confluence with the North Santiam River. The North and South Santiam Rivers form the Santiam River that terminates at the Willamette River north of Albany. ***Figure 2-5*** shows the Santiam and Willamette Rivers as well as other waterways in the Albany area. ***Table 2-1*** includes maximum instantaneous and average stream flows for the South Santiam River at the USGS Waterloo Gage (14187500) from 1990 to 2000. ***Table 2-2*** includes maximum instantaneous and average stream flows for the Santiam River at the USGS Jefferson Gage (14189000) from 1990 to 2000.

Table 2-1: South Santiam River Flows

	Instantaneous Peak Flow (Date) / (cfs)		*Annual Average (cfs)*
1990	Apr. 28, 1990	18,200	3,018
1991	Nov. 25, 1990	11,700	2,718
1992	Dec. 06, 1991	12,400	1,872
1993	Apr. 04, 1993	13,500	2,772
1994	Jan. 06, 1994	10,200	2,483
1995	Jan. 14, 1995	16,200	3,534
1996	Feb. 07, 1996	29,200	4,605
1997	Nov. 19, 1996	22,000	3,317
1998	Jan. 24, 1998	12,700	3,271
1999	Dec. 29, 1998	19,400	3,994
2000	Nov. 26, 1999	18,600	2,552

Table 2-2: Santiam River Flows

	Instantaneous Peak Flow (Date) / (cfs)		*Annual Average (cfs)*
1990	Apr. 28, 1990	473,006	7,618
1991	Nov. 25, 1990	341,006	7,123
1992	Dec. 06, 1991	384,006	4,819
1993	Mar. 23, 1993	434,006	7,045
1994	Feb. 24, 1994	303,006	6,344
1995	Feb. 18, 1995	516,006	9,388
1996	Feb. 07, 1996	1,680,006	12,290
1997	Nov. 19, 1996	896,006	8,789
1998	Jan. 15, 1998	330,006	8,129
1999	Dec. 28, 1998	768,006	9,484
2000	Nov. 26, 1999	828,006	6,198

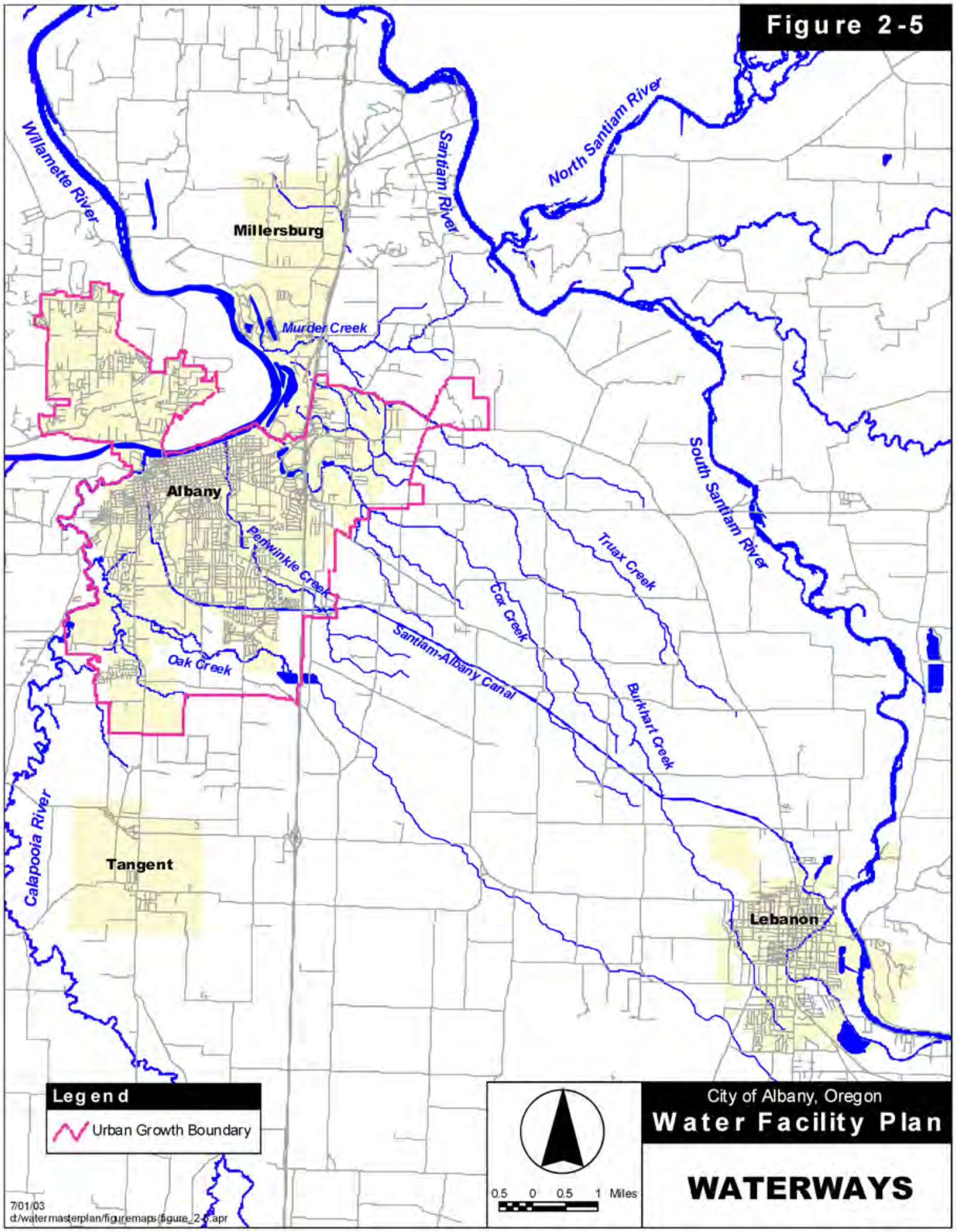


Figure 2-5
Willamette River
Santiam River
North Santiam River
South Santiam River
Millersburg
Murder Creek
Albany
Periwinkle Creek
Truax Creek
Cox Creek
Burkhart Creek
Santiam-Albany Canal
Oak Creek
Calapooia River
Tangent
Lebanon
Legend
Urban Growth Boundary
0.5 0 0.5 1 Miles
City of Albany, Oregon
Water Facility Plan
WATERWAYS
7/01/03
d:/watermasterplan/figuremaps/figure_2-5.apr

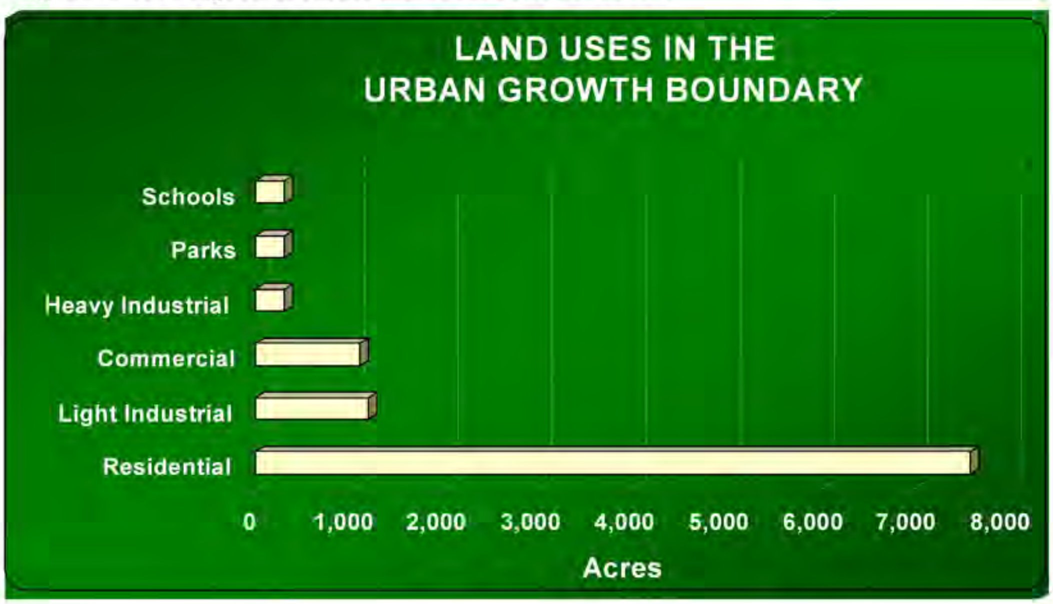
LAND USE

Albany's different land use zones are defined in the *Albany Development Code*[4] and the City's *Comprehensive Land Use Plan*[5]. The City's *Comprehensive Land Use Plan* has been used in *Chapter 4 - Population and Water Demands,* to help define buildout land use conditions and associated water demand projections. Property located in the urban fringe, the area between the city limits and the Urban Growth Boundary, is identified as Urban Residential Reserve in the *Comprehensive Land Use Plan*. In order to project future water demands in these areas, land uses as defined in the *Development Code* were assigned to these areas. This process is discussed in *Chapter 4 – Population and Water Demands*, and when compiled with the rest of the City's land uses resulted in the totals shown in ***Table 2-3*** and ***Figure 2-6.***

Table 2-3: Projected Land Use Totals at Buildout

Land Use	*Area (acre)*
Residential	7,600
Commercial	1,100
Heavy industrial	300
Light Industrial	1,200
Schools	300
Parks	300
Total Area	***10,800***

Figure 2-6: Projected Land Use Totals at Buildout



[4] City of Albany Planning Division, City of Albany Development Code, February 2003.
[5] City of Albany, *Comprehensive Plan,* January 1989.

CHAPTER 3 – EXISTING SYSTEM DESCRIPTION

INTRODUCTION

The City of Albany operates and maintains the water system that supplies water to the Cities of Albany and Millersburg, and limited areas outside Albany's city limits. This chapter provides descriptions of the existing water system, including water rights, water supply, and the distribution system.

The Cities of Albany and Millersburg are currently working on developing an additional water source as discussed in *Chapter 9- Joint Water Project*. In 2006, when this plant is brought on line, Millersburg's distribution system will be independent of Albany's. Therefore, facilities specific to the City of Millersburg are not included in this chapter.

WATER RIGHTS

The Oregon Water Resource Department (WRD) regulates water rights throughout Oregon. The first step in the water rights process is to obtain a water use permit. This requires the community to establish a need and demonstrate a beneficial use for the water requested. A water right can be perfected once the community demonstrates that the permitted water has been put to beneficial use. The priority date for the water right is the date of approval of the original permit. When comparing two water rights, the right with the earlier priority date is said to be "senior" and the right with the later priority date is said to be "junior". During a water shortage, water right holders with senior rights are allowed to access their full allocations prior to a junior water right holder.

Albany currently holds two municipal use water rights on the South Santiam River, totaling 50 cfs. One water right for twenty-one (21) cfs has been perfected with a priority date of 1878 and the remaining 29-cfs permit has a priority date of 1979 but has not been perfected. In addition to these municipal water rights, the City maintains a 275-cfs perfected water right from the South Santiam River for hydropower with a priority date of 1874.

As noted earlier, the City of Albany receives its water from the South Santiam River via the Santiam-Albany Canal. The state has issued approximately 382-cfs in water rights to Albany, Lebanon, and agricultural users along the Canal as shown in ***Table 3-1***.

WATER SUPPLY

A diversion dam located approximately 1 mile east of Lebanon on the South Santiam River diverts water to the Santiam-Albany Canal. The 18-mile Canal, discussed in detail in *Chapter 7 – Canal Evaluation*, conveys water for municipal uses, agricultural needs, and power generation. Currently, the South Santiam River via the Canal is the City's sole source of water supply.

Albany discontinued power production in 1991 because of permitting requirements from the Federal Energy Regulatory Commission (FERC). Since that time the City has obtained approval for their license and is working toward making the necessary improvements to resume power generation. A discussion of the implications of hydropower generation on required Canal capacity is included in *Chapter 7 - Canal Evaluation*.

Table 3-1: Summary of South Santiam Water Rights Along the Canal[6]

Name	*Quantity (cfs)*	*Priority Date*	*Use*	*Certificate Number*
City of Albany	275	1874	Hydroelectric Power	C-49387
City of Albany	29	1979	Municipal in Albany	P-44388
City of Albany	21	1878	Municipal in Albany	C-49386
City of Albany	5	1970	Recreation/Fish	P-34667
City of Albany	2	1968	Recreation/Fish	C-45179
Sub-Total	***332***			
City of Lebanon	18	1979	Municipal	P-44389
City of Lebanon*	10	1890	Municipal	TR-6110
Pacific Power & Light Co	9	1900	Municipal (Lebanon)	C-49385
Grand Prairie WCD	3.1	1966	Irrigate 249.5 acres	C-44767
Inez Gilbert Trust	2.45	1977	Irrigate 196 acres	P-41922
Inez Gilbert Trust	1.86	1974	Irrigate 148.5 acres	C-44642
Grand Prairie WCD	1.71	1967	Irrigate 137.1 acres	C-43277
Jacob Leichty	1.49	1965	Irrigate 119.3 acres	C-39223
Grand Prairie WCD	1	1969	Irrigate 137.4 acres	C-43283
IOOF & Masonic Cemeteries	0.48	1970	Irrigate 38.3 acres	C-43405
Grand Prairie WCD	0.24	1971	Irrigate 19.3 acres	C-43287
David Hickey	0.15	1975	Irrigate 11.8 acres	C-47834
Duane Eicher	0.015	1968	Agricultural	C-42012
Crown Zellerbach Crop	0.01	1972	Irrigate 1.1 acres	C-49938
Walter Jackson	0.01	1872	Irrigate 0.8 acres	C-49359
Albert French	0.005 0.038	1900 1900	Agricultural Irrigate 3 acres	C-49345
Daniel Stutzman	0.005	1873	Agricultural	C-49412
Glenn Huston	0.005	1873	Agricultural	C-49358
Jacob Leichty	0.005	1873	Agricultural	C-49365
John Kennel	0.005	1900	Agricultural	C-49362
Mildred Curths	0.005	1873	Agricultural	C-49336
Zella Burkhart	0.005	1873	Agricultural	C-49329
Zella Burkhart	0.005	1873	Agricultural	C-49330
Total Rights Along Canal =	***382***			

*** SHOWN AS CROWN ZELLERBACH CORP IN 1988 WFP. THE CITY OF LEBANON HAS SINCE ACQUIRED THIS RIGHT AND TRANSFERRED IT FROM AN INDUSTRIAL TO A MUNICIPAL USE.**

[6] Brown and Caldwell, *City of Albany Water Facility Plan*, 1988

Vine Street Water Treatment Plant (WTP)

The Vine Street WTP withdraws water from the Canal at 4th Avenue and Vine Street as shown in ***Figure 3-1***. The Vine Street Plant was constructed in 1912 and has gone through several upgrades, briefly described below.

- The original construction in 1912 included two settling basins and six filter beds,
- A new raw water pump station, flocculator, and clarifier were added to the treatment process in 1948,
- In the mid-1960's one of the existing sedimentation basins was converted into two filters (Filters 7 and 8),
- A solids contact basin and backwash ponds were added to the treatment process in the 1970's,
- In 1991, the treatment plant was expanded to increase the peak plant capacity from 15 MGD to a design capacity of 20 MGD and added features to ensure continued compliance with drinking water regulations. Some of the improvements included the addition of two raw water pumps, addition of tube settlers to Accelator #1, conversion of one of the settling tanks into a second clarifier with tube settlers (Accelator #2), the addition of filters 9 & 10, and the installation of a Hypalon baffle in the Maple Street Reservoir.
- In 1995 solids handling was improved through outlet modifications to the backwash/sludge holding lagoons and the addition of a second drying bed.

Historically, the maximum day flow produced by the plant is approximately 16 MGD. *Chapter 8 – Vine Street Water Treatment Plant*, discusses the Vine Street Water Treatment Plant evaluation and ***Table 3-2*** includes an itemized plant inventory.

DISTRIBUTION SYSTEM

Pressure Zones

Albany's distribution system is comprised of 3 pressure zones. Pressure zones are determined by elevation and are created to ensure desired pressures at a customer's service tap.

- Zone 1 is comprised of industrial, commercial and residential customers and is served by the High Service Pump Station at the Vine Street WTP, and the Queen, 34th, and Broadway Reservoirs. It ranges in elevation from 185 to 230 feet and has typical system pressures that range from approximately 40 to 85 psi.
- Zone 2 is almost entirely comprised of residential customers and is served by the Wildwood Reservoir and the North Albany Pump Station. It ranges in elevation from 230 to 350 feet and has typical system pressures from approximately 40 to 95 psi.


Figure 3-1
Control Building
3rd Ave.
Calapooia River
Intake
Accelator #2
Filters
Accelator #1
Maple St. Reservoir
Vine St.
Filters
4th Ave.
Chemical Storage Buildings
HSPS
Santiam-Albany Canal
Maple St.
5th Ave.
7/01/03
d:/watermasterplan/figuremaps/figure_3-1.apr
25 0 25 50 Feet
City of Albany, Oregon
Water Facility Plan
VINE STREET WTP SITE MAP

	1	Radial gate	Waterman Industries	8 feet	1991					
	1		EIM Controls							
			FCN 2-2	.25 HP, 1725 RPM						
	1	rotating	Custom	12 feet	1950					
				20 sq ft						
				13 mgd						
				20 mgd						
	1		Winsmith	1 HP	1998					
	1	sprayers	-	system pressure	1994					
					Added 1948		Pipeline inspection/repair		2005-2009	Install pi
	1	Vertical turbine	US Motors	50 hp, 1170 rpm	2003					
			PACO Model 18 FHXH	2,500 gpm @ 60 feet TDH	1972/rebuilt 2002					
	1	Vertical turbine	US Motors	50 hp, 1170 rpm	2003					
			PACO Model 18 FHXH	2,500 gpm @ 60 feet TDH	1972/rebuilt 2000					
	1	Vertical turbine	US Motors	50 hp, 1170 rpm	1991					
			Peerless Model 16HH	2,500 gpm @ 60 feet TDH	1991/Rebuilt 2000					
	1	Vertical turbine	US Motors	20 hp, 1200 rpm	1965					
			Layne and Bowler Pump Co.	2,000 gpm @ 32 feet TDH	1965					
			Model 18 FHXH							
	1	Vertical turbine	US Motors	50 hp, 1170 rpm	1972					
			Layne and Bowler Pump Co.	2,500 gpm @ 60 feet TDH	1972					
			Model 18 FHXH							
	1	Vertical turbine	US Motors	50 hp, 1170 rpm	1972					
			Layne and Bowler Pump Co.	2,500 gpm @ 60 feet TDH	1972					
			Model 18 FHXH							
	1	Vertical turbine	US Motors	30 hp, 1200 rpm	2001					
			Layne and Bowler Pump Co.	2,500 gpm @ 40 feet TDH	1965/Rebuilt 2001					
			Model 16KHH/ PACO							
			Peerless Model 16 KHH							
	1	Vertical turbine	US Motors	75 hp, 1775 rpm	1991					
			Robicon VSD	4,000 @ 60 feet TDH	1991					
			Peerless Model 16 HH		1991					
	1	Vertical turbine	US Motors	100 hp, 1775 rpm	rewound and rebuilt 2002					
			Robicon VSD	4,000 gpm @ 60 feet TDH	1991					
			PACO		2002					
			Model 14FHH							
				25,000 gpm						
				36.0 mgd						
				21,000 gpm						
				30.2 mgd						
	1	Submersible	Flygt Model CP3085-434	4" submersible	1991					
		Static in-line mixer		30" - 3 elements	1991					
							Pipeline inspection/repair		2005-2009	Install pi
	1	"Accelator"	Infilco	295,000 gallons	Concrete structure 1948					
			Model VC-4900	8 mgd	w/Accelator					
				58 feet						
				82,300 gallons						
				15 minutes						
				10 HP						
				212,700 gallons						
				38 minutes						
				1,800 sq ft						
				2.54 gpm/sq/ft						
				258 ft						
				23,250 gpd/ft						
				2,190 sq ft	Replaced 1998				2010-2014	Tube sett
	2		Infilco	4 inch	Replaced 1993					

				10 HP						
				221,330 gallons						
				40 minutes						
				sq ft						
				2.54 gpm/sq ft						
				ft						
				gpd/ft						
				2,336 sq ft	1991	Tubes are cracking			2005-2009	Inspect/r
						Need repair or replacement				
	4		Infilco	4 inch	1991					
						Valve replacement	Gallery piping needs re-coating		2005-2014	Recoat g
	6			10.5 ft x 20.33 ft each	1912	FTW piping undersized			2005-2009	Replace
	4			15.33 ft x 32 ft each	7-8, 1965					
					9-10, 1991					
	6			1,281 sq ft						
	4			1,962 sq ft						
				3,243 sq ft						
				23.4 mgd						
				20 mgd						
				16 mgd						
					1991	Reaching end of useful life			2010-2014	Filter me
				12						
				3						
				12						
				15					2010-2014	Replace
				BIF Hydrocone					2010-2014	Replace
					1965/1991	Reaching end of useful life			2010-2014	Filter me
				12						
				3						
				12						
				15					2010-2014	Replace
				BIF Hydrocone/Dual Lateral - 7 & 8					2010-2014	Replace
				Leopold - 9 & 10						
					1912	Lacks by-pass	Suspected Cracked/leaking		2010-2014	Repair cr
		CIP Concrete		292,000 gallons at 14 feet depth						
				50.5 feet x 62 feet						
				26 minutes						
								No Auto. Capability	2005-2009	Incorpora
						Unable to provide redundency for larger filters				
	1	Vertical Turbine	US Motors	75 hp, 1800 rpm	1960				2005-2009	Replace
			Layne and Bowler Pump Co.	3,000 gpm @ 80 feet TDH						
	1	Vertical Turbine	Allis Chalmers	75 hp, 1180 rpm	1965					
			Allis Chalmers Model 14MF	7,500 gpm @ 32 feet TDH						
				16 gpm/sq ft						
				16 gpm/sq ft						
	1	rotating spray	Leopold		~1987		Corrosion on pipe		2005-2009	Replace
	1	rotating spray	Leopold		~1991				2010-2014	Replace
	1		Pneumatech	100 SCFM, 150 PSIG						
			Model AD-100	Compressor 3/4 HP						
									2005-2009	Incorpora
	1	Scatter Meter	Hach		~1991					
									2005-2009	Incorpora
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					
	1	On-line Turbidimeter	Hach		1991					

	1	On-line Cl2 analyzer	Hach		1993					
	1	On-line Cl2 analyzer	Hach		1991					
	1	On-line Cl2 analyzer	Hach							
	1	Sonic	Parametrics	Strap on 30 inch pipe	2002					
	1	Sonic	Parametrics	Strap on, 20 inch pipe	2002					
	1	Sonic	Parametrics	Strap on, 24 inch pipe	2002					
	-	none	-		1996					
	1	constant on-line	Hach		1996					
	1	constant on-line	Hach		1996					
		Earth with lining				Liner needs replacement	In need of reshaping		2010-2014	Reshape/
	2									
				540,430 gallons	1974					
				662,430 gallons	1974					
				1,142,860 gallons						
				1,740 cu yds						
				2,940 cu yds						
				4,680 cu yds						
	2		FMC	10 inch	1995					
	2	Electric	Limitorque L120 Series		1995					
	2									
				10,700 sq ft	1974					
				16,450 sq ft	1995					
				27,150 sq ft						
				1,320 cu yds						
				2,030 cu yds						
				3,350 cu yds						
					1957		Pipeline inspection/repair		2005-2009	Install pi
	1	vertical turbine	US Motor	100 hp, 1700 rpm	1957					
				4,000 gpm @ 75 feet TDH	1957/ 2002					
	1	vertical turbine	US Motor	125 hp, 900 rpm	1965					
				6,400 gpm @ 65 feet TDH	1965/ 2000					
	1	vertical turbine	US Motor	200 HP, 1180 rpm	Rebuilt 2000 by Paco					
			Robicon VSD		1991					
			PACO, Cat U81-70634	6,500 gpm @ ?? Feet TDH	2001					
	1	vertical turbine	US Motor	150 HP, 1175 rpm	1991					
			Robicon VSD		1991					
			Peerless Model 24HXB	5,000 gpm @ 83 feet TDH	1991					
				31.5 mgd						
				15,417 gpm						
				22.2 mgd						
					1963					
									2005-2009	Replace
	3									Sodium
	3				Remodeled 1990					
				5,000 gallons	1963	Lacks containment wall	No seismic restraint		2010-2014	Install co
				400 sq ft						
				200 sq ft						
	1	v-notch	Wallace and Tiernan	500 lbs/day						
	1		Wallace and Tiernan	500 lbs/day						
	1		Wallace and Tiernan	500 lbs/day						
	1	Volumetric	Wallace and Tiernan	50 lbs/hr						
xer				50 gallon tank						

			Serial # AT-19930							
	1	Steel	Magnetek mixer, 1/4 HP	14 lbs/hr	1991					
	2		Not used since 1990		1963					
	2				1963					
	1	Hopper, feed screw, tank and mixer	Wallace and Tiernan Model AT 19935	35 lbs/hr	1991					
	1	Steel tank	Cleveland Mixer Model CGB	145 gallons	1963					
	1	Steel tank		145 gallons						
	2	Volumetric Pump	Wallace and Tiernan Model A747 123	~25 gph						
	1	Plastic tank	Lightnin mixer, 1/8 hp, 1725 rpm	150 gallons	1991					
	1	Plastic tank		150 gallons						
	2	Rotary Gear	Teel Pump Model 1P771A, 1/4" ports	1 gpm	1991					
	1	Tank	Steel	1,400 cu ft	1991					
	2		Wallace and Tiernan	150 lbs/hr	1991					
			#1- Model AT19932		1991					
			#2- Model AT19933		1991					
	2		Magnetek mixer, 1/4 HP							
							Pipeline inspection/repair		2005-2009	Install pi
					1960					
	1	split case horizontal	Allis Chalmers PACO Model 8015-3	150 HP, 1770 rpm 3,250 gpm @ 145 TDH	1960 2002					
	1	split case horizontal	US Motor Peerless Model 10AE16	300HP, 1770 rpm - soft start 6,700 gpm @ 145 feet TDH	2002 2002					
	1	split case horizontal	Allis Chalmers Allis Chalmers	100 HP, 1770 rpm 2,000 gpm @ 145 feet TDH	1960 1986					
	1	split case horizontal	Allis Chalmers Allis Chalmers	200 HP, 1760 rpm 4,000 gpm @ 145 feet TDH	1960 1960					
	1	split case horizontal	US Motor/ Peerless Peerless	300HP, 1770 rpm - soft start 6,700 gpm @ 145 feet TDH	1991/rewound 2002 1991 (New impeller 2002)					
				32.6 mgd						
				23.0 mgd						
				15,950 gpm						
							Pipeline inspection/repair		2005-2009	Install pi
	1	steel, above ground		2 M.G.	1960					
		Hypalon		93 feet diameter, 40 feet high	1991, repaired 2000	In need of replacement			2005-2009 & 2015-2024	Replace l
	1			24 inch piping		Short-circuiting			2005-2009	Install pi
	2			24 inch diameter						

- Zone 3 is the smallest pressure zone and is comprised almost entirely of residential customers. It is served by the Valley View Reservoirs and the Gibson Hill Road Pump Station. Zone 3 has elevations from 350 to 510 feet and typical system pressures from approximately 20 to 95 psi. Some Zone 3 piping serves customers outside the Urban Growth Boundary at pressure Zone 2 elevations. In these locations, system pressures can be in excess of 120 psi (See ***Figure 10-3***).

Pressure zone boundaries, pump stations, water treatment plant, and reservoir sites are shown in ***Figure 3-2*** and ***Figure 3-3*** shows a schematic profile of how each pressure zone is served.

Currently, a small low-pressure area exists within Zone 3. This area is centered around the Valley View Reservoirs on Valley View Drive. The topography and operating levels of the reservoirs result in low water pressure in this area. This low-pressure area is described further in the operational description of the Valley View Reservoirs later in this chapter.

Pressure Reducing Valves (PRVs)

PRVs are used in locations where normal system operations create higher than desired pressures at a customer's service tap. PRVs can be installed in-line on a distribution main serving several customers or individually on a customer's service line. Currently, the City has two main line PRVs, one located on Edgewood Drive and one located on Rondo Street, and multiple individual customer PRVs in North Albany. The plumbing code requires a PRV on any water service with a service pressure over 80 psi in order to protect a customer's plumbing fixtures.

Pipe Size, Material, and Quantity

Including pipes located in the City the Millersburg, the City of Albany maintains a pipeline network comprised of more than 228 miles of pipe. Of the 228 miles of pipe, 217 miles are from 4 inches to 30 inches in diameter and 11 miles are pipelines that are smaller than 4 inches in diameter. Approximately 8 of the 11 miles of pipes less than 4 inches in diameter are considered steel pipelines. Pipe materials include asbestos cement (AC), cast iron (CIP), wrought iron (WI), ductile iron (DI), galvanized (GI), polyvinyl chloride (PVC), outside diameter dipped and wrapped steel (ODDW), and steel (STL). ***Table 3-3*** shows the approximate total footage of existing pipelines by diameter and material type of all pipe 4 inches in diameter and greater, as shown in the City's GIS database in January 2003. ***Figure 3-4*** shows the length of pipeline by material as a percentage of total pipeline length.

Of the 228 miles of pipe there are approximately 28 miles of finished water transmission pipeline within Albany's water system. Transmission pipelines in this plan have been classified as pipelines greater than or equal to 16 inch diameter. Typically, transmission pipelines are designed to convey large volumes of water from one point to another without numerous service connections. However, most water utilities serving distribution systems, including Albany, do not have fully dedicated finished water transmission lines and integrate the transmission pipelines into the distribution system. Transmission pipelines account for approximately 12 percent of all water pipelines in Albany's distribution system. Of the 28 miles of transmission pipeline, only 7.6 miles (27%) are greater than or equal to 24 inches in diameter.

Pipe Age and Leak History

Pipe ages in Albany's water system range from over 100 years old to new. A summary of pipe ages is shown in ***Table 3-4*** and ***Figure 3-5*** as shown in the City's GIS database in January 2003. ***Table 3-4*** and ***Figure 3-5*** only includes pipes with a diameter 4 inches or larger.

Pipes that have exceeded their service life often lead to excessive leaking and pipe failures. Albany's steel water lines are a good example of this. The City has approximately 29 miles of steel lines in their system; steel pipe being classified as wrought iron (WI), galvanized iron (GI), steel (STL), outside diameter dipped and wrapped steel (ODDW), and unknown pipe types (UNK). 8 of the 29 miles of steel water lines are less than 4 inches in diameter and therefore are not shown in ***Table 3-3*** or ***Table 3-4*.**

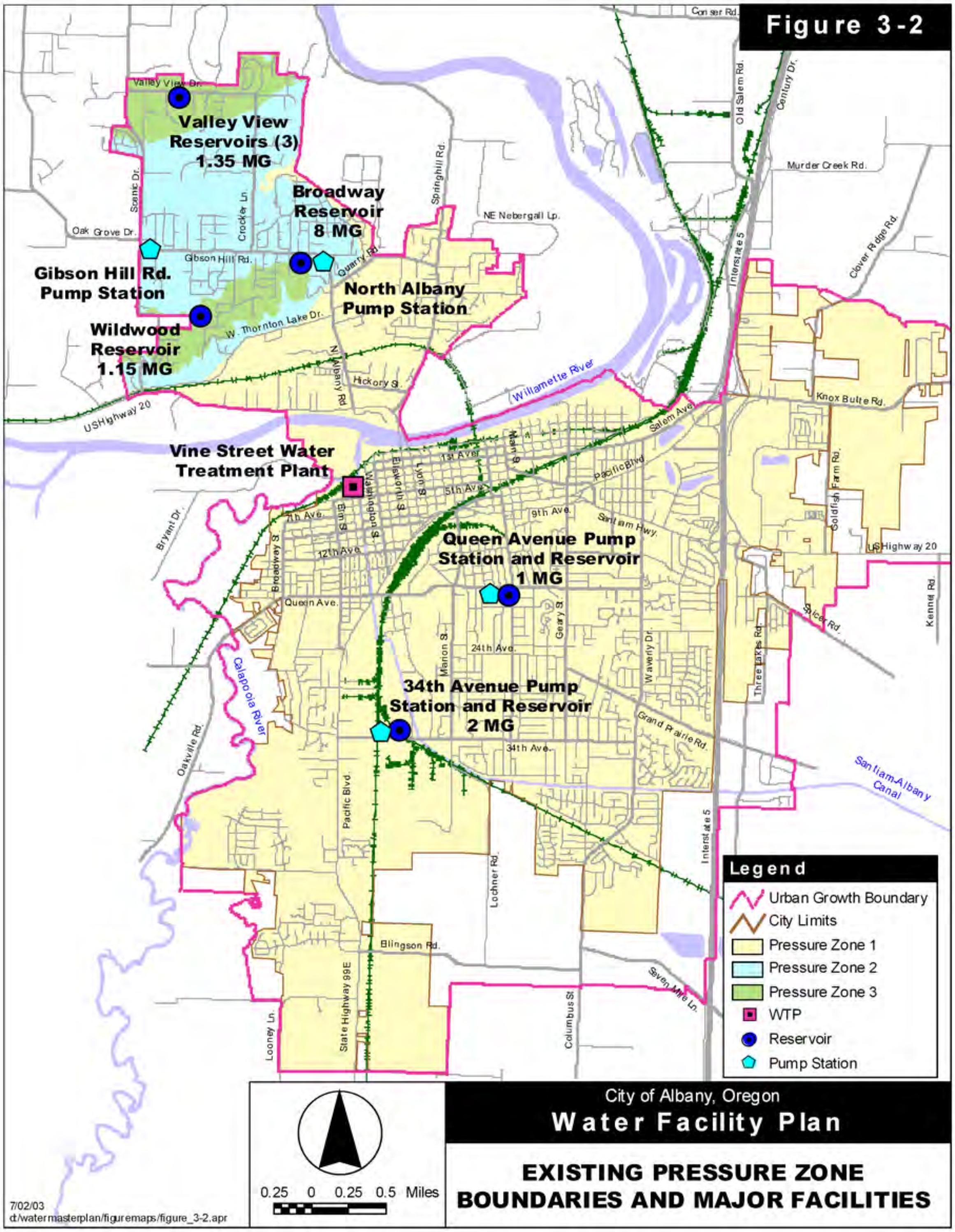
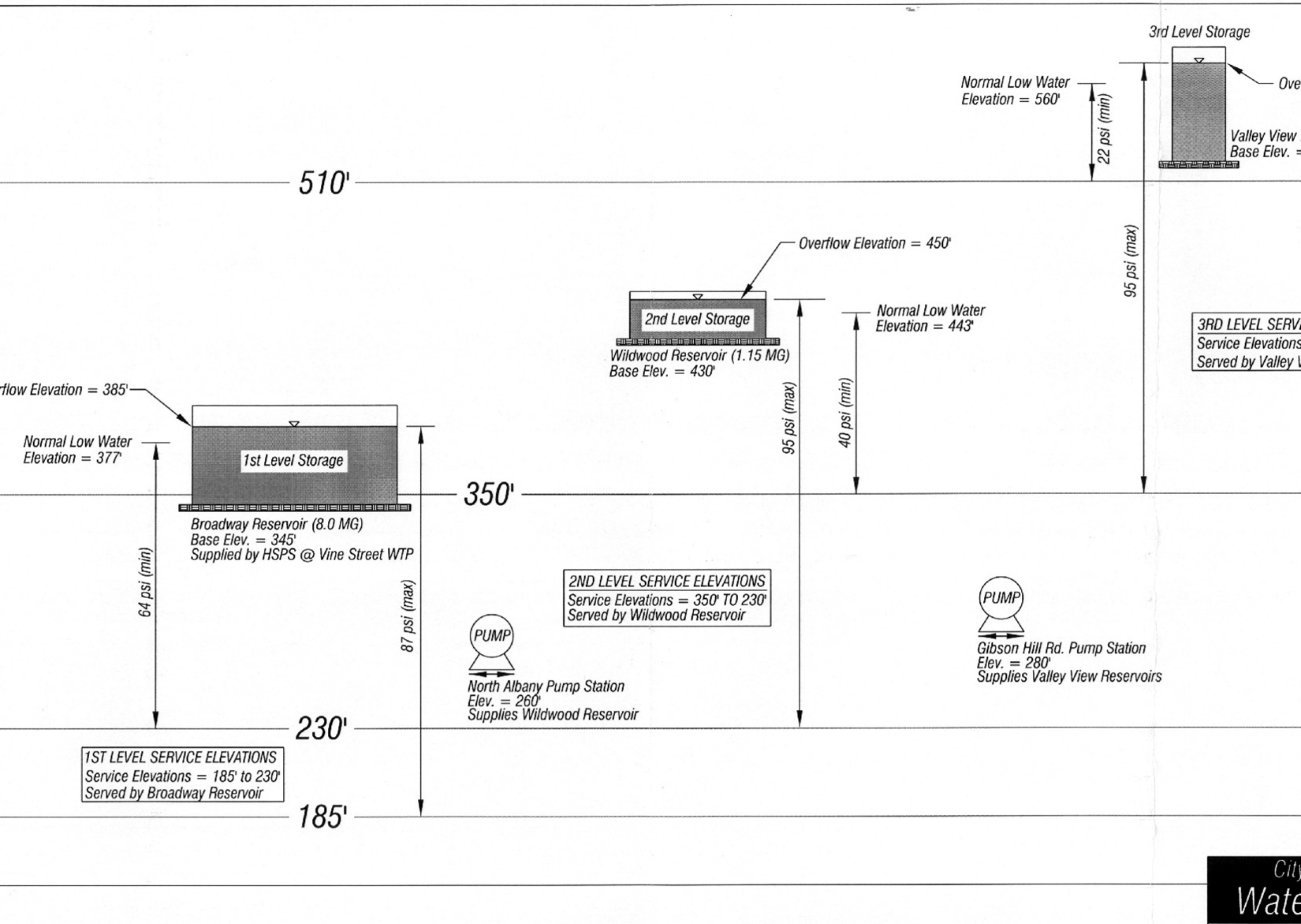


Figure 3-2
Valley View Reservoirs (3) 1.35 MG
Broadway Reservoir 8 MG
Gibson Hill Rd. Pump Station
North Albany Pump Station
Wildwood Reservoir 1.15 MG
Vine Street Water Treatment Plant
Queen Avenue Pump Station and Reservoir 1 MG
34th Avenue Pump Station and Reservoir 2 MG
Willamette River
Calapooia River
Santiam-Albany Canal
Legend
Urban Growth Boundary
City Limits
Pressure Zone 1
Pressure Zone 2
Pressure Zone 3
WTP
Reservoir
Pump Station
0.25 0 0.25 0.5 Miles
City of Albany, Oregon
Water Facility Plan
EXISTING PRESSURE ZONE BOUNDARIES AND MAJOR FACILITIES
7/02/03
d:/watermasterplan/figuremaps/figure_3-2.apr



NOTE: QUEEN AND 34TH AVENUE RESERVOIRS ARE AT GRADE PUMPED
STORAGE RESERVOIRS THAT SERVE PRESSURE ZONE 1 AND ARE

Table 3-3: Existing Distribution and Transmission Piping 4 inches in Diameter and Greater

	Pipe Material									*Total*	
Diameter (in.)	***AC (Feet)***	***CIP (Feet)***	***DI (Feet)***	***GI (Feet)***	***ODDW (Feet)***	***PVC (Feet)***	***STL (Feet)***	***WI (Feet)***	***UNK (Feet)***	***Feet***	***Miles***
4	31,914	37,645	6,819	758	14,132	7,226	19,616	9,900	4	128,014	24.25
6	210,845	26,817	29,730	21	14,735	4,450	20,919	8	2,316	309,841	58.68
8	95,747	7,032	168,017	741	5,680	2,086	7,033	0	1,138	287,474	54.45
10	27,928	12,657	2,825	396	4,241	0	3,649	0	0	51,696	9.79
12	93,741	14,511	113,191	0	243	0	463	0	0	222,149	42.07
14	35	0	0	0	0	0	0	0	0	35	0.01
Sub-Total Dist. (Feet)	*460,210*	*98,662*	*320,582*	*1,916*	*39,031*	*13,762*	*51,680*	*9,908*	*3,458*	*999,209*	-
(Miles)	*87.16*	*18.69*	*60.72*	*0.36*	*7.39*	*2.61*	*9.79*	*1.88*	*0.65*	-	*189.24*
16	38,430	273	24,100	0	0	0	83	0	0	62,886	11.91
18	0	257	1,043	0	0	0	0	0	0	1,300	0.25
20	9,301	0	34,812	0	0	0	45	0	0	44,158	8.36
24	0	0	37,903	0	0	0	1,362	0	0	39,265	7.44
30	0	0	1,086	0	0	0	0	0	0	1,086	0.21
Sub-Total Trans. (Feet)	*47,731*	*530*	*98,944*	*0*	*0*	*0*	*1,490*	*0*	*0*	*148,695*	-
(Miles)	*9.04*	*0.10*	*18.74*	*0.00*	*0.00*	*0.00*	*0.28*	*0.00*	*0.00*	-	*28.16*
Total (Feet)	***507,941***	***99,192***	***419,526***	***1,916***	***39,031***	***13,762***	***53,170***	***9,908***	***3,458***	***1,147,904***	**-**
(Miles)	***96.20***	***18.79***	***79.46***	***0.36***	***7.39***	***2.61***	***10.07***	***1.88***	***0.65***	**-**	***217.41***

Table 3-4: Water Line Length by Decade for Piping 4 inches in Diameter and Greater

	1890	***1900***	***1910***	***1920***	***1930***	***1940***	***1950***	***1960***	***1970***	***1980***	***1990***	***2000***	***Unknown***	***Total***
Feet	833	3,631	8,965	3,317	5,389	52,804	92,577	252,833	238,670	143,854	301,955	39,406	3,670	***1,147,904***
Miles	0.16	0.69	1.70	0.63	1.02	10.00	17.53	47.89	45.20	27.25	57.19	7.46	0.70	***217.41***

Figure 3-4: Percentage of Total Length by Material Type

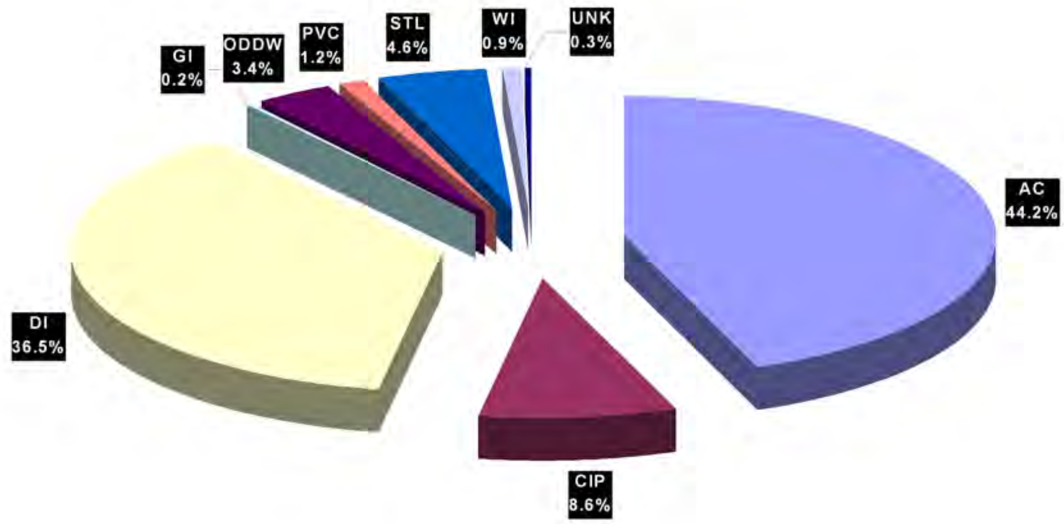
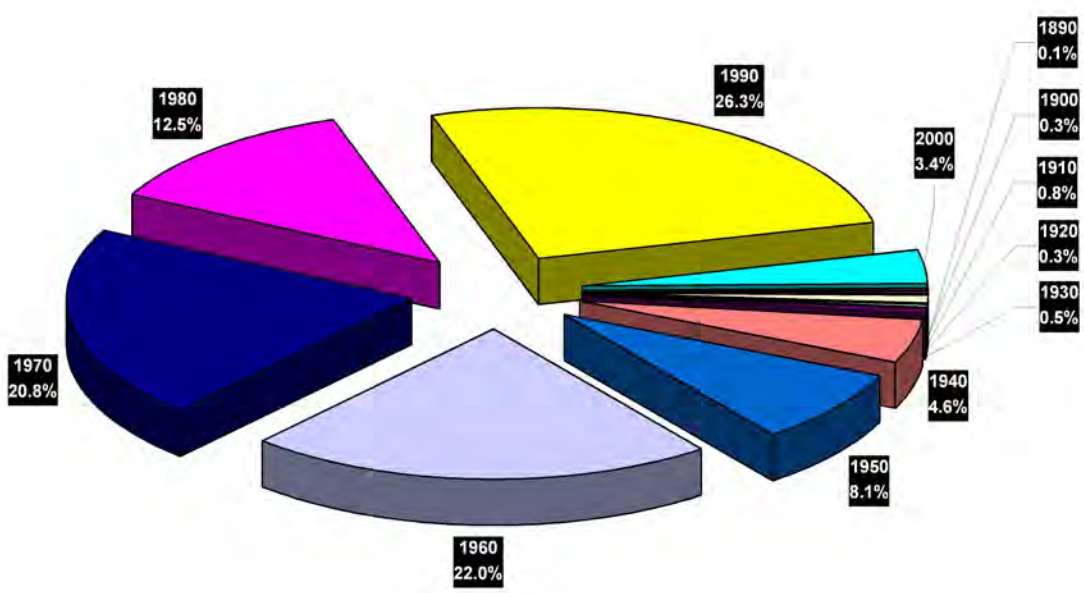


Figure 3-5: Percentage of Total Length by Decade



Steel pipes began being installed in the early 1900's and have exceeded their service life, deteriorating as shown in ***Photograph 3-1***. Operations and maintenance records show that the majority of water line leaks observed in Albany's system occur in steel water lines. Steel line replacement is discussed in *Chapter 10 – Distribution System Evaluation* and ***Figure 10-7*** depicts locations of steel water lines. The replacement program will improve system reliability and minimize customer service interruption.

Photograph 3-1: Deteriorating Steel Water Line

Water Meters

The City maintains approximately 14,400 water meters throughout the Albany and Millersburg water system, ranging in size from ¾ inch to 10 inch. A summary of these different meter sizes is shown in ***Table 3-5***. Small meters are meters less than 3 inches and large meters are meters 3 inch or larger. Meter sizes listed, except for the 5/8" x 3/4", are the standard sizes allowed in Albany's water system. Over time the 5/8" X 3/4" meters will be replaced with 3/4" X 3/4".

Table 3-5: Water Meter Sizes

	Meter Size	*Number Active*[1]
Small Meters		
	5/8" X 3/4" and 3/4" X 3/4"	13,052
	1"	821
	1 1/2"	210
	2"	237
	Sub-Total =	*14,320*
Large Meters		
	3"	40
	4"	23
	6"	11
	8"	3
	10"	1
	Sub-Total =	*78*
	Total =	***14,398***

1. Results of Springbrook and Hansen query, August 2003.

Water meters of several different types and manufacturers exist in Albany's system, complicating meter maintenance. In addition, meters installed in parts of North Albany prior to 2000 provide measurements in gallons instead of hundreds of cubic feet, the standard unit of measure for all other meters. In most instances, the City is now requiring the following meter types, all measuring in hundreds of cubic feet, be installed for all new water meter installations.

- 1" diameter or less; Sensus SR II or equivalent
- 1 ½" diameter; Sensus SR
- 2" diameter; Sensus Compound meters
- 3" diameter or greater; Sensus Compound or Fireline

Requiring installation of a standard meter type will make it easier and more cost effective to maintain meters throughout the service area. Standardizing meter types will also make it easier for upgrades to an automated reading system in the future.

Telemetry Locations and Monitoring Points

Currently, the City has telemetry and monitoring points located at Queen Avenue Reservoir and Pump Station, 34th Avenue Reservoir and Pump Station, Broadway Reservoir, North Albany Pump Station, Wildwood Reservoir, Gibson Hill Road Pump Station, Valley View Reservoirs, and the Millersburg meter vault. These locations are used to monitor and control system operations. Telemetry specific to each facility is discussed briefly with each pump station and reservoir description included in this chapter. The City does not have any monitoring points on pipes within the distribution system.

Chlorine Residual Monitoring Points

The Oregon Drinking Water Program (ODWP) requires that water providers take water samples each month to monitor water quality. Based on population, Albany is required to take 40 bacteriological (bac-t) tests per month per Oregon Administrative Rule 333. Each bac-t test is also monitored for chlorine residual. Albany takes 11 to 12 samples per week, in a 4-week month. Sample data is compiled and reported monthly to ODWP. Currently, the City has twenty fixed sample sites and is working to develop additional sites. The goal is to establish 45 to 50 fixed testing stations that don't require entering buildings to obtain samples. Residual chlorine levels and bac-t tests were reviewed for the time period of 1994 to 2000 and were found to be in full compliance with all drinking water requirements. Detailed discussions of chlorine residual testing and distribution system monitoring requirements are presented in *Chapter 6 – Water System Regulatory Review* of this report.

Fire Protection

The City of Albany is responsible for providing water for fire protection to customers within the Urban Growth Boundary. Adequate fire protection requires sufficient storage, a reliable distribution system with sufficient capacity to meet fire flow requirements and a comprehensive distribution of fire hydrants. The City of Albany maintains approximately 1,425 fire hydrants including those located in Millersburg. The City has developed a hydrant in-fill program to place additional hydrants in areas that do not have adequate coverage to fight a fire. The Public Works Department and the Fire Department work together to determine areas in need of additional hydrants and set aside money annually to add hydrants. Planning criteria to meet fire flow requirements is established in *Chapter 5 - Planning Criteria*, and improvements required to meet fire protection requirements are discussed in *Chapter 10 - Distribution System Evaluation*.

Pump Stations

The City maintains five water pumping stations excluding the treatment process pumps, transfer pumps, and raw water pumps at the Vine Street WTP. Information about these pump station facilities is summarized in ***Table 3-6***. Pump curves and field inspection forms are included in ***Appendices D*** and ***E***, respectively, and locations are as shown in ***Figure 3-2***. The firm capacity indicated in ***Table 3-6*** is the capacity of the pump station with the largest pump out of service. Pumps were evaluated with the station's rated firm capacity (see *Chapter 10 – Distribution System Evaluation*). These capacities will vary with system pressures and therefore will not be realized under all demand scenarios.

Performance evaluations of the High Pressure Pump Station, 34th Avenue Pump Station and the Queen Avenue Pump Station were completed in 2001 as part of a study prepared by Brown and Caldwell.[7] The original manufacturer's curve for these pumps did not represent actual capacity shown in the field test data collected in July 2000. The field test data better represented these pumps due to consideration of headlosses in the pump suction and discharge piping. As a result, the in-situ pump curves developed as a result of the Brown and Caldwell study were used in the hydraulic model of the water system as discussed in *Chapter 10 - Distribution System Evaluation.*

These pump stations and other major facilities in the City's distribution system are integrated with a central computer at the Vine Street WTP using telemetry and monitoring equipment. Detailed information such as flow rates, reservoir levels, and system pressures are transmitted to the Plant from pump stations, reservoirs, and control valves. Monitoring points distributed at three points in pressure Zone 1, the High Service Pump Station (HSPS), the 34th Avenue Pump Station, and the Queen Avenue Pump Station, report to a pre-set pumping program for these stations. This program is designed to provide uniform and efficient pumping for a variety of demands and pressure conditions.

The pumping program works through a specific series of pumping sequences and steps up and down in the same order. When system pressures drop to pre-set low pressure levels (typically 70 psi for the HSPS and 55 psi for Queen and 34th Avenue Pump Stations) the program steps up to the next pumping sequence to increase system pressures, with a lag time of two minutes to avoid frequent pump cycling. If the desired pressure can not be obtained the next pumping sequence is triggered. If pressures increase to a pre-set high pressure level (typically 75 psi for the HSPS monitor, 80 psi for the 34th Avenue monitor, and 80 psi for the Queen Avenue monitor) the program will step down to the preceding pumping sequence.

Currently, low pressures monitored at a specific site do not necessarily trigger the pumps at that station. However, the program does allow operators to override pump and other settings remotely from the Vine Street WTP when needed. As mentioned above, settings for reservoirs, control valves, and upper level pump stations are also controlled through this program. Telemetry and monitoring equipment for these facilities is discussed in each of their respective sections below.

[7] Brown and Caldwell, *City of Albany Water System High Service Pumping Evaluation*, January 2001

Table 3-6: Distribution Pump Station Characteristics

Facility Name	Pump Name	Pump Size (hp)	Pump Flow Rate (gpm)	Pump TDH (ft)	Station Firm Capacity (gpm)	Station Nominal Capacity (gpm)	Total Horsepower (hp)	Year Built	Source/ Reservoir	Service Area/ Reservoir
High Service Pump Station (701 4th Avenue SW)	11	150	3,250	145	15,950	22,650	1,050	Original: 1959 Improvements: 2001	WTP, Maple Street Reservoir	Zone 1, Queen Ave, 34th Avenue and Broadway Reservoirs
	12	300	6,700	145						
	13	100	2,000	145						
	14	200	4,000	145						
	15	300	6,700	145						
34th Avenue Pump Station (475 34th Avenue SW)	41	50	800	150	2,800	5,800	275	Original: 1971 Improvements: 1990	34th Ave. Reservoir	Zone 1
	42	100	2,000	150						
	43	125	3,000	150						
Queen Avenue Pump Station (950 Queen Avenue SE)	21	30	500	150	500	1,900	105	Original: 1955 Improvements: 1990	Queen Ave. Reservoir	Zone 1
	22	75	1,400	150						
North Albany (Zone 2) Pump Station (1552 North Albany Road)	51	75	1,400	82	1,400	2,800	150	Original: 1980 Improvements 1998	Broadway Reservoir	Zone 2 and Wildwood Reservoir
	52	75	1,400	82						
Gibson Hill Road (Zone 3) Pump Station (3400 Gibson Hill Road NW)	61	75	900	138	900	1,800	150	Original 1998	Wildwood Reservoir	Zone 3 and Valley View Reservoirs
	62	75	900	138						

Notes:

1. Firm capacity represents the combined station capacity with the largest pump out of service.
2. Pump flow rate and TDH are based on design flow characteristics.

High Service Pump Station (701 4th Avenue SW)

Structure

The High Service Pump Station (HSPS) is located at the Vine Street Water Treatment Plant. The pumps are housed in a reinforced masonry building with a composite PVC roof installed in 2001. The structure was built in 1959 and is in satisfactory condition with no obvious sign of distress, leakage or water damage on the roof or walls. ***Photograph 3-2*** shows this facility as seen during the field inspection.

Photograph 3-2: High Service Pump Station

Equipment and Power

The HSPS is the system's largest pump station with five horizontal, centrifugal pumps and has a firm capacity 15,950 gpm (23.0 MGD). The pumps vary in size from 100 HP to 300 HP as shown in ***Table 3-6***. At the time of the inspection, pumps 12 and 15 were equipped with variable frequency drives (VFDs). However, since the inspection the City has removed the VFDs from these pumps and are not currently using VFDs at this facility. The VFDs had surpassed their service life and were replaced with soft starts. Each pump is equipped with a suction isolation valve, discharge isolation valve, pump control valve and check valve. An individual pressure gauge is mounted on the suction and discharge side of each pump.

The HSPS does not have emergency backup power. The generator onsite at the Vine Street WTP provides backup power to the Maple Street sewage pumps only. In addition, no emergency interconnect capability exists for this station, so a portable generator cannot be used to power the facility in the event of a power failure.

Telemetry and Controls

As previously mentioned a pre-set pumping program that is dependent on system pressures controls this pump station. Although this pump station is considered part of the treatment plant it can operate when the plant is off line based on the pumping program. The flow, on/off status and discharge pressure are recorded for each pump in the HSPS. Flows through the two lines leaving the pump station are metered and the data is recorded at the Vine Street WTP.

Operations

This pump station is operated based on the pre-set pumping program discussed above.

34th Avenue Pump Station (475 34th Avenue SW)

Structure

This structure was built in 1971 and improvements were made in 1990 as discussed below. The facility is comprised of a cinder block structure with a built up roof with gravel. The structure is currently in satisfactory condition with no obvious signs of distress, leakage, or water damage. ***Photograph 3-3*** shows this facility.

Photograph 3-3: 34th Avenue Pump Stations

Equipment and Power

The pump station has three horizontal, centrifugal pumps, a 50 HP, 100 HP, and 125 HP pump. Each pump has been designed for a total dynamic head (TDH) of 150 feet with flows of 800, 2,000 and 3,000 gpm, respectively. The firm capacity of the 34th Avenue Pump Station is 2,800 gpm (4.0 MGD). Each pump is equipped with a suction isolation valve, discharge isolation valve, pump control valve, and check valve. An individual pressure gauge is mounted on the suction and discharge side of each pump. None of the pumps at this station are equipped with or require variable frequency drives.

This station has a Cla-Val pressure reducing/pressure sustaining valve that, based on system pressures, allows water to flow into the station and fill the 34th Reservoir through a common feed/drain line. Typical sustaining pressure settings on the valve are 50 or 60 psi depending on how fast operators want the reservoir to fill.

The 34th Avenue Pump Station does not have an emergency generator on site or the ability to connect to a portable generator in the event of a power failure.

Telemetry and Controls

In 1990 this pump station was upgraded with telemetry and monitoring improvements. This pump station controls 34th Avenue Reservoir levels through the pre-set pumping program discussed earlier. The flow, on/off status and discharge pressure are recorded for each pump in the 34th Avenue Pump Station. The 34th Avenue Pump Station also has a Panametric flow meter that allows observations of station flow rates while on site. This information is also transmitted to, and stored at, the Vine Street WTP.

Operations

The 34th Avenue Pump Station is used to boost system pressure in Zone 1 in the vicinity of the 34th Avenue Reservoir. The pumps are operated based on the pre-set pumping program discussed earlier. The pressure sustaining/pressure reducing valve is programmed to close when the water surface elevation in the reservoir reaches 254.9 feet or when the pumps are running.

Queen Avenue Pump Station (950 Queen Avenue SE)

Structure

This structure was built in 1955 and telemetry and monitoring improvements were made in 1990 as discussed below. The pumps are housed in a cinder block building with a built up roof with gravel. The facility has windows on three sides of the building. Upon field inspection, the structure is considered to be in satisfactory condition. ***Photograph 3-4*** shows this facility.

Photograph 3-4: Queen Avenue Pump Station

Equipment and Power

The station has two horizontal centrifugal pumps (30 HP and 75 HP) with a design TDH of 150 feet and a design flow of 500 and 1,400 gpm, respectively. The firm capacity of the Queen Pump Station is 500 gpm (0.7 MGD). Each pump is equipped with a suction isolation valve, discharge isolation valve, pump control valve, and check valve. An individual pressure gauge is mounted on the suction and discharge side of each pump. None of the pumps at this station are equipped with, or require, variable frequency drives.

This station has a motorized butterfly fill valve. Based on system pressures monitored at the Vine Street WTP, the valve is opened to let flow into the station and fill the Queen Avenue Reservoir through a common feed/drain line. Typically the valve is only partially opened to minimize the change in outside system pressures.

The Queen Avenue Pump Station does not have an emergency generator on site or the ability to connect to a portable generator in the event of a power failure.

Telemetry and Controls

In 1990 this pump station was upgraded with telemetry and monitoring improvements. This pump station controls Queen Avenue Reservoir levels through the pre-set pumping program discussed earlier and the motorized butterfly valve. The flow, on/off status and discharge pressure are recorded for each pump in the Queen Avenue Pump Station. This station is equipped with a Panametric flow meter so that staff can observe station flow rates on site. This information is also transmitted to, and stored at, the Vine Street WTP.

Operations

The Queen Avenue Pump Station is used to boost pressure in Zone 1 in the vicinity of the Queen Avenue Reservoir. The pumps are operated based on the pre-set pumping program discussed above. The motorized butterfly valve is programmed to close when the water surface elevation in the reservoir reaches 260.1 feet or when the pumps are running.

North Albany (Zone 2) Pump Station (1552 North Albany Road)

Structure

This structure was built in 1980 and improvements were made in 1998, which are described below. The station is comprised of a brick building with a built up roof that is considered to be in satisfactory condition. No roof leakage or water damage was apparent within the pump house. ***Photograph 3-5*** shows this facility.

Photograph 3-5: North Albany (Zone 2) Pump Station

Equipment and Power

The station is comprised of two, 75 HP, horizontal centrifugal pumps with a design TDH of 82 feet and a design flow of 1,400 gpm each. The North Albany Pump Station was upgraded in 1998 concurrent with construction of the Wildwood Reservoir and pressure Zone 2/Zone 3 separation improvements. The upgrade consisted of the replacement of both pumps and installation of a magnetic flow meter. The firm capacity of the North Albany Pump Station is 1,400 gpm (2.0 MGD). Each pump is equipped with suction isolation valve, discharge isolation valve, pump control valve, and check valve. An individual pressure gauge is mounted on the suction and discharge side of each pump. None of the pumps at this station are equipped with or require variable frequency drives. This station is equipped with Cla-Val pressure relief valve for surge protection. If a surge occurs, water is routed through a drain line that connects to a drainage ditch. This station is also equipped with a Cla-Val pressure sustaining/pressure reducing valve vault that can be used to flow water from pressure Zone 2 back to pressure Zone 1.

The North Albany Pump Station does not have a backup generator on site. However, in the event of a power failure, the station is equipped with an emergency generator receptacle so that it can be brought back in service with the use of a portable generator.

Telemetry and Controls

This pump station is controlled by water surface elevations in Wildwood Reservoir. Using telemetry and monitoring equipment Albany staff can remotely override pump settings from the Vine Street WTP. The flow, on/off status and discharge pressure are recorded for each pump in the North Albany Pump Station. This station has a Metron magnetic flow meter that allows station flows to be read on site. This information is also transmitted and stored at the Vine Street WTP.

Operations

The North Albany Pump Station is used to boost pressure in Zone 2 and to fill the Wildwood Reservoir. This station is programmed to turn on when the water surface elevation in Wildwood Reservoir drops to 442.6 feet (winter) or 444.8 feet (summer) and turn off when the water surface elevation reaches 447.8 feet.

Gibson Hill Road (Zone 3) Pump Station (3400 Gibson Hill Road NW)

Structure

This pump station, the newest in Albany's system, was constructed in 1998 as part of the pressure zone separation improvements and is in excellent condition with no observable environmental wear on the pump house structure or roof and no observable corrosion on the internal pipeline or instrumentation cabinetry. This new structure consists of a sloped asphalt shingle roof on a wood framed building. ***Photograph 3-6*** shows the pump station.

Photograph 3-6: Gibson Hill Road (Zone 3) Pump Station

Equipment and Power

Two horizontal, 75 HP, centrifugal pumps with a design TDH of 138 feet and a design flow of 900 gpm each are used to serve pressure Zone 3. The firm capacity of the Gibson Hill Road Pump Station is 900 gpm (1.3 MGD). Each pump is equipped with suction isolation valve, discharge isolation valve, pump control valve, and check valve. An individual pressure gauge is mounted on the suction and discharge side of each pump. None of the pumps at this station are equipped with, or require, variable frequency drives. This station is equipped with a Cla-Val pressure relief valve for surge protection. If a surge occurs the pressure relief valve diverts water to a storm drain line that daylights at a ditch on Gibson Hill Road. This station is also equipped with a Cla-Val pressure sustaining/pressure reducing valve vault that can be used to flow water from pressure Zone 3 back to pressure Zone 2.

The Gibson Hill Road Pump Station does not have a backup generator on site. In the event of a power failure, the station is equipped with an emergency generator receptacle so that it can be brought back in service with the use of a portable generator.

Telemetry and Controls

Operation of this pump station is based on water surface elevations in the Valley View Reservoirs. Using telemetry and monitoring equipment Albany staff can remotely override pump settings from the Vine Street WTP. The flow, on/off status and discharge pressure are recorded for each pump in the Gibson Hill Road Pump Station. This station has a Metron magnetic flow meter that allows station flows to be read on site. This information is also transmitted and stored at the Vine Street WTP.

Operations

The Gibson Hill Road Pump Station is used to boost system pressure in Zone 3 and to fill the Valley View Reservoirs. This pump station is programmed to turn on when the water surface elevations in the Valley View Reservoirs drop below 560 feet and programmed to turn off when the water surface elevation reaches 565.7 feet. An elevation of 560 feet is used based on the topography surrounding the Valley View Reservoir site. The significance of this elevation is discussed further in the description of the Valley View Reservoirs in this chapter.

Reservoirs

The City's water system has eight reservoirs, seven used for finished water storage and one (Maple Street Reservoir) used for treatment process storage. The seven finished water storage reservoirs provide a total storage capacity of 13.5 million gallons as shown on ***Figure 3-2***. However, only 12.35 million gallons is effective storage based on operational limits for the Valley View Reservoirs as discussed below. Effective storage is defined by the amount of storage volume that is available for use. ***Table 3-7*** summarizes information about the reservoirs and ***Appendix E*** includes field inspection results. Information about the general purposes of reservoir storage is given in *Chapter 5 - Planning Criteria*. Maple Street Reservoir volume is not included in the total storage figure shown above. This reservoir is not considered finished water storage as it serves as a chlorine contact chamber for the Vine Street WTP. Each reservoir serving the Albany system is described below.

Table 3-7: Storage Reservoir Characteristics

Facility Name	Type	Volume (MG)	Diameter (ft)	Bottom Elevation (ft)	Overflow Elevation (ft)	Associated Pump Station
Maple Street Reservoir	At Grade Steel	2*	93	220.0	259.8	Vine Street WTP
Queen Avenue Reservoir	At Grade Steel	1	74	229.0	260.5	High Service Pump Station
34th Avenue Reservoir	At Grade Steel	2	104	224.0	255.5	High Service Pump Station
Broadway Reservoir	At Grade Concrete	8	210	345.0	385.0	High Service Pump Station
Wildwood Reservoir	At Grade Concrete	1.15	140	430.0	450.0	North Albany Pump Station
Valley View Reservoir 1	At Grade Steel	0.25	25	520.0	567.5	Gibson Hill Road Pump Station
Valley View Reservoir 2	At Grade Steel	0.25	25	520.0	567.5	Gibson Hill Road Pump Station
Valley View Reservoir 3	At Grade Steel	0.85	55	520.0	569.5	Gibson Hill Road Pump Station
	Total *	***13.5***				

Notes:

* The Maple Street Reservoir volume is not included in the total system volume of 13.5 MG.
The Maple Street Reservoir is used as a chlorine contact chamber and is not considered system storage.

Maple Street Reservoir (817 4th Avenue SW)

General Characteristics

Built in 1960, the Maple Street Reservoir is a 2 MG at-grade steel reservoir. It is located at the Vine Street Water Treatment Plant site and serves as a clearwell for the HSPS and as a chlorine contact chamber for the Vine Street WTP. ***Photograph 3-7*** shows the Maple Street Reservoir and its caged access ladder.

Photograph 3-7: Maple Street Reservoir

Draining & Dechlorination Facilities

The Maple Street Reservoir drains to the sanitary sewer system when it is drained for cleaning or repairs. Typically, the majority of the water is pumped into the system and only a small portion of the stored volume needs to be drained to the sewer system. The overflow piping connects to the drain line and therefore also drains to the sanitary sewer system. Dechlorination facilities are not required before draining to the sanitary sewer system.

Inlet, Outlet, & Overflow Piping

The Maple Street Reservoir has one feed line from the Vine Street WTP and two outlet lines that connect to opposite ends of the HSPS. A Hypalon baffle was installed in 1991 as part of the Vine Street WTP upgrade. The baffle increases chlorine contact time before water enters the distribution system. The City has conducted several CT tests at different reservoir levels and flow rates to determine the effectiveness of the existing baffle configuration and conducted an internal inspection in 2002. Test results showed that modifications to the baffle system are needed in order to optimize the chlorine contact time in the reservoir. These tests and the existing baffle configuration are discussed further in *Chapters 6 – Water System Regulatory Review*, *Chapter 8 – Vine Street Water Treatment Plant* and ***Appendix C***.

The Maple Street Reservoir overflow is set at 259.8 feet. Overflow piping is internal to the reservoir and connects to the drain line mentioned above. Although there have not been any problems with the overflow piping in the past, it appeared to be undersized based on observations made during the field inspection.

Seismic & Cathodic Protection

Constructed in 1960 this reservoir was subject to less stringent seismic design criteria than is required today. No improvements or upgrades have been performed on this reservoir to increase protection from a seismic event.

Cathodic protection in the form of sacrificial anodes was added to this reservoir in 1987.

Coatings

The exterior of this reservoir was last painted in 1986. Reservoir painting is a normal maintenance activity and the existing alkyd coat is anticipated to be overcoated within the next two years. The overcoat will help maintain the integrity of the existing exterior coating.

The interior of the Maple Street Reservoir was last painted in 1986 and the floor was re-coated again in 1991. City staff last inspected the interior in 2002 while evaluating the baffle system. No paint delamination, large rust areas, or tubercle build up were noted. The interior of this reservoir is anticipated to be repainted in the next five years.

Telemetry and Controls

The water level in the Maple Street Reservoir is recorded remotely and maintained in the City of Albany's System Control And Data Acquisition (SCADA) system. If the water surface elevation in the reservoir reaches 257.2 feet, 2.6 feet below the overflow, a "high level" alarm alerts plant operators at the Vine Street WTP. If the water surface elevation reaches 258.9 feet then filters 7 through 10 automatically shut down and if an elevation of 259.2 is reached then all filters are shut down. The reservoir is also equipped with an external level indicator.

Operations

Vine Street WTP production and the output of the HSPS determines water levels in the reservoir.

34th Avenue Reservoir (475 34th Avenue SW)

General Characteristics

The 34th Avenue Reservoir is an at-grade steel reservoir that has a capacity of 2 million gallons. The reservoir is located on 34th Avenue between Pacific Boulevard (State Highway 99E) and Calapooia Street. The facility was built in 1971 to serve pressure Zone 1. ***Photograph 3-8*** shows the 34th Avenue Reservoir's caged access ladder and manway.

Photograph 3-8: 34th Avenue Reservoir

Draining & Dechlorination Facilities

The 34th Avenue Reservoir drains to the stormwater system when drained for cleaning or repairs. The majority of the water is pumped into the water system and only a small portion of the stored volume needs to be drained to the stormwater system. As required by the Department of Environmental Quality (DEQ)[8], City staff adds a dechlorination agent as water is discharged into the stormwater manhole. Overflow piping is connected to the drain line, but the existing facilities do not provide for dechlorination of an overflow.

Inlet, Outlet, & Overflow Piping

The 34th Avenue Reservoir has a common inlet and outlet configuration. The overflow is set at an elevation of 255.5 feet. The overflow piping is internal to the reservoir and as mentioned above is connected to the drain line. The overflow piping for this facility is considered undersized. The undersized overflow piping was a problem in 2000 when the fill valve failed to close and water continued to flow into the reservoir even though it was full. The overflow didn't cause any structural damage to the reservoir but leaks in the overflow line (that have since been repaired) created some holes in the yard.

Seismic & Cathodic Protection

Constructed in 1971 this reservoir was subject to less stringent seismic design criteria than is required today. No improvements or upgrades have been performed on this reservoir to increase protection from a seismic event.

Cathodic protection in the form of sacrificial anodes was added to this reservoir in 1987.

[8] DEQ. Management Practices for the Disposal of Chlorinated Water, October 2000.

Coatings

This reservoir's exterior has not been painted since its original construction in 1971. During an inspection in 1995, the exterior coating of the 34th Avenue Reservoir was considered to be in fair condition[9]. Numerous points of spot damage were found. The coating was tested and the results confirm the coating to be a lead-based paint. Overcoating the existing alkyd coat is anticipated to occur within the next three years as part of normal operations and maintenance activities. This overcoat will help maintain the integrity of the existing exterior coating.

The 34th Avenue Reservoir maintains its original epoxy interior coating, which upon inspection in 1995, appeared to be in good condition[10]. Some pinhole coating failure was observed at the weld seams and some crevice corrosion was observed at the ladder rungs. In addition, some iron tubercles were identified on the floor near the inlet pipe. The interior of this reservoir is anticipated to be repainted within the next three years.

Telemetry and Controls

Fill valve settings and a pre-set pumping program for the 34th Avenue Pump Station control the reservoir level. Using telemetry and monitoring equipment Albany staff can remotely override pump and fill valve settings from the Vine Street WTP. The water level in the 34th Avenue Reservoir is recorded remotely and maintained in the City's SCADA system. The fill valve is programmed to close when a water surface elevation of 254.9 feet is reached. If the water surface elevation continues to rise to 255.3 feet, 0.2 feet below the overflow, a warning alarm at the Vine Street WTP will notify operators that the reservoir is approaching overflow conditions. The reservoir is also equipped with an external level indicator.

Operations

This reservoir along with the 34th Avenue Pump Station is used to provide adequate pressure to Zone 1 customers in the vicinity of the 34th Avenue Reservoir. A pressure sustaining/pressure reducing valve on the fill line and the pump station control the water level in the reservoir. The tank is filled by system pressure and drained by the pump station. Typical fill valve and pump station settings are provided in the description of the 34th Avenue Pump Station.

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[9] Greenberger, S. *City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings*, February, 1996

[10] Greenberger, S. *City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings*, February, 1996

Queen Avenue Reservoir (950 Queen Avenue SE)

General Characteristics

The Queen Avenue Reservoir is an at-grade, steel reservoir that has a capacity of 1 million gallons. The tank was constructed in 1955 and serves pressure Zone 1 customers in the vicinity of the reservoir site. ***Photograph 3-9*** shows the Queen Avenue Reservoir access ladder (equipped with a safety climb system) and overflow piping.

Photograph 3-9: Queen Avenue Reservoir

Draining & Dechlorination Facilities

The Queen Avenue Reservoir drains to the sanitary sewer system when drained for cleaning or repairs. Typically, the majority of the water is pumped into the system and only a small portion of the stored volume needs to be drained to the sewer system. Overflow piping is connected to the drain line and therefore also drains the sanitary sewer system. Dechlorination is not required before draining to the sanitary sewer system.

Inlet, Outlet, & Overflow Piping

The tank has an elevated common inlet/outlet configuration, resulting in some minor dead storage in the tank. The overflow piping is external to the reservoir and is undersized. The overflow piping and drain line both drain to a vault located on site. This vault then drains to the sanitary sewer as discussed above. The overflow is set at an elevation of 260.5 feet.

The undersized overflow piping was a problem in 1994. The Queen Avenue Reservoir overfilled when communications with the reservoir were lost while filling the reservoir. At the time, the communications system was not updating readings at the Vine Street WTP, and the programmable logic control (PLC) did not recognize the high reservoir level (that would have caused the control valve to be closed). Instead, the reservoir continued to fill. The overflow pipe was unable to release the water at an adequate rate; the surrounding streets were flooded and the reservoir temporarily expanded. Fortunately, no structural damage resulted from the overflow.

Seismic & Cathodic Protection

Being constructed in 1955 this reservoir was subject to less stringent design criteria than is required today for seismic protection. No improvements or upgrades have been performed on this reservoir to increase protection from a seismic event.

Cathodic protection was added to this reservoir in 1987. The cathodic protection system has platinum/niobium anodes and zinc reference cells[11].

Coatings

The interior and exterior of this reservoir were last painted in 1986. During an inspection in 1995, the exterior coating of the Queen Avenue Reservoir was considered to be in good condition[12]. Some damage to the coating was observed on the west side of the tank. The coating was tested and the results confirm the coating to be a lead-based paint. Overcoating the existing alkyd coat is anticipated to occur within the next two years as part of normal maintenance activities. This overcoat will help maintain the integrity of the existing exterior coating.

The interior coating on the Queen Avenue Reservoir consists of an epoxy coating. During the inspection performed in 1995, the coating appeared to be in excellent condition. Some minor pinhole coating failure was observed at the weld seams and some iron tubercles were identified on the floor near the inlet pipe. The interior of this reservoir is anticipated to be repainted within the next two years.

Telemetry and Controls

A pre-set pumping program and a motorized butterfly valve at the Queen Avenue Pump Station control the reservoir level. Using telemetry and monitoring equipment Albany staff can remotely override pump and valve settings from the Vine Street WTP. The water level in the Queen Avenue Reservoir is recorded remotely and maintained in the City of Albany's SCADA system. The fill valve is programmed to close at a water surface elevation of 260.1 feet. If the water surface elevation continues to rise to an elevation of 260.3 feet, 0.2 feet below the overflow, a warning alarm will notify operators at the Vine Street WTP. This reservoir is also equipped with an external level indicator.

Operations

This reservoir works with the Queen Avenue Pump Station to provide adequate pressure to Zone 1 customers in the vicinity of the reservoir. A motorized butterfly valve on the fill line and the Queen Avenue Pump Station control the water level in the reservoir. Typical control settings are provided in the description of the Queen Avenue Pump Station.

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[11] Greenberger, S. *City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings*, February, 1996

[12] Greenberger, S. *City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings*, February, 1996

Broadway Reservoir (1501 Broadway Street NW)

General Characteristics

Broadway Reservoir was constructed in 1992 and serves pressure Zone 1. It has a capacity of 8 million gallons and is constructed of concrete. The reservoir is located south of Gibson Hill Road near Broadway Street. ***Photograph 3-10*** shows the Broadway Reservoir.

Photograph 3-10: Broadway Reservoir

Draining & Dechlorination Facilities

When drained for cleaning or repairs the Broadway Reservoir drains to the stormwater system. Typically, the majority of the storage volume is allowed to feed into the water system and only a small portion of the stored volume needs to be drained to the stormwater system. City staff adds a dechlorination agent as water is discharged into a stormwater manhole in compliance with DEQ requirements[13]. Overflow piping is also connected to the manhole but the existing facilities do not provide for dechlorination of an overflow.

Inlet, Outlet, & Overflow Piping

This reservoir has a common feed and drain line to the reservoir. However, once inside the reservoir, the feed/drain line splits and the feed line outputs approximately a quarter of the way around the reservoir, while the drain line is near the point of entry. Both lines are fitted with check valves to avoid short circuiting. This type of configuration promotes water circulation within the reservoir.

The overflow piping is internal to the reservoir and connects to the drain line as mentioned above. The overflow is set at an elevation of 385.0 feet and appeared to be adequately sized based on observations made during the field inspection.

Seismic & Cathodic Protection

This reservoir was constructed in 1992 and was designed to meet more current seismic design standards (AWWA D-100 seismic zone 3) than the other reservoirs serving pressure Zone 1. Although this reservoir has structural seismic protection it is not equipped with seismic valves to prevent the reservoir from draining during an earthquake.

Cathodic protection is not required on concrete reservoirs.

[13] DEQ. Management Practices for the Disposal of Chlorinated Water, October 2000

Coatings
Broadway Reservoir is constructed of concrete and does not require internal coatings. The exterior is coated with the original stucco coating and some small cracks were visible at the time of the field inspection. The small cracks in the stucco may simply be cosmetic but should be watched for growth, leakage or discoloring. If the cracks continue to grow in length and width or leakage or discoloring around the cracks is noted, then a more comprehensive structural assessment should be made.

Telemetry and Controls
Reservoir levels are monitored at the Vine Street WTP using telemetry and monitoring equipment. The water level in the Broadway Reservoir is recorded remotely and maintained in the City's SCADA system. A "high level" alarm alerts operators at the Vine Street WTP when the water surface elevation reaches 383.7 feet. If the water surface elevation continues to rise to an elevation of 384.4, 0.6 feet below the overflow, an overflow alarm notifies operators. This reservoir is also equipped with an external level indicator and a remotely activated butterfly valve that can be shut if needed.

Operations
This reservoir fills and drains based on system pressures resulting from HSPS operation and system demands. Existing piping configurations make it difficult to fill the Broadway Reservoir during peak demand periods, sometimes taking up to 15 hours to fill. During high demand periods, filling the reservoir at a faster rate would require increasing Zone 1 system pressures beyond allowable limits. Broadway reservoir is typically operated so that filling begins when the water surface elevations drops between 377 feet (winter) and 379 (summer), depending on seasonal demands. Lower elevations would result in system pressures below the desired 65 to 70 psi in pressure Zone 1. If the reservoir were allowed to drain down any further during peak demand periods the reservoir would be harder to fill.

Wildwood Reservoir (890 Edgewood Drive)

General Characteristics

The Wildwood Reservoir is an at-grade, concrete reservoir that has a capacity of 1.15 million gallons. Built in 1999, Wildwood is the newest reservoir in Albany's water system and is the only reservoir that serves pressure Zone 2. ***Photograph 3-11*** shows Wildwood Reservoir.

Photograph 3-11: Wildwood Reservoir

Draining & Dechlorination Facilities

When drained for cleaning or repairs the Wildwood Reservoir drains through a stormwater manhole to an open ditch. The majority of the storage volume is allowed to feed into the water system and only a small portion of the stored volume needs to be drained to the stormwater system. City staff adds a dechlorination agent as water is discharged into the manhole as required by DEQ[14].

Overflow piping is connected to the drain line but the existing facilities do not provide for dechlorination of an overflow.

Inlet, Outlet, & Overflow Piping

This reservoir has a common feed and drain line to the reservoir. However, similar to Broadway Reservoir, once inside the reservoir the feed/drain line splits and the feed line outputs approximately a quarter of the way around the reservoir, while the drain line is near the point of entry. Both lines are fitted with check valves to avoid short-circuiting. This type of configuration promotes water circulation within the reservoir.

The overflow is set at an elevation of 450.0 feet and appeared to be adequately sized based on observations made during the field inspection. It is internal to the reservoir and connects to the reservoir drain line.

Seismic & Cathodic Protection

This reservoir was constructed in 1999 and was designed to meet more current seismic design standards (AWWA D-100 seismic Zone 3). Although this reservoir has seismic protection structurally, it is not equipped with seismic valves to prevent the reservoir from draining during an earthquake.

[14] DEQ. Management Practices for the Disposal of Chlorinated Water, October 2000.

Cathodic protection is not required on concrete reservoirs.

Coatings

Wildwood Reservoir is constructed of concrete and does not require internal coatings. The original external painting is in good condition. However, some minor blistering was observed below the access ladder. Damage is minor, but spot repairs should be performed.

Telemetry and Controls

Reservoir levels are monitored at the Vine Street WTP using telemetry and monitoring equipment. Water levels are recorded remotely and maintained in the City's SCADA system. When the water surface elevation in the reservoir reaches 448.8 feet, 1.2 feet below the overflow, a warning alarm will notify operators at the Vine Street WTP. This reservoir is also equipped with an external level indicator.

Operations

The Wildwood Reservoir fills when the North Albany Pump Station turns on and drains based on system pressures in Zone 2. The pump station is programmed to turn on when the water surface elevation in the reservoir drops between 442.6 feet (winter) and 444.8 feet (summer) and programmed to turn off at an elevation of 447.8 feet. Pump station settings are discussed in the description of the North Albany Pump Station in this chapter.

Valley View Reservoirs (3240 Valley View Drive)

General Characteristics

Three reservoirs serve pressure Zone 3, including two small steel reservoirs (0.25 MG each) and one larger steel reservoir (0.85 MG). The total storage capacity of the three reservoirs is 1.35 MG. The two smaller reservoirs were constructed in 1963 and 1967, and the largest reservoir was built in 1982. ***Photograph 3-12*** shows the Valley View Reservoirs and the caged access ladder as seen on the site visit. A cat walk is used to walk between the tops of the three reservoirs.

Photograph 3-12: Valley View Reservoirs

Draining & Dechlorination Facilities

The Valley View Reservoirs drain to the stormwater system when drained for cleaning or repairs. City staff adds a dechlorination agent, as required by DEQ[15], as water is discharged into a stormwater manhole near Valley View Drive. Overflow piping is connected to the drain line, but the existing facilities do not provide for dechlorination of an overflow. Down stream capacity of the stormwater system is inadequate to meet required flows when draining the reservoirs and some flooding of drainage ditches has been experienced in the past. The City recently installed connections to the reservoirs so that the storage of an isolated reservoir can be pumped either into the system or into another reservoir. This will avoid wasting water and should minimize flooding of drainage ditches when the remaining water is drained.

Inlet, Outlet, & Overflow Piping

Each of these reservoirs are connected to a common feed/drain line. The overflow piping on these reservoirs is restricted and undersized. Each overflow is internal to the reservoir and connects to the reservoir drain line. The overflow for the two smaller reservoirs is set at 567.5 feet and the overflow for the larger reservoir is set at 569.5 feet.

Seismic & Cathodic Protection

Constructed in the 1960's and early 80's these reservoirs were designed to meet less stringent seismic design standards than are required today. Therefore, these reservoirs are vulnerable to damage during a seismic event.

[15] DEQ. Management Practices for the Disposal of Chlorinated Water, October 2000.

Cathodic protection was installed for these reservoirs in 1987. Each of the three tanks has an impressed current cathodic protection system, with an automatically controlled rectifier, segmented mesh anodes and copper/copper sulfate reference cells[16].

Coatings

The two smaller .25 MG reservoirs were last re-coated, interior and exterior, in 1981. The larger .85 MG reservoir retains its original coating systems with some spot repairs to the interior weld seams in 1986. The three tanks were last inspected in 1995[17]. All three tanks have alkyd, lead-based exterior coatings. During the inspection, the coatings had appreciable algae and mildew buildup and were heavily chalked. Due to the lead-based content of the coating the exterior coating of all three tanks is anticipated to be overcoated or encapsulated in the next four years as part of normal maintenance activities.

All three tanks have a coal tar interior coating. During the inspection, the coatings were observed to be in generally good condition with some blistering identified at the weld seams. The interior of these reservoirs are anticipated to be repainted in the next four years.

Telemetry and Controls

Reservoir levels are monitored at the Vine Street WTP using telemetry and monitoring equipment. The water levels are recorded remotely and maintained in the City of Albany's SCADA system. If the water surface elevation in the reservoirs reaches an elevation of 566.0 feet, 1.5 feet below the overflow, a warning alarm will notify operators at the Vine Street WTP. The larger reservoir is equipped with an external level indicator.

Operations

All of the reservoirs operate at the same hydraulic grade line, with a maximum elevation equal to 567.5 feet, the overflow elevation of the two smaller reservoirs. Due to the surrounding topography, the full capacity of the reservoirs cannot be utilized, resulting in an effective capacity of 0.2 million gallons for the three tanks. When the water surface elevation in the reservoirs approaches 560 feet, homes at an elevation of 468 feet and higher (approximately 40 homes) experience service pressures less than 40 psi and customers at elevations near 510 feet (approximately 10 homes) experience pressure approaching the state minimum of 20 psi. To avoid this situation the Gibson Hill Road Pump Station is programmed to turn on and fill the Valley View Reservoirs when the water surface elevation in the reservoir drops to 560.0 feet. The pump station is programmed to turn off at an elevation of 565.7 feet.

[16] Greenberger, S. City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings, February, 1996

[17] Greenberger, S. *City of Albany Water Reservoirs and Secondary Digester Survey-Cathodic Protection and Coatings*, February, 1996

CHAPTER 4 – POPULATION AND WATER DEMANDS

INTRODUCTION

Reviewing existing water use patterns and projecting future demands are fundamental to development of a sound water facility plan. This chapter summarizes Albany's existing water demands and the methodology used to project future water demands. This methodology relied on population projections for residential customers and land use planning analysis for non-residential uses within Albany's future service area. This area includes the Urban Growth Boundary (UGB) and limited service to areas outside the UGB in North Albany and does not include the City of Millersburg. Millersburg will be served by the Joint Water Project as discussed in *Chapter 9 - Joint Water Project* and consequently has not been included in population and demand forecasts.

POPULATION

Current Population

The City of Albany's current population (2000) is 40,852[18] based on data provided by the U.S. Census Bureau. The Center for Population and Research and Census at Portland State University (PSU) also maintains historic population data and this data is summarized in ***Table 4-1*** and in ***Figure 4-1***. The information from PSU represents estimates of Albany's population on July 1 of each year. These estimates are based on census counts published by the US Census Bureau every ten years. Annual estimates between census counts are derived by analyzing supplemental data, including economic changes, building permits, vehicle registrations, annexations and other data. On average, Albany's population has increased at a rate of approximately 2.8 percent annually since 1950. Albany's population increased sharply between 1990 and 1991 as a result of annexing the North Albany area. Since 1991, Albany's population has grown at a rate approximately 2.0 percent annually.

Population Projections

The projected population at buildout (full development) of the UGB is based on 2000 census data and an estimate of additional population as undeveloped residential areas are fully developed. Estimates of additional population are based on an inventory of residentially-zoned land within the UGB.

Residentially-zoned areas outside Albany's city limits but within the UGB are identified as "Rural Residential" in the City's *Comprehensive Plan*[19]. City Planning and Engineering staff projected urban residential, commercial and public land uses for these areas to estimate the buildout population and non-residential demands at full development of the UGB.

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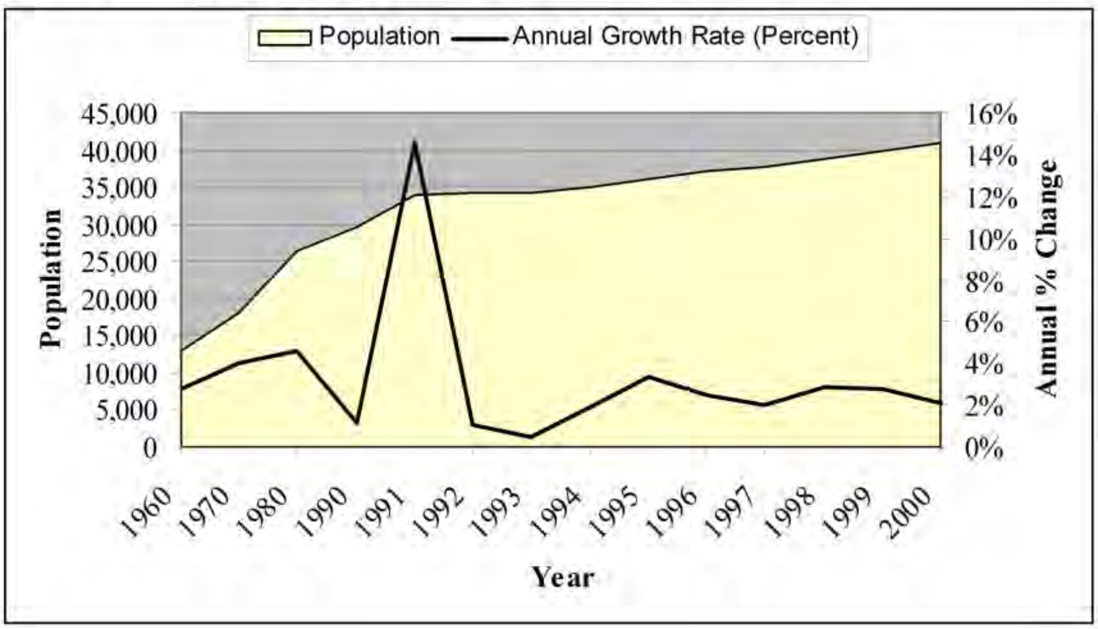
[18] US Census Bureau., *2000 Census of Population and Housing, Oregon.*, May 2001

[19] City of Albany, *Comprehensive Plan,* January 1989.

Table 4-1: City of Albany Historic Population Data[20,21]

Year	Population	Annual Growth Rate (Percent)
1950	10,115	-
1960	12,926	2.78%
1970	18,181	4.07%
1980	26,540	4.60%
1990	29,540	1.13%
1991	33,850	14.59%
1992	34,200	1.03%
1993	34,350	0.44%
1994	35,020	1.95%
1995	36,205	3.38%
1996	37,095	2.46%
1997	37,830	1.98%
1998	38,925	2.89%
1999	40,010	2.79%
2000	40,852	2.10%

Figure 4-1: Albany's Population Growth, 1960-2000



[20] Official decennial count provided by the United States Bureau of Census, 1950
[21] Portland State University Center for Population Research and Census, *Annual Estimate*, July 1, 1991

The inventory of residential land uses was screened to determine the net acreage available for future residential development. This screening was done on a parcel by parcel basis using the City's geographic information system (GIS) database. A ratio of assessed improvement value to parcel area was used to classify a parcel as fully developed, partially developed or vacant. For example, the ratio of assessed improvement value to lot area for a parcel with an assessed improvement value of $60,000 and a 12,000 square foot lot is 5.0. Several ratios were considered and contrasted with actual levels of development and field checked for sample properties. Based on this evaluation, a ratio of improvement value to developed lot area of 3.40 was selected to identify parcels that are fully developed. The following criteria were used to distinguish residential parcels as fully developed, partially developed or vacant:

- Developed: ratio of improvement value/lot area is greater than or equal to 3.4:1;
- Partially developed: improvement value is greater than $20,000 and ratio of improvement value to lot area is less than 3.4:1, and
- Vacant: improvement value is less than $20,000.

Developed areas were not used to project additional population. Partially developed and vacant areas were further screened to remove uses that are permitted in residential zones but that will not develop residentially. Examples of these areas include cemeteries, schools, wetlands, and flood plains. These areas were not included in the net area available for residential development.

Population projections typically rely on the projected number of homes or dwelling units per acre for residential land uses and on estimated household densities or persons per household. An average household density of 2.46 persons per household was used to project the buildout population. This is consistent with the average household density developed with the *1998 Wastewater Facility Plan*[22] and was confirmed with data from the 2000 Census[23]. Dwelling unit densities were projected for vacant and partially developed residential land uses based on the City's *Development Code*[24] and the resulting population densities per acre (ppa) are summarized in ***Table 4-2***.

Table 4-2: Projected Population Densities per Acre

Residential Land Use		*DU/Acre*		*Population/Acre*	
Description	***LU Code***	***(Vacant)***	***(Partial Dev.)***	***Vacant***	***Part. Dev.***
Low density SFD[a]	RS 10	4	3	10	7
Low density urban SFD	RS 6.5	6	4	15	10
Low-Medium density SFD	RS 5	6	5	15	12
Low-medium density MFD[b]	RM 5	12	10	30	25
Medium-high density MFD	RM 3	20	16	49	39

[a] Single Family District

[b] Multiple Family District

[22] CH2M-Hill, *City of Albany Wastewater Facility Plan*, June 1998
[23] US Census Bureau, *Profiles and General Demographic Characteristics, 2000*
[24] City of Albany, *The Albany Development Code*., Adopted September 1981, last amended July 11th, 2001

The projected population increase was determined by multiplying the total area available for each residential land use by the population per acre densities shown in ***Table 4-2***. The population increase was added to the current (2000 census) population to determine the total projected service population at buildout. ***Table 4-3*** summarizes these population projection calculations.

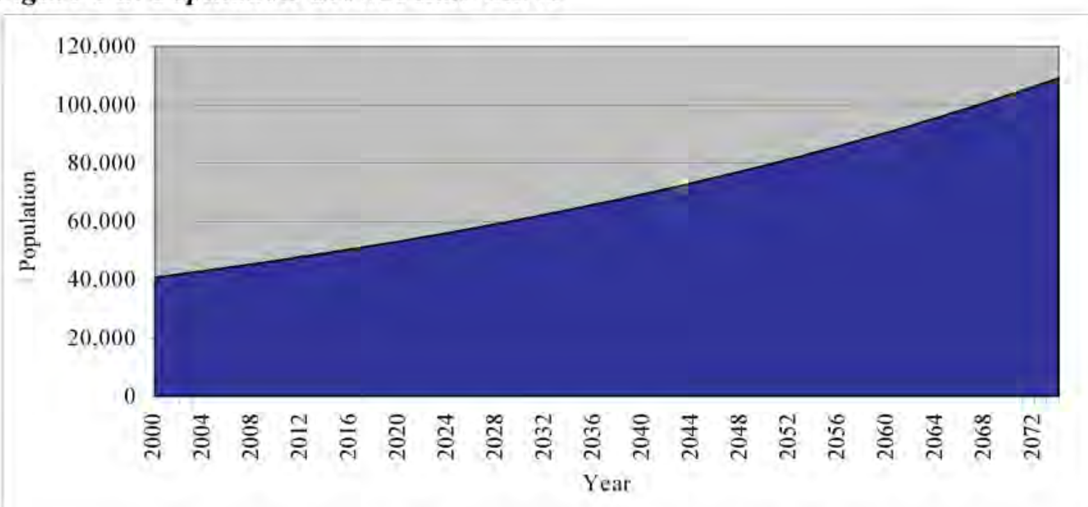
Table 4-3: Projected Population at Buildout

Residential Land Use		Vacant		Partial Dev.		Total
Description	***LU Code***	***Acres***	***Population***	***Acres***	***Population***	***Population***
Low density SFD	RS 10	806	7,931	947	6,989	14,920
Low density urban SFD	RS 6.5	1,706	25,181	1,157	11,385	36,565
Low-medium density SFD	RS 5	55	812	7	86	898
Low-medium density MFD	RM 5	268	7,911	115	2,829	10,740
Medium-high density MFD	RM 3	95	4,674	10	394	5,068
	Total	*2,930*	*46,509*	*2,236*	*21,682*	*68,191*
					2000 population	*40,852*
					Total	109,043
					Total buildout population (rounded)	***109,000***

Population Growth Rate

The City of Albany coordinated with Linn and Benton Counties to develop a population projection of 53,200 for 2020[25]. This population was used to establish the rate of change or growth from the 2000 census population. The projected 2020 population of 53,200 is an increase of approximately 12,400 persons and represents an average annual growth rate of approximately 1.34 percent. An annual growth rate of 1.34 percent was used to project population growth beyond 2020 until the UGB is fully developed (2074). ***Figure 4-2*** and ***Table 4-4*** illustrate projected population increases over time based on this growth rate.

Figure 4-2: Population Growth Rate Curve



[25] Coordinated population projections prepared in accordance with ORS 195.036, Linn County Order No. 99-324 adopted June 23, 1999, and Benton County Ordinance No. 99-0149, adopted February 10, 1999.

Table 4-4: Projected Population Growth to Buildout of the UGB

Population	*2000*	*2005*	*2010*	*2015*	*2020*	*2025*	*2074*
Current	40,852	-	-	-	-	-	-
Projected	-	43,600	46,600	49,800	53,200	56,900	109,000

As discussed above, the projected growth rate is based on regional, formally adopted population forecasts for 2020. The projected growth rate is lower than Albany has experienced and may understate the pace of actual growth. This rate has been used, however, for consistency with the City's *Comprehensive Plan*.

WATER DEMAND

Historic Water Demand

Water production and consumption records are available from meter data collected at the Vine Street Water Treatment Plant and through metered usage records available from the City's utility billing database. The seven-year period from 1994 through 2000 was used as a basis to establish historic water demands and characteristics. Water production data and fluctuations in reservoir levels were used to characterize system demands over this period. Utility billing records were used to characterize use by customer class and to record total metered sales. The difference between production and metered sales, including unmetered but accounted for water uses (i.e. hydrant flushing, fire fighting), is the volume of water unaccounted for in a system. Unaccounted for water includes leakage losses, stolen water, illegal connections, and meter error.

A record of demands during the period from 1994 through 2000 is presented in ***Table 4-5***. The annual average demand has increased approximately 9 percent during this period, from 7.5 MGD to 8.2 MGD. The historic demands shown in ***Table 4-5*** were used to develop the existing system demand estimates used in the plan as well as the existing and projected maximum to average day demand multipliers. An 8.0 MGD estimate of existing average day demand is used based on the average daily demands over the last 7 years.

Figure 4-3 depicts water treatment plant production, calculated demand, total metered sales and an estimate for unaccounted for water from 1995 through 2000. Unaccounted for water shown in this figure was calculated by subtracting metered sales from the corresponding estimate of water production at the Vine Street Water Treatment Plant (WTP). Water audits used to estimate unaccounted for water normally include an allowance for unmetered uses such as fire hydrant flushing, fire suppression and water used in wastewater collection system maintenance. These allowances are typically a very small fraction of a community's total annual water production and have not been included in our estimate of unaccounted water because of the approximate method used to estimate finished water production.

In projecting water demands it is important to understand seasonal variations due to the influence of temperature and precipitation. As would be expected, water demands increase during hot dry weather, primarily to meet irrigation needs. The relationship of average daily temperature and precipitation to demands is shown in ***Figure 4-4***.

Table 4-5: Summary of Demands from 1994 through 2000

HISTORICAL DEMAND (MGD)[1]									
YEAR	ANNUAL AVERAGE	PEAK SEASON[2] AVERAGE	OFF-SEASON[3] AVERAGE	MINIMUM MONTHLY AVERAGE		MAXIMUM MONTHLY AVERAGE		MAXIMUM DAILY[3]	
				DATES	VALUE	DATES	VALUE	DATE	VALUE
1994	7.5	10.1	6.2	2/1-2/28	5.9	7/1-7/31	12.1	7/20	15.5
1995	7.4	9.9	6.2	2/1-2/28	5.9	7/1-7/31	11.2	7/17	14.8
1996	7.8	10.3	6.6	1/1-1/31	6.1	7/1-7/31	8.8	7/12	14.3
1997	7.6	9.6	6.5	2/1-2/28	6.2	7/1-7/31	10.8	8/20	17.4
1998	8.2	11.0	6.8	2/1-2/28	6.4	7/1-7/31	12.1	7/27	16.1
1999	8.2	10.7	7.0	2/1-2/28	6.7	7/1-7/31	11.7	7/27	15.0
2000	8.2	11.0	6.8	2/1-2/28	6.5	7/1-7/31	12.5	8/1	15.0
7 YR AVG	***7.8***	***10.4***	***6.6***		***6.2***		***11.3***		***15.4***

[1] Calculated based on Raw Water Flow less 1.5 x Backwash Water Used - Total Water Stored - Estimate of Unaccounted-for Water
(On average, raw water pumping is approximately 5% higher than plant production. This 5% is assumed in the 1.5 x Backwash Water Used.)

[2] Peak season is June through September

[3] Off season is October through May

[4] Values Exceeding 16.0 MGD for Max Daily Flow are potentially suspect due to possible internal overflows and meter inaccuracies.

Figure 4-3: Production, Demand and Unaccounted for Water from 1994 through 2000

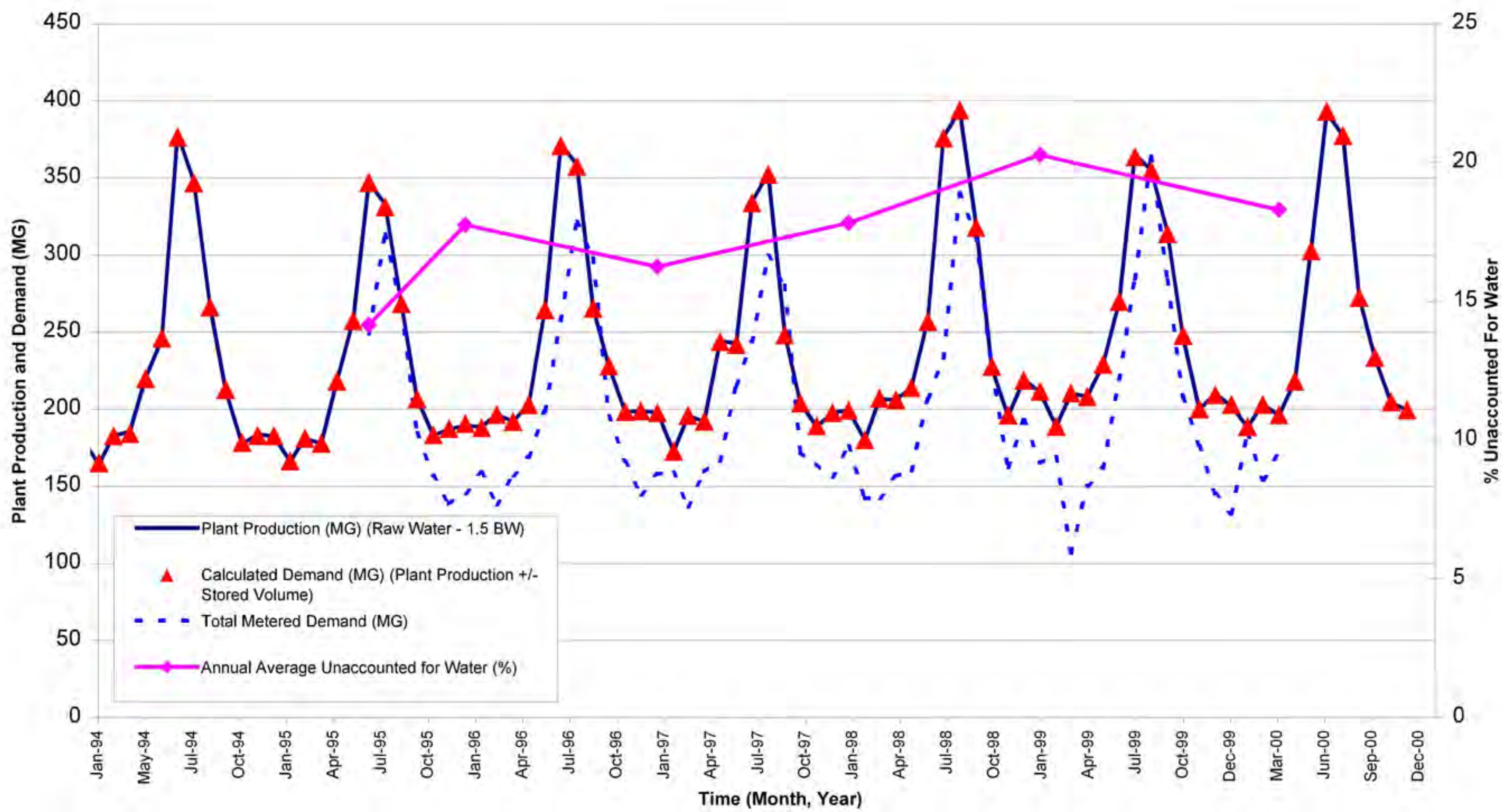
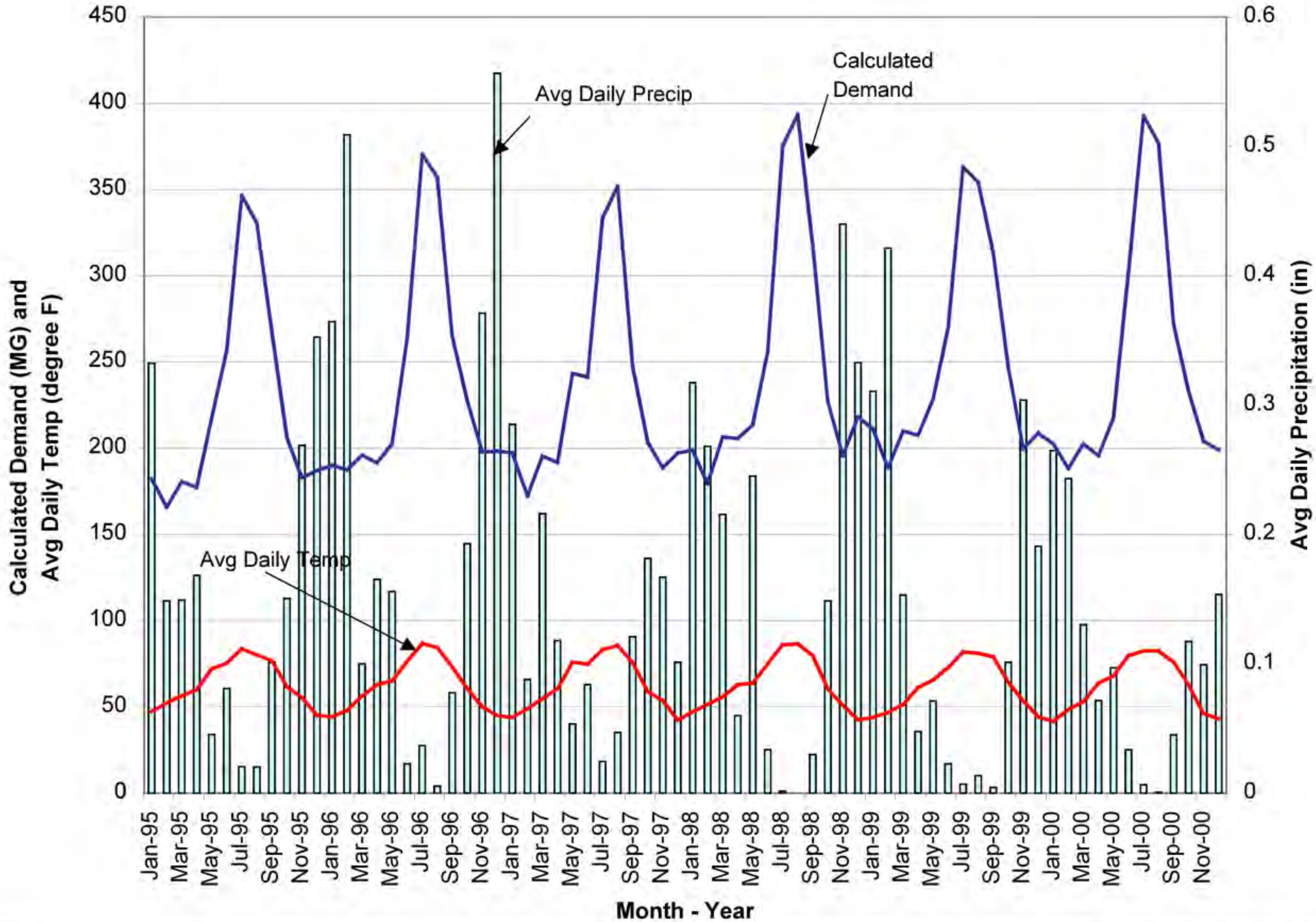


Figure 4-4: City of Albany Calculated Demand with Temperature and Precipitation



Projected Water Demand

A discussion of the approach used to estimate residential and non-residential demands, water loss and non-residential water reserve is presented below.

Residential

Residential demands are based on projected population estimates and unit demands per capita. Utility billing data for single-family residential customers was used to estimate average annual residential water demands. Residential billing data for four years (July 1995 to July 1999) was analyzed and a residential average per capita demand of approximately 95 gpcd was determined. This compares favorably with the *1988 Albany and Millersburg Water System Facility Plan*[26] that estimated an average demand of 98 gallons per capita per day (gpcd).

Based on this data an average per capita demand factor of 100 gpcd has been used to project residential demands. This is a slightly conservative per capita demand for residential uses and should be updated in future planning efforts to reflect the influence of water conservation and curtailment programs overtime.

Non-Residential Demand

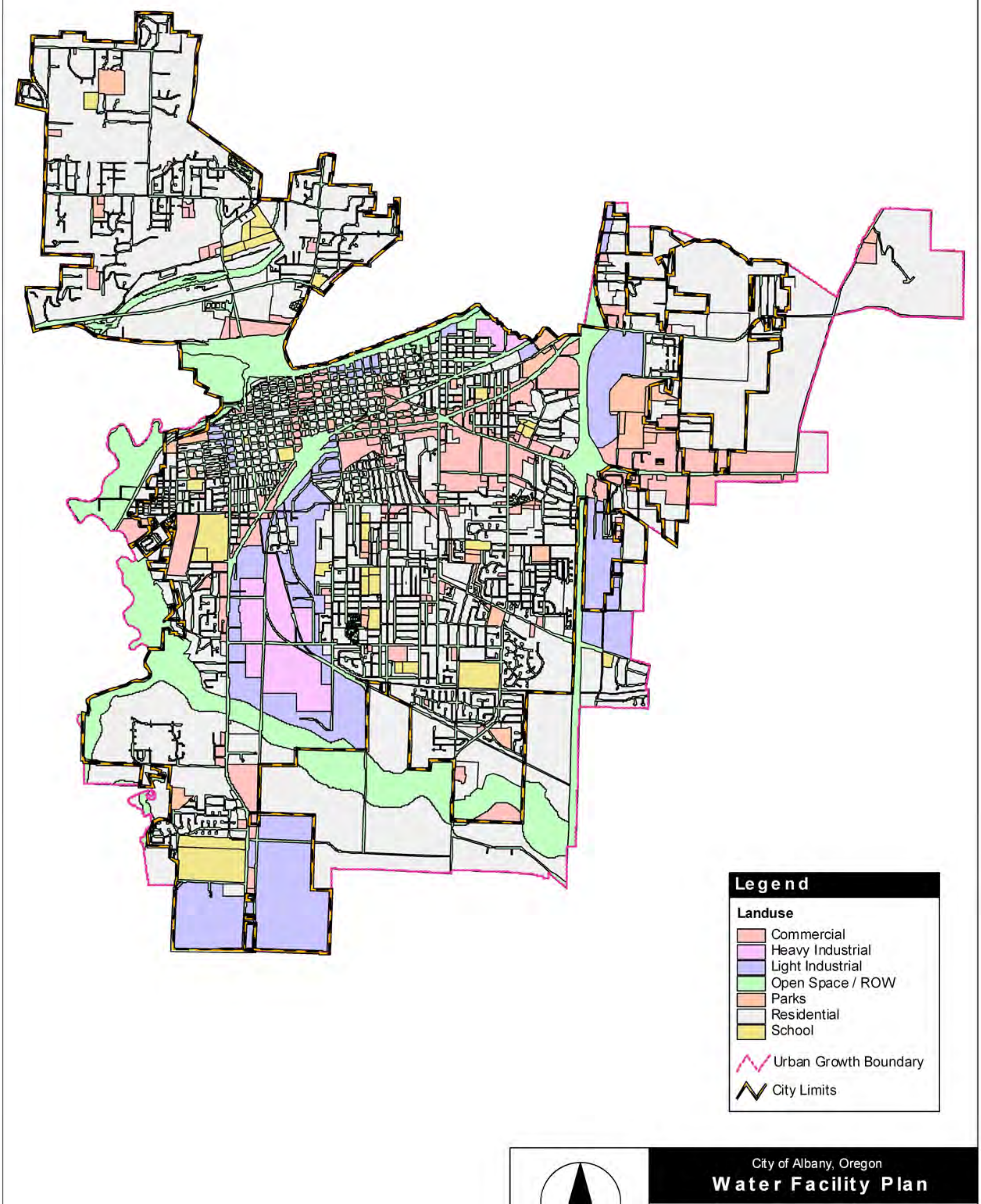
Non-residential demands include all commercial, heavy and light industrial, school and park demands. Demand estimates for these uses are estimated based on land use and a corresponding unit demand per acre. A summary of non-residential land uses is shown in ***Table 4-6*** and a map showing the location of these land uses is included as ***Figure 4-5***.

Table 4-6: Non-Residential Land Uses in the UGB

Land Use	*Area (ac)*
Commercial	1,100
Heavy Industrial	300
Light Industrial	1,200
Schools	300
Parks	300
Total Area	***3,200***

The American Water Works Association (AWWA) is a nationally recognized data source used to forecast water demands. According to AWWA[27], "in a metered community, the best way to determine water demand by land use is to examine actual water usage for the various types of land uses. The goal of evaluating water demands by land use is to develop unit water duties *(demands)* for non-residential land uses that can be used for future planning."

[26] Brown and Caldwell Consulting Engineers. *Albany and Millersburg Water System Facility Plan*. February 1988
[27] AWWA, *American Water Works Association (AWWA) Manual 32, Distribution Network Analysis for Water Utilities*, 1989


Legend
Landuse
Commercial
Heavy Industrial
Light Industrial
Open Space / ROW
Parks
Residential
School
Urban Growth Boundary
City Limits
City of Albany, Oregon
Water Facility Plan

AWWA has published[28] a range of typical water demands based on land uses. This range was contrasted with a survey of Albany demands for non-residential customers. Unit demands for each land use were then selected. Sample non-residential demand data for year 2000 usage was divided by each customer's developed acreage to determine an average demand per acre per day. This comparison and the average daily unit water demands used to forecast non-residential demands for this plan are summarized in ***Table 4-7***.

Table 4-7: AWWA and Albany Unit Demands for Non-Residential Uses

Land Use	*AWWA (gpad)* Low	High	Average	*Albany Survey (gpad)*	*Albany Water Facility Plan (gpad)*
Office Commercial	1,100	5,100	2,030	-	-
Retail Commercial	1,100	5,100	2,040	-	-
Commercial (Albany)	-	-	-	1,350	2,000
Light Industrial	200	4,700	1,620	1,830	1,600
Heavy Industrial	200	4,800	2,270	4,830	4,800
Schools	400	2,500	1,700	620	600
Parks	400	3,100	2,020	730	700

As can be noted from ***Table 4-7***, AWWA average water demands for commercial uses were greater than Albany's average commercial demand. The survey of Albany's commercial demands found the largest demand variation between sites and was based on a small sample size. AWWA average values were therefore selected for commercial customers because of this variation and small sample size. Rounded AWWA water demands were used for light industrial land use based on the close correlation between these unit demands and Albany's demand data. The high end of AWWA's unit demands for a heavy industrial use was selected because it reflects Albany's relatively water intensive industrial uses such as metals casting and food processing. Unit demands for schools and parks are based on Albany's survey data and are near the lower range of the AWWA typical demands for these uses.

Reserve

A buildout reserve of three million gallons per day of non-residential water demand is included in the total water demand forecast. This is a policy recommended by the City's Water Task Force as demand decisions were developed early in the planning process. The non-residential reserve is intended to provide flexibility and capacity to accommodate more water intensive commercial and industrial customers. For facility planning purposes, the added demand for a non-residential reserve has been spread uniformly over non-residential

[28] Mays, Larry (in association with AWWA), *American Water Works Association (AWWA) Water Distribution System Handbook*. McGraw-Hill, 2000

properties in pressure Zone 1. The reserve is expected to be phased in, with the first 1-MGD capacity provided by 2025 and the remaining 2-MGD capacity provided as projected demands increase between 2025 and buildout of the UGB. The reserve recognizes the difficulty in using a single unit water demand for any land use and the advantages of having reserve capacity in place should a water intensive customer express interest in locating in Albany. Unlike other demands, peaking factors have not been applied to the non-residential reserve in anticipation that these more water intensive uses will be relatively consistent demands.

Water Loss

Water loss is defined as the difference between water that is produced and water that is sold or otherwise accounted for. Unmetered uses that are often accounted for as estimates include

such activities as street sweeping, construction, hydrant flushing and fire suppression. Water losses are typically attributed to leaks, unmetered and unaccounted for water use, and inaccurate metering equipment both at the source (Water Treatment Plant) and for individual customer water meters.

An acceptable water loss rate is considered to be no greater than 15 percent[29]. The current loss rate for Albany is estimated to be substantially greater than 15 percent, although the actual loss rate is difficult to determine because of metering problems at the Vine Street Water Treatment Plant and difficulty in accessing recent consumption data from the City's utility billing database. Based on production estimates and available historic utility billing data, approximately 20 percent of the total water production is currently unaccounted for on an annual basis. The majority of this loss is assumed to be the result of leakage from deteriorated steel water lines. One of the recommendations of this plan, noted in *Chapter 10 - Distribution System Evaluation*, is development of an aggressive steel pipe replacement program. This program is expected to replace all steel water lines over the next 15 to 20 years. Over time this program will reduce the current water loss rate. The City has established a long-term goal of reducing the current loss rate to 15 percent and this allowance is included as part of the projected water demand requirement at buildout of the UGB.

Demand Variations

The water demands discussed above are average daily demand (ADD) values expressed on a per capita or per acre basis. Variations in these average demands are used to size treatment plant, reservoir, pump station and transmission and distribution facilities. Treatment plants, transmission lines and pump stations serving a pressure zone with reservoir storage are sized to meet maximum day demands (MDD) expected for their respective design periods. Peak hourly demands (PHD) are used as one of the criteria for sizing distribution water lines, reservoirs, and pump stations serving pressure zones without reservoir storage.

For water facility planning purposes MDD and PHD are typically expressed as a ratio of ADD. The ratios or peaking factors used in development of Albany's facility plan are discussed below.

[29] Mays, Larry (in association with AWWA), *American Water Works Association (AWWA) Water Distribution System Handbook*. McGraw-Hill, 2000

Maximum Day Demand (MDD)

As shown in ***Table 4-8***, Albany's average MDD peaking factor is 2.0 from 1994 through 2000. This value is contrasted with six other Willamette Valley communities in ***Table 4-9***. Similar to Albany, these MDD peaking factors reflect system-wide demand peaks, including residential, commercial, industrial, institutional and other use categories.

Table 4-9: MDD Peaking Factors for Other Willamette Valley Communities

Community	ADD (MGD)	MDD (MGD)	PHD (MGD)	Peaking Factors (MDD/ADD	(PHD/ADD)
City of West Linn	2.7	5.8	11.6	2.1	4.3
Clackamas River Water (Area 1)	10.8	22.8	45.6	2.1	4.2
City of Milwaukie	2.6	4.9	9.8	1.9	3.8
City of Phoenix	0.6	1.2	2.4	2.0	4.0
City of Wilsonville	2.4	4.8	9.6	2.0	4.0
City of Lake Oswego	5.7	11.1	22.2	1.9	3.9
			Average Peaking Factor	2.0	4.0

According to AWWA Manual 32, *Distribution Network Analysis for Water Utilities*, MDD peaking factors typically range from 1.2 to 2.5. Based on the calculated value and comparison to other Willamette Valley communities, a peaking factor of 2.0 has been used in this plan to project future MDDs.

Peak Hourly Demand (PHD)

The PHD is the greatest hourly demand expected during the MDD. It is more cost effective to meet these short-term demands through reservoir storage than added treatment plant capacity. Because meter information used to develop the MDD/ADD multiplier did not provide hourly data; a PHD/ADD ratio of 4.0 was used based on the higher-end of the "common range" developed in the AWWA *Water Distribution System Handbook*. The common range values are 2.5:1 – 4.0:1[30].

Summary of Projected Demands

Projected unit demands per capita and per acre have been multiplied by the projected buildout population and inventory of land uses to develop a total water demand at buildout. These calculations are summarized in ***Table 4-10*** and result in a projected MDD of 40 MGD.

[30] Mays, Larry (in association with AWWA). *American Water Works Association (AWWA) Water Distribution System Handbook*. McGraw-Hill, 2000. "Chapter 3 System Design: An Overview Table 3-6."

Table 4-8: Water Demand Variations, 1994 through 2000

Month	Average Monthly Demand							Average
	1994	*1995*	*1996*	*1997*	*1998*	*1999*	*2000*	*1994-2000*
January	5.81	5.88	6.13	6.37	6.42	6.81	6.54	6.28
February	5.89	5.93	6.70	6.16	6.42	6.74	6.48	6.33
March	5.89	5.83	6.32	6.31	6.66	6.78	6.54	6.33
April	6.13	5.91	6.39	6.39	6.86	6.92	6.54	6.45
May	7.08	7.02	6.53	7.85	6.90	7.37	6.97	7.10
June	8.19	8.56	8.80	8.05	8.54	8.99	9.98	8.73
July	12.13	11.18	11.95	10.76	12.11	11.71	12.54	11.77
August	11.17	10.67	11.52	11.35	12.69	11.43	12.29	11.59
September	8.86	8.93	8.83	8.26	10.59	10.45	9.16	9.30
October	6.84	6.65	7.34	6.57	7.33	7.97	7.56	7.18
November	5.94	6.10	6.60	6.29	6.53	6.66	6.82	6.42
December	5.89	6.03	6.40	6.37	7.04	6.74	6.65	6.45
Demands								
Minimum Monthly Average Day[1]	5.8	5.8	6.1	6.2	6.4	6.7	6.5	6.2
Maximum Monthly Average Day[1]	12.1	11.2	12.0	11.4	12.7	11.7	12.5	11.9
Average Peak Season[2] Day	10.1	9.9	10.3	9.6	11.0	10.7	11.0	10.4
Average Non-Peak[3] Season Day	6.2	6.2	6.6	6.5	6.8	7.0	6.8	6.6
Maximum Day Demand (MDD)	15.5	14.8	14.3	17.4	16.1	15.0	15.0	15.4
Average Day Demand[1] (ADD)	7.5	7.4	7.8	7.6	8.2	8.3	8.2	7.8
Peaking Factors								
MDD/ADD	2.1	2.0	1.8	2.3	2.0	1.8	1.8	2.0
MDD/Average Peak Season Day	1.5	1.5	1.4	1.8	1.5	1.4	1.4	1.5
Average Peak Season Day/ADD	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3

[1] Calculated based on Raw Water Flow less 1.5 x Backwash Water Used - Total Water Stored - Estimate of Unaccounted-for Water

[2] Peak season is June through September

[3] Non-peak season is October through May

Table 4-10: Projected Water Demand at Buildout

Customer Type	Demand Rate	Units	Population/ Acres	Buildout Land Use (acres)	Average Day Demand (MGD)	Maximum Day Demand (MGD)
Residential	100	gal/capita/day	109,000		10.9	
Commercial	2,000	gal/day/acre	1,100	1,100	2.2	
Light Industrial	1,600	gal/day/acre	1,200	1,200	1.9	
Heavy Industrial	4,800	gal/day/acre	300	300	1.4	
Parks	700	gal/day/acre	300	300	0.2	
School District & LBCC	600	gal/day/acre	300	300	0.2	
				Subtotal (rounded)	*16.9*	*34*
				Water Loss (15% of ADD)	2.5	3
				Total, w/o Reserve (rounded)	*19.4*	*37*
				Industrial Reserve	*3.0*	*3*
				Total, w/ Reserve (rounded)	***22.4***	***40***

As noted earlier, an annual population growth rate of 1.34 percent has been used to project population increases. Based on this growth rate, buildout of the UGB is expected to occur in approximately 2074. A demand curve representing this incremental growth

rate and additional non-residential demands is shown as ***Figure 4-6***. Projected average and maximum day demands are also shown in ***Table 4-11***. Both ***Figure 4-6*** and ***Table 4-11*** reflect a 1-MGD demand decrease (MDD) in 2006, as Millersburg demands will be met by the Joint Water Project.

Table 4-11: Projected Demands to Buildout of the UGB

Demand	2000	2005	2010	2015	2020	2025	2050	Buildout
ADD	8.0	8.5	8.4	9.2	10.1	11.0	16.0	22.4
MDD	16.0	16.9	17.2	18.5	19.9	21.3	29.6	40.0
PHD	32.0	33.8	34.7	37.0	39.3	41.8	56.2	74.0

The populations and water demands developed in this chapter are used to project the location, amount and type of water demands expected as the City grows. Buildout demands have been included in the hydraulic model of the distribution system to determine required water line sizes and incremental growth of demands have been used to forecast timing for reservoir, pump station and treatment plant improvements.

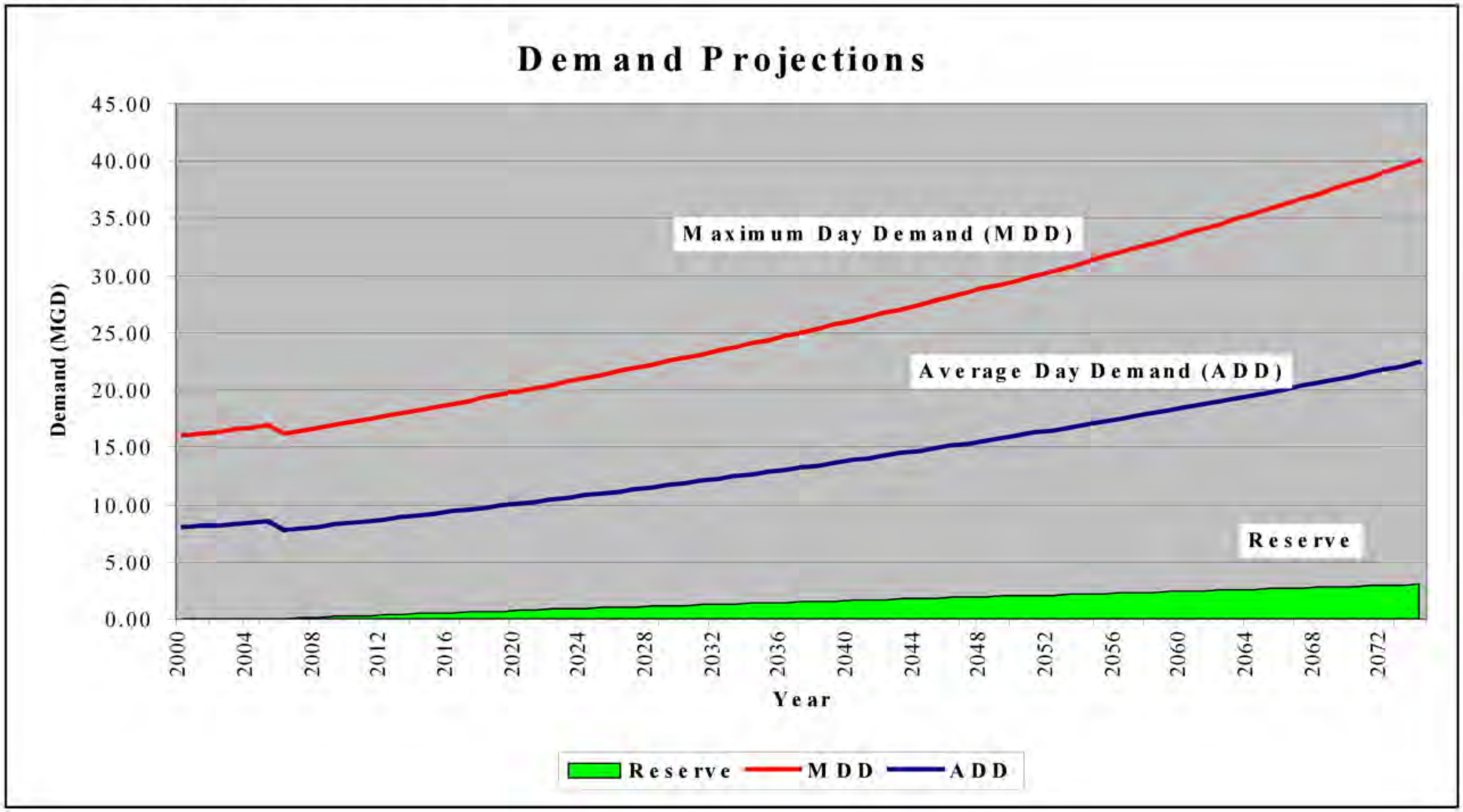
A summary of projected population and demands by pressure zone is shown in ***Table 4-12***. This table includes population and demand projections for pressure Zone 4, an area currently within pressure Zone 3 that will be separated into a higher service level as discussed in *Chapter 10 – Distribution System Evaluation*.

Table 4-12: Projected Population and Demands at Buildout of the UGB by Pressure Zone

Pressure Zone	Buildout Population	ADD (MGD)	MDD (MGD)
1	97,000	20.81	37.0
2	8,400	0.79	1.5
3	2,700	0.59	1.1
4	900	0.21	0.4
Total	***109,000***	***22.4***	***40.0***

The pace that demands will increase (system wide and by pressure zone) will be influenced by population growth and changes in land uses within each zone.

Figure 4-6: Projected Water Demand Curve



CHAPTER 5 – PLANNING CRITERIA

INTRODUCTION

This chapter presents planning criteria used to evaluate and plan Albany's water system. These criteria were developed to provide planning level guidelines; they are not intended to be rigid requirements. Planning criteria were used as the basis for evaluation in field inspections, modeling, operational tests and review of historic data.

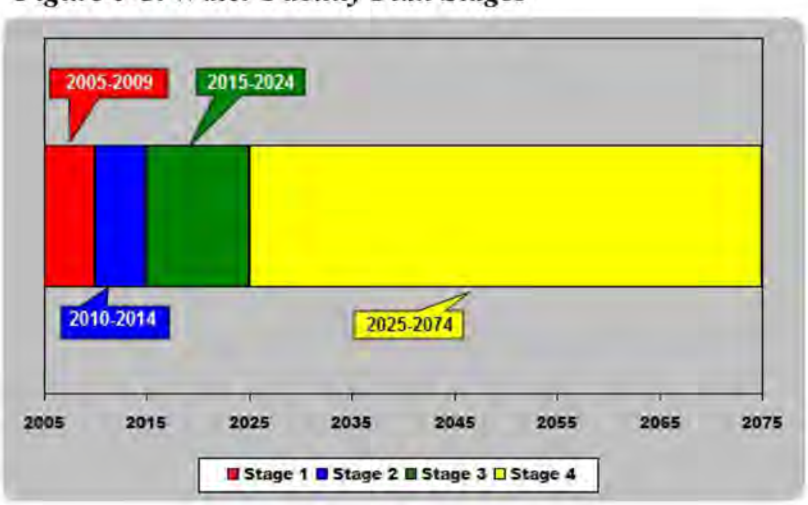
PLANNING PERIOD

This water facility plan considers two planning horizons. The first horizon extends to 2025 and was used to determine improvements to the Vine Street Water Treatment Plant. The second extends to "buildout" of the Urban Growth Boundary (UGB) and was used to determine improvements to pipelines, pump stations, and reservoirs and to determine the capacity requirements for treatment facilities. Two horizons were necessary since the service life of waterlines, storage reservoirs, and pump stations exceed the 20-year planning window used for treatment facilities. In order to help prioritize the recommended improvement projects the planning horizons have been divided into four stages as outlined below, and as shown in ***Figure 5-1***.

- Stage 1: 2005 – 2009
- Stage 2: 2010 – 2014
- Stage 3: 2015 – 2024
- Stage 4: 2025 - 2074

Recommended projects are prioritized into these stages based on projected water demands presented in *Chapter 4 – Population and Water Demands,* and planning criteria identified in this chapter. Timing for construction of each project should be based on actual water demands. For example, it is impossible to predict when development driven projects shown in Stage 4 will actually be needed. Timing for those projects is entirely development dependent and it is important that the plan have flexibility to respond to development as it occurs.

Figure 5-1: Water Facility Plan Stages



PLANNING AREA

Albany's water service area currently includes the Cities of Albany and Millersburg, and limited service to areas outside Albany's city limits. As discussed in *Chapter 9 – Joint Water Project*, the Cities of Millersburg and Albany are currently working on a joint water project that will serve both communities. The joint facility is expected to be brought online in January 2006. Once this project is complete, Millersburg's distribution system will be independent of Albany's. Consequently, although Millersburg is part of the existing service area, this facility plan focuses entirely on Albany's water demands and water system. The service area considered in this water facility plan includes the area within Albany's UGB and limited service outside the UGB in North Albany, as shown in ***Figure 5-2***.

Limitations for additional water service to areas outside Albany's UGB in North Albany are established by Albany City Council Resolution #3363 (included as ***Appendix H***), and limits service to:

- Residential, non-fire flow service;
- Limit of 1 residential meter per tax lot (tax lots existing on July 1, 1991);
- Service will only be provided to lots adjacent to existing water lines, and
- No service will be provided to any future property development outside the city limits if it adversely impacts existing customers.

As outlined above, Albany agreed only to maintain the current level of service at the time the City took over responsibility for operation and maintenance of the North Albany distribution system from Benton County. Therefore, areas outside the City's UGB in North Albany have not been evaluated for planning criteria but their water demands were considered on the system.

The City has developed and adopted a *Comprehensive Land Use Plan*[31] and an UGB to plan and manage growth and extension of services. The UGB encompasses an area of approximately 13,900 acres that includes the current City incorporated area, 10,200 acres, as shown in ***Figure 5-2***. Some parcels located in the 3,700 acres between the city limits and the UGB (urban fringe) are developed and tracts of undeveloped land within city limits are available for development.

PLANNING CRITERIA

Treatment Facilities

For planning purposes, the total capacity of treatment facilities should be capable of meeting the maximum day demand (MDD) at firm capacity: meaning, with the largest pump, or other key component out of service. Development of future sources of supply or capacity upgrades should occur once the existing plant capacities reach between 90 and 95 percent of MDD.

During a maximum day demand event the demand usage will fluctuate over the course of a day, resulting in a peak hour demand (PHD) and a slack hour demand. The capacity to meet water demands in excess of the MDD is provided by reservoir storage. This storage, known

[31] City of Albany, *Comprehensive Plan.* January 1989.

as “equalization” storage, is generally less expensive to provide than additional treatment and pumping capacity necessary to meet these demands. Equalization storage is discussed in more detail under reservoir planning criteria.

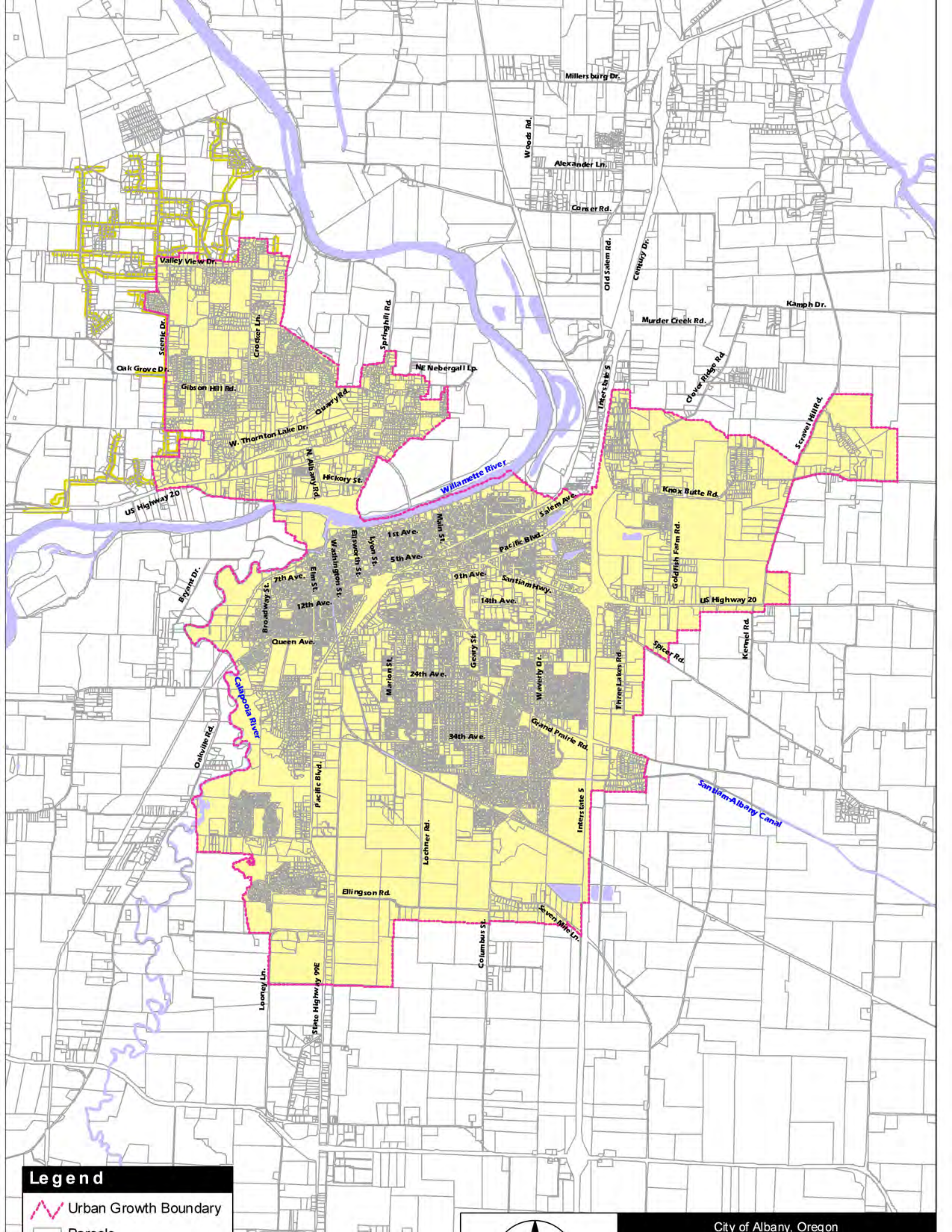
It is important for a city with a single source of supply to have an emergency source in case their primary source becomes temporarily unavailable. An emergency source should be capable of meeting the annual average day demand (ADD), for a maximum of one week (the estimated time to restore the primary supply to at least this capacity). This emergency source could be provided through an intertie with another public water system, or through a secondary source. Currently, the City has a single source of supply and no emergency source. However, the Joint Water Project will provide Albany with a second source of supply. The increase in system reliability resulting from a two-source system reduces the need for an emergency source. Therefore, this plan does not recommend developing an emergency source of supply. Although an emergency source of supply is not recommended, emergency storage, discussed later in this chapter, is required in each pressure zone.

Service Pressure

The American Water Works Association (AWWA) states that, generally, the desired range of system pressures is between 30 and 80 pounds per square inch (psi)[32]. The absolute minimum pressure that must be maintained in the water system is 20 psi (46 ft) per Oregon Drinking Water Program (ODWP) standards. This pressure must be maintained even during a fire flow event during a maximum demand day. This plan recommends maintaining normal operating pressures between 40 and 80 psi. Pressures at a customer’s service tap are directly related to water levels within storage reservoirs. The water level in reservoirs fluctuates during the day as water is being fed into the distribution system and conversely while storage is being replenished. Due to the configuration of existing reservoirs some areas at the upper and lower ends of each pressure zone may experience pressures outside of this range.

Reservoirs should be designed to meet the 40 psi minimum system pressure criteria when they are three-fourths full or 10 feet below the overflow water level, whichever is higher. This criterion was selected to represent a reservoir during normal operating conditions at the end of a peak event. Designing for this condition ensures areas in the higher elevations of each pressure zone will have adequate pressure at lower reservoir water levels. This reservoir condition has also been selected because it represents high flow rate conditions and correspondingly high-pressure head losses while replenishing reservoir storage.

[32] Mays, Larry (in association with AWWA), American Water Works Association (AWWA) Water Distribution System Handbook. McGraw-Hill, 2000



Millersburg Dr.
Woods Rd.
Alexander Ln.
Conser Rd.
Old Salem Rd.
Century Dr.
Kamph Dr.
Murder Creek Rd.
Valley View Dr.
Scenic Dr.
Crocker Ln.
Springhill Rd.
NE Nebergall Lp.
Oak Grove Dr.
Gibson Hill Rd.
Quarry Rd.
W. Thornton Lake Dr.
N. Albany Rd.
Hickory St.
Willamette River
US Highway 20
Interstate 5
Clover Ridge Rd.
Scravel Hill Rd.
Knox Butte Rd.
Salem Ave.
Main St.
1st Ave.
Lyon St.
Ellsworth St.
Washington St.
5th Ave.
Pacific Blvd.
Elm St.
7th Ave.
Bryant Dr.
Broadway St.
12th Ave.
9th Ave.
Santiam Hwy.
14th Ave.
Goldfish Farm Rd.
US Highway 20
Queen Ave.
Spicer Rd.
Kennel Rd.
Geary St.
Marion St.
24th Ave.
Waverly Dr.
Three Lakes Rd.
Calapooia River
Oakville Rd.
34th Ave.
Grand Prairie Rd.
Santiam-Albany Canal
Pacific Blvd.
Lochner Rd.
Interstate 5
Ellingson Rd.
Seven Mile Ln.
Columbus St.
Looney Ln.
State Highway 99E
Legend
Urban Growth Boundary
Parcels
City of Albany, Oregon

Pipelines

According to the AWWA M-32 *Distribution Network Analysis for Water Utilities*, pipe segments are considered 'potentially deficient' if they violate the conditions presented in ***Table 5-1***.

Table 5-1: Maximum Pipeline Velocities and Head Loss Criteria

Pipeline Type	*Maximum Velocity* (*feet per second, fps*)	*Maximum Head Loss* (*feet per 1,000 feet*)
Transmission Pipelines (Pipes greater than or equal to 16" in diameter)	5	3
Distribution Pipelines (Pipes less than 16" in diameter)	10	10

For planning purposes, transmission pipelines should meet MDD with a maximum velocity of 5 feet per second (fps) and maximum head losses of 3 feet per 1,000 feet. Transmission pipelines are considered pipelines greater than or equal to 16 inches in diameter. These pipes convey large volumes of water to reservoirs, high demand users, and feed distribution mains. According to AWWA M-32, as velocities in a transmission pipeline exceed 5 fps, these velocities may inhibit the ability of the system to provide adequate pressure to customers drawing water at the extreme ends of the system. Also, smaller head losses are necessary for transmission pipes because they generally convey water longer distances.

PHD and MDD plus fire flows are generally not used to size transmission pipelines unless they are in a pressure zone served only by pumped storage. During peak hour demand and fire flow events reservoir storage will provide the pressure and volume needed to meet fire flow and PHD. The transmission pipeline from the source or pump station will supply the MDD and the transmission pipeline from the storage reservoir will supply the remaining flow to meet the demand requirement. This remaining flow will be either the fire flow demand or the difference in demand between PHD and MDD. Therefore, the transmission pipeline should not experience flow rates in excess of MDD.

Pipes less than 16 inch in diameter but greater than or equal to 4 inch diameter were evaluated as distribution pipes. Distribution pipes are fed by interconnected transmission lines and provide service to water users throughout the distribution system. As water is being delivered, smaller diameter pipelines will experience larger fluctuations in velocity than transmission pipelines. Unlike transmission pipelines that are sized to convey more uniform flow rates, smaller diameter pipelines must convey peak hour or MDD plus fire flow demands. Therefore, this plan recommends that distribution pipelines meet the larger of peak hour demand or maximum day demand plus fire flow, with a maximum velocity of 10 fps and maximum head losses of 10 feet per 1,000 feet.

The minimum pipeline diameter for new and replacement distribution pipelines should be 8 inches in single-family residential areas and 12 inches in industrial, manufacturing, commercial, and multi-family areas. In general, lines less than 8 and 12 inches in diameter are considered to be inadequate to meet fire flow requirements for the lowest fire demand in the specified areas. However, smaller distribution mains serving residential dead-end streets with no more that 12 residences may be acceptable in some instances. Any existing pipelines less than the recommended minimum should be upgraded before being equipped with a fire hydrant.

Water Loss

The City has established a long-term goal of reducing the current water loss rate to 15 percent. This allowance is included as part of the projected water demand requirement at buildout of the UGB. Water loss is discussed in detail in *Chapter 4 - Population and Water Demands*.

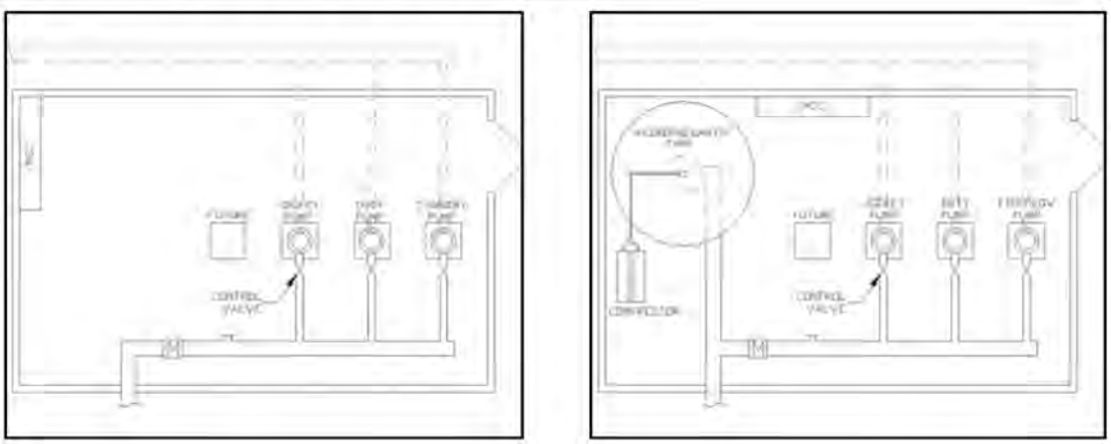
Pressure Reducing Valves

Water system pressure reducing valves on public water systems should supply the PHD within the continuous flow rating of the valve. Fire flows through a pressure reducing valve should be delivered within the intermittent flow rating of the valve.

Pump Stations

Pump stations serving areas with reservoirs should be sized for a firm capacity equal to MDD. Pump stations serving areas without reservoirs should be sized for a firm capacity equal to the higher of peak hour demand or maximum day demand plus the required fire flow. Firm capacity is defined as the capacity of the pump station with the largest pump out of service. ***Figure 5-3*** shows typical pump station layouts.

Figure 5-3: Typical Pump Station Layout



Pump Station with Reservoir Storage ***Pump Station without Reservoir Storage***

For reliability, pump stations should have either two sources of primary power fed from two separate power sub-stations, one main power source and a standby emergency generator, or capability to operate from an emergency power connection. The standby emergency power supply should be sized so that pumping capacity is equal to the flow criteria used to size the pump station.

Pumped storage requires that the water volume stored must be pumped to the distribution system and therefore the storage is not available during a pump station outage. This plan recommends that pumped storage not be considered available during an emergency condition unless emergency backup power is available. Recommendations for continued reliance on pumped storage are included in *Chapter 10 - Distribution System Evaluation*.

Storage

Water storage requirements vary by community but are generally provided for three purposes:

1. Equalization storage; should be included at all reservoirs to meet peak demands within its area of influence
2. Fire storage; should be included at each reservoir to meet the greatest fire flow demand within its area of influence, and
3. Emergency storage; can be stored at one reservoir or distributed between several reservoirs as long as the total storage meets the emergency demands within its pressure zone.

The total storage required in any reservoir is the sum of these components. It is important to keep these components efficiently balanced within each zone. The components of storage and the requirements for Albany's water system are discussed in the following paragraphs.

Equalization Storage

Equalization storage is needed in a water system to meet instantaneous water system demands in excess of the transmission/pumping delivery capacity from the supply source to the system. The volume of required equalization storage is a function of supply system capacity, transmission piping capacity between treatment plant(s), reservoirs and pump stations, and system demand characteristics. Typical equalization storage volumes range from 20 to 30 percent of the MDD for each pressure zone. Equalization storage for this plan is based on 25 percent of the MDD.

Fire Storage

Fire storage is provided to meet the single most severe fire flow demand within the pressure zone served by the storage facility. The fire storage volume required in a pressure zone is determined by multiplying the greatest fire flow rate by its required duration. In pressure zones served by more than one reservoir, this storage should be balanced between reservoirs to provide the most hydraulically efficient storage delivery.

Fire flow requirements are established by assessing a building or structure's vulnerability based upon structure type, exposure, and occupancy. Three methods are typically utilized to establish fire flow requirements: the Insurance Services Office (ISO) Method, the Iowa State University Method, and the Illinois Institute of Technology Research Institute Method.

AWWA M-31 *Distribution System Requirements for Fire Protection* describes each method and the information required to establish fire flow requirements. The closer a water utility comes to satisfying the fire flow requirements of any one of the methods described in AWWA M31, the better the ISO rating will be. ISO ratings are used to establish fire insurance premiums throughout the water service area.

The methodologies mentioned above for determining fire flow requirements are based on individual characteristics of a building including separation from other structures, building material and on-site fire protection facilities. For planning purposes a fire flow rate by land use classification has been used in this plan. Land use classifications were determined based on land use zoning established by the City of Albany and are summarized in ***Table 5-2***. These fire flow requirements were used to determine fire storage volumes and were used in the hydraulic model to evaluate pipeline and pump station hydraulic capabilities under fire flow conditions. Fire flows for commercial and industrial land uses were applied only in those areas where such zoning occurs. Fire flows for residential uses were applied throughout the distribution system.

Emergency Storage

The purpose of emergency storage is to provide water during emergencies such as power outages, equipment failures, pipelines failures, natural disasters, or loss of source of supply. The amount of emergency storage provided is dependent upon an assessment of risk and the desired degree of system reliability. Its primary function is to provide water until an emergency or alternate supply can be activated (if available).

For a water system like Albany's with a single source of supply, typical emergency storage volumes range from two to three average day demands. However, the completion of the Joint Water Project will provide a secondary source of supply and increased system reliability. With a two-source system the possibility of being solely reliant on emergency storage is significantly less than with a single source system. Therefore, enough storage to supply one average day demand has been used as the emergency storage component for this water facility plan.

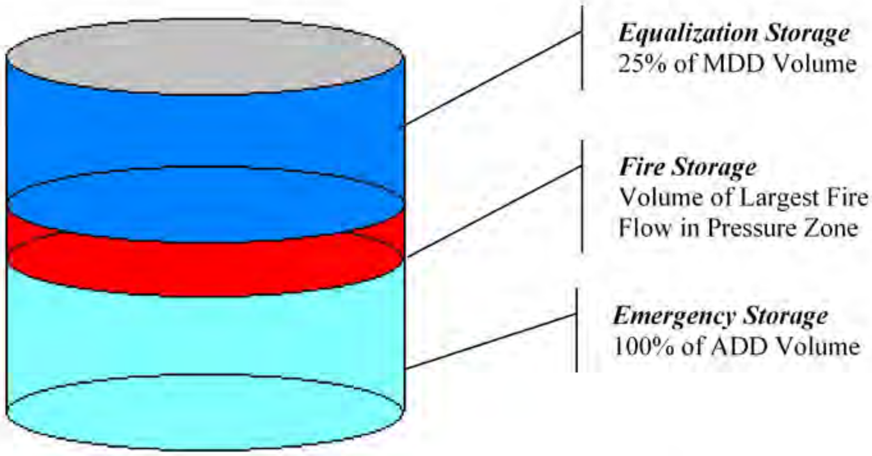
Storage Summary

In summary, storage volume standards for each pressure zone utilized in this plan consist of:

1. Equalization Storage: 25% of projected MDD
2. Fire Flow Storage: Volume needed to meet the largest fire flow demand within a pressure zone.
3. Emergency Storage: One average day demand
4. Storage should be balanced within each pressure zone to provide the most efficient delivery.

Figure 5-4 summarizes planning criteria used to evaluate storage requirements in this plan.

Figure 5-4: Reservoir Storage Schematic



Required Storage = .25(MDD) + Fire Storage + 1(ADD)

SUMMARY

The planning criterion used to evaluate and size recommended improvement projects is summarized in ***Table 5-2***.

Table 5-2: Planning Criteria

Criteria		*Criteria*	*Notes*
Service Area		Albany UGB + NA CSD	No fire svc to NA CSD
Planning Period			
WTP Facilities		2025	
Trans./Dist./Storage/WTP Capacity		Buildout	
Phasing	Phase 1	2005-2009	
	Phase 2	2010-2014	
	Phase 3	2015-2024	
	Phase 4	2025 to Buildout	
Water loss rate @ BO			
Loss Rate		15	
WTP Capacity			
Capacity		MDD	
Distribution System			
Size	Distribution	8"-<16"	Use standard pipe diameters
	Transmission	= and >16"	8,12,16,20,24,30,36,42,48
Max Velocity	Distribution	10 fps	
	Transmission	5 fps	
Headloss	Distribution	10'/1,000'	
	Transmission	3'/1,000'	
Pressure	Zone	1st/2nd/3rd/4th	
	Min. operating	40	
	Max. operating	80	
	Fire Min.	20	
Reservoirs			
Volume	Equalization	25% MDD	
	Fire	Largest fire demand in zone	Balance storage for most hydraulically efficient transmission lines
	Emergency	1 ADD	
Operating Level		75% or within 10' of overflow, whichever is higher	
Pump Stations			
Capacity	Serving Zone w/ Reservoir Storage	MDD	
	Serving Zone w/o Reservoir Storage	Greater of MDD+FF or PHD	
Power		2 sources (sub-stations) or 1 source & generator	
PRV			
Capacity	Continuous	PHD	
	Intermittent	MDD + Fire Flow	
Emergency Supply Source			
Source		Not Required	Scravel Hill WTP
Fire Flows/Durations			
Residential	Low density	1,500 gpm	2 hours
	Medium density (SF & MF)	2,500 gpm	2 hours
	High density (MF)	3,500 gpm	3 hours
Commercial	Office Professional	3,500 gpm	3 hours
	Neighborhood	3,500 gpm	3 hours
	Community/Heavy	3,500 gpm	3 hours
	Tourist services	3,500 gpm	3 hours
Industrial	Park	5,000 gpm	4 hours
	Light	5,000 gpm	4 hours
	Heavy	5,000 gpm	4 hours
Other	Mixed use	3,500 gpm	3 hours
	Schools	5,000 gpm	4 hours
	Institutional (hospital/Jail)	3,500 gpm	3 hours

CHAPTER 6 – WATER SYSTEM REGULATORY REVIEW

INTRODUCTION

This section provides a summary of key regulations that govern operation of the City of Albany's (City) water system. A summary of compliance issues associated with anticipated future regulations is also included. This regulatory summary is current as of January 2003. A more detailed regulatory and historical compliance review can be found in ***Appendix C*** of this facility plan. Only regulations that pertain to Albany's water supply (filtered surface water) are included in this review.

CURRENT DRINKING WATER QUALITY REGULATIONS —OAR 333-061

All water systems must operate in compliance with the federal Safe Drinking Water Act (SDWA). In Oregon, the SDWA is administered by the Oregon Department of Human Services Drinking Water Program (ODWP) under the Oregon Drinking Water Quality Act. There are currently drinking water quality standards for 96 primary contaminants (those with heath concerns) and 12 secondary contaminants (those with only aesthetic impacts) regulated by the ODWP. Each contaminant has either an associated established maximum contaminant level (MCL – a measurable maximum amount of the contaminant that may be present), or a recommended treatment technique (a requirement to install and operate a certain type of water treatment process). These contaminants are grouped into six general categories, and are discussed individually in the following subsections.

- Microbial Contaminants,
- Disinfectants and Disinfection By-Products,
- Inorganic Chemicals,
- Organic Chemicals,
- Radiologic Contaminants, and
- Unregulated Contaminants.

Microbial Contaminants

Surface water systems are protected from microbial contamination through a combination of treatment plant performance and distribution system monitoring. Turbidity removal and microorganism inactivation must be assured at the treatment plant. The absence of coliform bacteria is assured through the presence of chlorine residuals in the distribution system.

Turbidity Monitoring Requirements

Regulatory requirements for filtered water turbidity were recently modified. Prior to January 1, 2002, the regulations required a *combined* filter effluent turbidity less than 0.5 nephelometric turbidity units (NTU, a measure of turbidity based on visible light scatter through a stream of water), in 95 percent of the measurements, never to exceed 5.0 NTU. In January 2002, filtered water turbidity requirements were revised and a more stringent standard adopted. Under this new standard water turbidity is required to be less than 0.3 NTU in 95 percent of the measurements, never to exceed 1.0 NTU. These new requirements apply to each individual filter in addition to the combined filter effluent. The City's Vine Street WaterTreatment Plant (WTP) has historically met or exceeded the regulatory requirements for filtered water turbidity. The plant has had no difficulties in meeting the new, lower turbidity requirements. Instrumentation improvements to ensure continued compliance with these more stringent turbidity regulatory requirements are included as recommended improvements discussed in *Chapter 8 – Vine Street Water Treatment Plant.*

Disinfection Performance

Disinfection performance requirements to inactivate *Giardia*, viruses and bacteria that may be present in the raw water supply to the WTP, measured as a level of inactivation, have consistently been met or exceeded at the Vine Street WTP based on the monitoring data. Free chlorine has been employed as the primary disinfectant to achieve this purpose.

In order to determine the level of microbial inactivation that is achieved during disinfection with chlorine, the EPA developed the "CT" concept. "CT" is the product of disinfectant residual concentration (or "C"), measured at the outlet of a disinfection section, and the available reaction time (or "T"). A review of the available historical data in this facility plan led to recommendations for minor adjustments to the methodology used for plant's "CT" calculation that more accurately represents disinfection performance at the Vine Street WTP. A discussion of CT calculations and the recommended adjustments are included in ***Appendix C***.

A tracer test was recommended and performed on the Maple Street Reservoir to determine the appropriate time value (known as T_{10}/T), for use in the "CT" calculation. Results from the tracer tests over a range of treatment plant flows and reservoir levels suggest the reservoir's interior baffle wall and inlet piping need to be repaired or replaced to maximize contact time through the reservoir. This is particularly important if Albany decides to delay chlorine addition until after clarification to reduce disinfection by-products. Recommended improvements to the Maple Street Reservoir and baffle are discussed in *Chapter 8 – Vine Street Water Treatment Plant*.

Historic distribution system monitoring data indicates consistent compliance with the ODWP requirements for levels of coliform bacteria. However, two coliform sampling violations are on record at the ODWP, dated March 1, 1996, and August 1, 1996. In both cases, the violations corresponded to an inadequate number of samples submitted to the state. No violations with regard to coliform presence in drinking water are on record.

Free chlorine is also used to maintain a residual in the distribution system. The Albany water system data shows consistent compliance with disinfection residual monitoring and disinfection residual concentrations in the distribution system.

Disinfectants and Disinfection By-Products (DBPs)

TTHM and HAA_5

Disinfection treatments used to inactivate pathogens in drinking water can react with naturally occurring organics, (total organic carbon, TOC) and inorganic matter in water to form disinfection byproducts (DBPs). The type and concentration of DBPs in treated water depends largely on the type of disinfectant and concentrations of DBP precursor material.

For Albany, the two most important classes of DBPs are total trihalomethanes (TTHMs), that have been regulated for a number of years and five haloacetic acids (HAA_5), for which regulations are more recent.

Albany's data indicates that total trihalomethanes (TTHM) have consistently been below the MCL. Quarterly average TTHM concentrations have ranged from 0.022 to 0.074 mg/L, and have consistently been lower than the allowable MCLs (0.10 mg/L, prior to January 2002).

In January 2002, new regulations for disinfection byproducts came into effect. The TTHM MCL dropped to 0.080 mg/L and a new regulation for five HAAs, (called HAA_5) was set at 0.060 mg/L. If monitoring results are within 80 percent of these levels, additional studies (called disinfection profiling) are required. Recent quarterly samples for disinfection by-products taken over a two-year period, partly in anticipation of these regulations, average 0.034 mg/L and 0.042 mg/L for TTHM and HAA_5, respectively. Though TTHM levels are well below any action levels, HAA_5 concentrations are approaching the disinfection profiling action level.

As a result, studies intended to better understand HAA_5 formation at the water treatment plant were performed. These studies concluded that much of the material that ultimately forms HAAs upon chlorination is removed in the clarification step of the treatment process. Based on these studies, the City has decided to relocate the point of chlorine addition downstream of the clarifiers. By relocating the point of chlorine addition later in the treatment process, the Maple Street reservoir will need to provide almost all of the chlorine contact time required for disinfection to meet Surface Water Treatment Rule requirements for *Giardia* inactivation, making the previously recommended improvements to the interior baffle wall a high priority. Because the Maple Street Reservoir cannot provide adequate contact time during periods of high demand, the point of chlorine addition will continue to be ahead of the clarifiers during these periods until baffle and inlet/outlet piping improvements are completed.

Total Organic Carbon (TOC)

Another measure of the overall potential for disinfection byproduct formation is the level of TOC in the water. Historical TOC levels in the Santiam-Albany Canal have been relatively low (<2.0 mg/L). However, recent sampling indicates that TOC levels in the raw water are, at times, at or above the 2.0 mg/L, the "trigger" level that can require enhanced coagulation for optimal TOC removal. The percent of TOC removal required at the plant depends on raw water TOC and alkalinity. For Albany, TOC removal requirements are not expected to exceed 35-percent (see *Chapter 8* - *Vine Street Water Treatment Plant,* for results from recent sampling efforts, as well as plant TOC removal).

TOC removal efficiencies and their relationship to alkalinity and regulatory requirements are discussed in detail in ***Appendix C***. To ensure ongoing compliance with expected TOC requirements, we recommend Albany purchase a bench-top ultraviolet (UV) spectrophotometer, and incorporate daily UV absorbance (at 254 nm) monitoring at the Vine Street WTP as a surrogate for TOC. UV_{254} sampling is relatively inexpensive, simple and accurate. This purchase is included as a recommended improvement as discussed in *Chapter 8 – Vine Street Water Treatment Plant*.

Inorganic Chemicals

Sixteen inorganic compounds are currently regulated in OAR 333-061. The Vine Street WTP has consistently complied with all inorganic chemical MCLs. A few of these contaminants merit discussion for long-term compliance. This discussion is summarized below and more detailed information is included in ***Appendix C***.

Nitrate/Nitrite

Albany's nitrate/nitrite concentrations in treated water averages 0.14 mg/L-N, with a maximum of 0.38 mg/L-N recorded on October 29, 1996. The MCL is 1.0 mg/L-N. Nitrate/Nitrite can result from introduction of agricultural runoff into the Canal. The current treatment plant process will not readily remove this contaminant. Therefore, ongoing monitoring of the Canal is important.

Asbestos

Approximately 96 miles or 44 percent of Albany's distribution system (4-inch diameter and larger) is asbestos-cement (AC) pipelines. In some water systems, these pipes have released asbestos fibers into the drinking water. The MCL of asbestos fibers is 7 million fibers per liter (MF/L). The most recent samples for asbestos fibers from Albany's distribution system were taken on August 26, 2002, in accordance with regulatory requirements, and reported 0.34 MF/L.

Though historic concentrations of asbestos are either absent or low, sampling for asbestos fibers should be continued at a frequency of every 3 years to assure any changes in water quality do not influence the release of asbestos fibers.

Lead and Copper

Another key concern in water systems is the leaching of lead and copper from home plumbing systems into the water at the tap. Initial lead and copper sampling from home plumbing systems began in Albany in the fall of 1992 in response to regulations for these contaminants (called the lead and copper rule, or LCR), and periodic sampling has occurred since then. This rule is a treatment technique rule, with a specified action level that triggers certain responses from the utility if exceeded. The action level for lead has been established at 0.015 mg/L in 90 percent of the samples, while the action level for copper is 1.3 mg/L in 90 percent of the samples.

All test results for lead and copper have been below the above referenced action levels. Through treatment process optimization at the Vine Street WTP, lead and copper concentrations in the distribution system and household plumbing have steadily decreased since the adoption of the LCR. The most recent measurements, taken on September 27, 2002, report that all of the 36 samples collected are below action levels, with 90 percent of the test results below 0.0043 mg/L for lead and 0.384 mg/L for copper.,

Organic Chemicals

The Vine Street WTP has consistently complied with all organic chemical MCLs. No concentrations of regulated volatile organic carbons (VOCs) or soluble organic carbons (SOCs) above the detection limit are on record. Detailed information concerning regulated organic chemicals MCLs, and testing results is included in ***Appendix C***.

The Vine Street WTP does not have continuous protection against organic contaminants that may be present in the raw water. The WTP is equipped with powdered activated carbon (PAC) that can be used to remove organic contaminants, but the successful implementation of this process requires detection and intervention by the WTP staff. The Canal source is vulnerable to contamination from agricultural and urban runoff that may contain some organic contaminants. Future replacement of current anthracite filter media with granular activated carbon (GAC) will provide continuous protection from organic contaminants and thereby enhance treatment reliability and safety. This is recommended in the long-term to enhance the current level of protection and reduce contamination risks.

Radiologic Contaminants

Radiologic samples were taken on October 22, 1992, October 29, 1996, and December 08, 2000. The six regulated contaminants are listed in ***Appendix C*** and none were detected on the above testing dates. Albany has fully complied with all ODWP standards with regard to radiologic contaminants.

Unregulated Contaminants

The federal Environmental Protection Agency (EPA) and the state periodically require water systems to conduct monitoring for contaminants that are not regulated. The purpose of this monitoring is to gather data about these contaminants for potential future regulations. The City of Albany has remained in compliance with unregulated contaminant monitoring requirements. Additional detail concerning unregulated contaminants is discussed in ***Appendix C***.

FUTURE DRINKING WATER REGULATIONS

A number of EPA rules that could impact the operation of the City of Albany's water system are currently pending or in development. Estimates of the timetables for promulgation for these rules, and the projected effects on the City of Albany are presented below. These pending and future regulations include:

- Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)
- Stage 2 Disinfection By-product Rule (DBPR)
- Radon Rule
- Arsenic Rule
- Unregulated Contaminant Monitoring

Long Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

The purpose of the LT2ESWTR is to improve the control of microbial pathogens in drinking water, especially *Cryptosporidium*, in public water systems serving more than 10,000 people. A group of interested and affected public and private people chartered by the EPA, developed an Agreement in Principle that outlines this rule. The Agreement and proposed regulations have been circulated by EPA for review and comment. This Agreement and draft language for the LT2ESWTR requires increased filtration and disinfection performance criteria based on system vulnerability, specific *Cryptosporidium* inactivation, incorporation of a multi-barrier disinfection strategy, and demonstration of low *Cryptosporidium* levels in treated (i.e. filtered) water. Promulgation of the rule has been delayed as EPA has grappled with the implications of the events of September 11, 2001, on water system security. The rule is now anticipated to come into effect in 2004, with compliance beginning in 2006.

To better prepare for the LT2ESWTR, incorporation of *Giardia* and *Cryptosporidium* sampling in the raw water monitoring program is recommended, as levels of these microorganisms in the raw water will determine the amount of treatment that is required. At the moment, there is no reason to believe that additional treatment process steps will be needed. However, this can be confirmed with the additional sampling. If Albany is required to add treatment to inactivate for *Cryptosporidium* in the future, installation of a disinfectant stronger than chlorine (e.g. ultraviolet (UV) irradiation) may be necessary at the Vine Street Water Treatment Plant, as chlorine is a relatively ineffective disinfectant for *Cryptosporidium*. The proposed Scravel Hill Water Treatment Plant, discussed in *Chapter 9 - Joint Water Project*, will use membrane filtration as a method of treatment and is expected to exceed *Cryptosporidium* treatment requirements through a 4-log removal performance.

Additionally, the installation of particle counters on the individual filter effluent lines is recommended to better understand the removal of pathogenic organisms through the Vine Street WTP, and improve process control. The draft rule focuses on individual filter performance, instead of combined filter effluent.

Stage 2 Disinfection By-product (DBPR)

A companion rule to the LT2ESWTR is the Stage 2 DBPR. The purpose of the Stage 2 DBPR is to further reduce health risks associated with disinfection by-products. The same EPA chartered group also developed an Agreement in Principle that outlines this rule and the EPA has circulated draft language for review. This Agreement and draft language for the rule change the method for determining compliance with the MCLs of 0.080 mg/L for TTHMs and 0.060 mg/L for HAA_5s. Instead of calculating compliance based on a running annual average of samples throughout the distribution system, compliance will be based on the results of samples at each individual point around the distribution system. This method of sampling is called a Locational Running Annual Average (LRAA). Locations for monitoring compliance with the rule must be selected through an Initial Distribution System Evaluation (IDSE) that will be the first step in the compliance process. Compliance with this new rule is expected concurrent with compliance with the LT2ESWTR.

Based on available data, moving the point of chlorine addition in the Vine Street WTP to just ahead of the filters should be sufficient for continued compliance when the Stage 2 DBPR is adopted. Results of ongoing monitoring for DBPs should be reviewed as this rule is developed, however, to assure that no additional steps will be needed to comply.

Arsenic Rule

EPA recently lowered the MCL for arsenic to 0.010 mg/L from 0.050 mg/L, compliance with this new MCL is required by January 2006. The Vine Street WTP is not expected to have problems meeting this rule. The most recent testing for arsenic was completed on July 23, 2002, and arsenic was not detected with a detection limit of 0.002 mg/L.

Radon Rule

EPA has attempted to develop new rules for radon for a number of years. This contaminant is a naturally present radionuclide that can be present in some groundwater systems. The regulation has previously been proposed, but the final promulgation has been delayed a number of times for a variety of reasons. As a surface water supply, the Santiam River and the Canal do not have radon, consequently compliance issues are not anticipated for Albany when and if it is promulgated.

Unregulated Contaminants

A new round of quarterly sampling for EPA unregulated contaminants is required for at least one year, between 2001 and 2003. Additional future monitoring will likely be required; Albany may need to increase sampling frequency to accommodate the ongoing, federally run program as requirements are published.

MASTER PLAN REQUIREMENTS—OAR 333-061-0060 (5)

Oregon communities with 300 or more service connections are required to maintain a current water master plan under the ODWP. These plans must be prepared by a professional engineer, and must consider the needs of the water system for at least a twenty-year period. Upon completion, the plan must be submitted to the Health Department for review and approval. Each master plan must include the following elements:

- A summary of the entire plan, including water quality and service goals, present and future water system deficiencies, recommended alternatives for achieving goals and repairing deficiencies, and a recommended implementation schedule and financing program for constructing improvements.
- A description of the existing water system, including source(s) of supply, status of water rights, current status of drinking water quality and compliance with regulatory standards. In addition, the plan is required to present maps indicating size/location of existing facilities, estimates of water use and operational and maintenance requirements.
- A description of the water quality and level of service goals for the water system, including existing and future regulatory requirements, additional water quality needs, flow and pressure requirements, and capacity needs related to water use and fire flow needs.
- An estimate of the projected growth of the water system during the master plan period and the impacts on the service area boundaries, water supply source(s) and availability, and customer water use.
- An evaluation of the ability of the existing water system facilities to meet the water quality and level of service goals.
- Identification of alternative engineering solutions, environmental impacts and associated capital and operation and maintenance costs, to correct water system deficiencies and achieve system expansion to meet anticipated growth.

- A description of alternatives to finance water system improvement programs including the recommended engineering alternative and associated costs, maps or schematics showing size and location of proposed facilities, the recommended financing alternative and a recommended schedule for water system design and construction.

- The plan should also address the water management and conservation plan requirements, if applicable.[33]

This water facility plan and companion water financial and water management plans now in progress satisfy these requirements. These documents should be periodically updated as changes occur in the water system and the community, to reflect up-to-date needs and solutions to water system issues. Updates to the water facility plan are discussed as recommended improvements in *Chapter 8 – Vine Street Water Treatment Plant* and *Chapter 12 – Recommended Plan.*

NPDES REQUIREMENTS

In addition to the requirements of the Safe Drinking Water Act, the City of Albany's water system must operate in compliance with the federal Clean Water Act. This Act is administered in Oregon by the Department of Environmental Quality (ODEQ) and establishes wastewater discharge limitations for backwash lagoons at the WTP. The City must ensure discharges from backwash lagoons and settling basins at the Vine Street Treatment Plant comply with conditions of a National Pollutant Discharge Elimination System (NPDES) permit. The purpose of the NPDES permit is to protect water quality of receiving streams. The backwash lagoons and settling basins are currently operating under an NPDES General Permit (200J) dated January 8, 1998. Permits are issued for a five year term and the City submitted a renewal application for this permit on December 11, 2001. The January 8, 1998, 200J permit continues in force pending DEQ's action on the renewal application. Key elements of this permit include:

- Settable solids cannot exceed a concentration of 0.1 mg/L and the daily pH must be in the range of 6.0 to 9.0,

- Filter backwash, settling basin and reservoir cleaning wastewater may be land applied under certain conditions.

- Prior to discharge to any water body, all filter backwash effluent must pass through a settling pond or other approved treatment system and meet the effluent conditions above

The implications of these NPDES permit requirements for the Vine Street WTP are discussed in *Chapter 8 - Vine Street Water Treatment Plant*. Implications for reservoir improvements to meet dechlorination requirements when drained are discussed in *Chapter 10 - Distribution System Evaluation.*

[33] OAR Chapter 333, Division 061—Public Water Systems. January 15, 2002.

PUBLIC HEALTH SECURITY AND BIOTERRORISM PREPAREDNESS AND RESPONSE ACT OF 2002

Drinking water utilities today find themselves facing new responsibilities. While their mission has always been to deliver a dependable and safe supply of water to their customers, the challenges inherent in achieving that mission have expanded to include security and counter-terrorism. In the Public Health Security and Bioterrorism Preparedness and Response Act of 2002, Congress recognized the need for drinking water systems to undertake a more comprehensive view of water safety and security. The Act amends the Safe Drinking Water Act and specifies actions community water systems and the U.S. Environmental Protection Agency (EPA) must take to improve the security of the nation's drinking water infrastructure.

The Bioterrorism Preparedness and Response Act requires every community water system that serves a population greater than 3,300 persons to conduct a vulnerability assessment. The basic elements of a vulnerability assessment are:

- Characterization of the water system, including its mission and objectives,
- Identification and prioritization of adverse consequences to avoid,
- Determination of critical assets that might be subject to malevolent acts,
- Assessment of the likelihood of such malevolent acts,
- Evaluation of existing counter measures,
- Analysis of current risk and development of a prioritized plan to reduce risks,
- Certify and submit a copy of the assessment to the EPA Administrator (by June 30, 2004, for communities serving a population less than 50,000),
- Prepare or revise an emergency response plan that incorporates the results of the vulnerability assessment, and
- Certify to the EPA Administrator, within 6 months of completing the vulnerability assessment, that the system has completed or updated their emergency response plan.

The vulnerability assessment needs to be completed by the end 2004 and the emergency response plan by mid-2005.

SUMMARY AND CONCLUSIONS

Specific recommendations for water system regulatory related improvements are included in *Chapter 8 – Vine Street Water Treatment Plant* and *Chapter 12 – Recommended Plan*.

As noted earlier, this regulatory summary is current as of January 2003. A more detailed regulatory and historical compliance review is included as ***Appendix C***.

CHAPTER 7 - CANAL EVALUATION

INTRODUCTION

As introduced in *Chapter 3 – Existing System Description*, the Santiam-Albany Canal (Canal) is a key component of the City's water supply system, conveying water from the South Santiam River upstream of Lebanon to the Vine Street Water Treatment Plant located in Albany. This chapter provides a brief overview of the Canal's history, a description of its characteristics and current use, and an evaluation of the Canal's existing condition and future needs. ***Figure 7-1*** provides a map of the entire Santiam-Albany Canal and includes the locations of major facilities discussed in this chapter.

History

A canal between Albany and the South Santiam River was first discussed in 1858 when the Legislative Assembly of the Territory of Oregon approved an act to incorporate the Albany Canal and Manufacturing Company[34]. A canal was believed to be a viable way of transporting goods between the Cities of Lebanon and Albany. Construction of a 12-mile long Santiam-Albany Canal routing South Santiam River water (RM 18.1) through the Cities of Lebanon and Albany began in 1872. Once construction was completed logs and grain were transported downstream to Albany industries, but upstream transportation was nearly impossible due to unexpectedly strong current. Commerce being a failure, Canal water was sold to turn water wheels of manufacturing plants and was being used by Albany as a drinking water source by 1880.

In 1891-1892 a six-mile Santiam-Lebanon Canal was constructed to provide the City of Lebanon with drinking water. The Canal traveled from the South Santiam River (RM 20.9), north through Lebanon, and connected with the Santiam-Albany Canal. Connection with the Santiam-Albany Canal became a necessity in 1921 when a major flood destroyed the timber diversion dam for the Albany Canal. From that point on the upper reaches of the Santiam-Albany Canal were abandoned and the headworks for the Santiam-Lebanon Canal became the sole diversion for the canal system. In 1923 the Canals came under one ownership and functioned like the Canal as we know it today. Today, the Canals are referred to solely as the Santiam-Albany Canal (Canal).

Canal Overview and Characteristics

The Canal originates on the South Santiam River 325 feet upstream of the Lebanon Dam located two miles southeast of Lebanon. It travels approximately 18 miles from its point of origin on the South Santiam River, through the Cities of Lebanon and Albany, to its point of termination on the Calapooia River. While traversing its course, it drops approximately 170 feet in elevation. The Canal varies from approximately 20 to 40 feet in width with steep side slopes for the entire length. Its original design capacity is estimated from record drawings to be up to 395 cfs[35]. The Canal has 12 flow control structures along its length that control flows through the various reaches. These structures are discussed later in this chapter.

[34] Oregon Statesman, 19 January 1858:1

[35] Harza Northwest, Inc., Albany Hydroelectric Project Fish Passage Study, Prepared for the City of Albany, July 15, 1994

The Canal provides water for municipal uses in the Cities of Albany and Lebanon, hydropower generation, irrigation for agriculture, and flow augmentation in urban streams. Irrigation withdrawals consist of point withdrawals along the Canal, both permitted as assigned by water rights, and un-permitted.

Currently, the Canal is also used for stormwater conveyance. Albany has begun the process of separating the stormwater from the Canal, and plans to reroute the majority of stormwater entering the Canal over time. The Canal was also a source of hydroelectric power for the Mountain States Power Company, Pacific Power and Light, and the City of Albany between 1892 and 1991. Albany currently holds a Federal Energy and Regulatory Commission (FERC) license authorizing re-commissioning of the hydroelectric project located near the Canal's terminus. The Canal is a licensed component of the hydro facility.

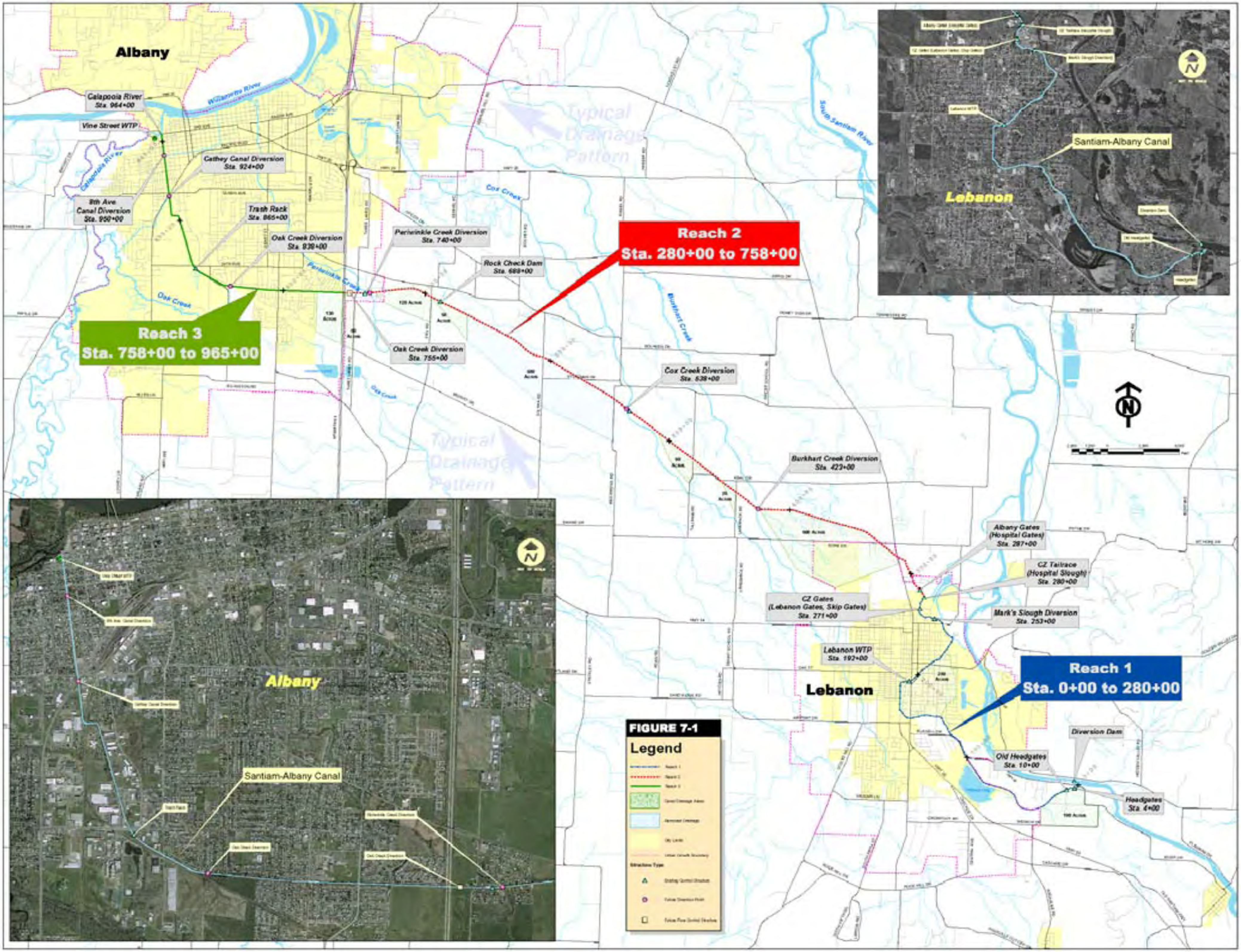
GOALS FOR MANAGEMENT OF THE CANAL

As discussed above, the Canal is relied upon for municipal drinking water, hydropower, irrigation and urban stream flow augmentation. The Canal also serves to convey stormwater runoff, however this use is expected to diminish over time as stormwater is removed from the Canal and historic drainage patterns are reestablished. Albany has identified goals for management of the Canal that are summarized below.

Flow Control

Goals for managing flows along the Canal have been developed in recognition of seasonal flow and use variations. During wet weather the Canal is primarily used to meet municipal and hydroelectric needs and for stormwater conveyance. During dry weather the Canal is primarily used to meet peak municipal needs, and for irrigation and urban stream flow augmentation. Flow control goals include:

- Provide adequate capacity at flow control structures to meet municipal, hydroelectric, irrigation, and stream flow augmentation uses and interim stormwater capacity to reduce the potential for flooding;
- To the extent practical, prevent flooding during wet weather months by routing a portion of peak flows to side channels where sufficient capacity is available;
- Ensure adequate stream flow velocities year round to minimize sedimentation;
- Augment urban stream flows during dry weather periods to improve riparian habitat and stream water quality; and
- Use instrumentation and controls at flow control structures to balance flows.



Albany
Calapooia River
Sta. 964+00
Vine Street WTP
Willamette River
Calapooia River
Cathey Canal Diversion
Sta. 924+00
9th Ave.
Canal Diversion
Sta. 950+00
Trash Rack
Sta. 865+00
Oak Creek Diversion
Sta. 839+00
Periwinkle Creek Diversion
Sta. 740+00
Rock Check Dam
Sta. 688+00
Oak Creek Diversion
Sta. 755+00
Cox Creek
Oak Creek
Periwinkle Creek
Typical Drainage Pattern
Reach 3
Sta. 758+00 to 965+00
Reach 2
Sta. 280+00 to 758+00
Cox Creek Diversion
Sta. 638+00
Burkhart Creek
Burkhart Creek Diversion
Sta. 423+00
South Santiam River
Santiam-Albany Canal
Lebanon
Albany Gates
(Hospital Gates)
Sta. 287+00
CZ Tailrace
(Hospital Slough)
Sta. 280+00
CZ Gates
(Lebanon Gates, Skip Gates)
Sta. 271+00
Mark's Slough Diversion
Sta. 253+00
Lebanon WTP
Sta. 192+00
Reach 1
Sta. 0+00 to 280+00
Diversion Dam
Old Headgates
Sta. 10+00
Headgates
Sta. 4+00
Albany
Santiam-Albany Canal
FIGURE 7-1
Legend

Ensure Canal Capacity

Maintaining the Canal's capacity to meet varying seasonal demands is a fundamental part of Canal management. Goal's for ensuring Canal capacity include:

- Provide adequate capacity at bridge and culvert crossings to meet municipal, hydroelectric, irrigation, and stream flow augmentation uses and interim stormwater capacity to reduce the potential for flooding;
- Provide adequate channel capacity through sediment removal and raising Canal bank elevations where needed;
- Where practical, reroute lateral inflows to reduce the required maximum capacity.

Channel Restoration and Water Quality

The Canal serves as Albany's sole source of drinking water and will continue to be relied upon as a source of drinking water in the future. Consequently, protecting the quality of the source water is an essential element of managing the Canal and the water system. Channel restoration and water quality goals include:

- Where practical, reroute lateral inflows from the Canal to reduce contamination risks and increased turbidity due to rapid changes in channel flows;
- Repair and maintain unstable banks and channel beds to protect the Canal's water quality and minimize turbidity increases from erosion;
- Protect the Canal from potential sources of contamination;
- Remove excessive bank vegetation that impedes flow; and
- Provide and maintain shade where practical to minimize stream temperatures during summer months.

Maintaining Canal Access

To the extent practical, Albany needs legal and physical access along the Canal and sufficient resources to conduct routine inspection, operation and maintenance activities. Goals for maintaining Canal access include:

- Secure legal and physical access along the Canal where practical;
- Remove encroachments in/on Canal right of way; and
- Remove debris and excessive bank vegetation.

CANAL WATER RIGHTS AND SUMMARY OF USES

Canal Water Right Summary

The state has issued a total of 382 cfs in water rights for Canal water (originating from the South Santiam River) to Albany, Lebanon, and agricultural customers along the Canal as shown in ***Table 3-1***. Water rights are issued for the following categories:

- Municipal water for the Cities of Albany and Lebanon,
- Irrigation for agriculture and stock water,

- Augmentation of flows in urban streams for fish and recreation,
- Hydroelectric power generation.

Hydropower Flow Considerations

As noted in *Chapter 3 – Existing System Description*, the City maintains a 275-cfs perfected water right from the South Santiam River for hydropower with a priority date of 1874. Use of these rights, however, has recently been modified by the 1998 FERC hydropower license. The new FERC license limits withdrawal of water used for hydropower generating purposes to 190 cfs. This represents only 69% (190/275 cfs) of the water permitted by the Water Resource Department to be used for hydropower. The City has applied to convert excess hydropower water rights (85 cfs) for use in urban stream flow augmentation during low flow conditions.

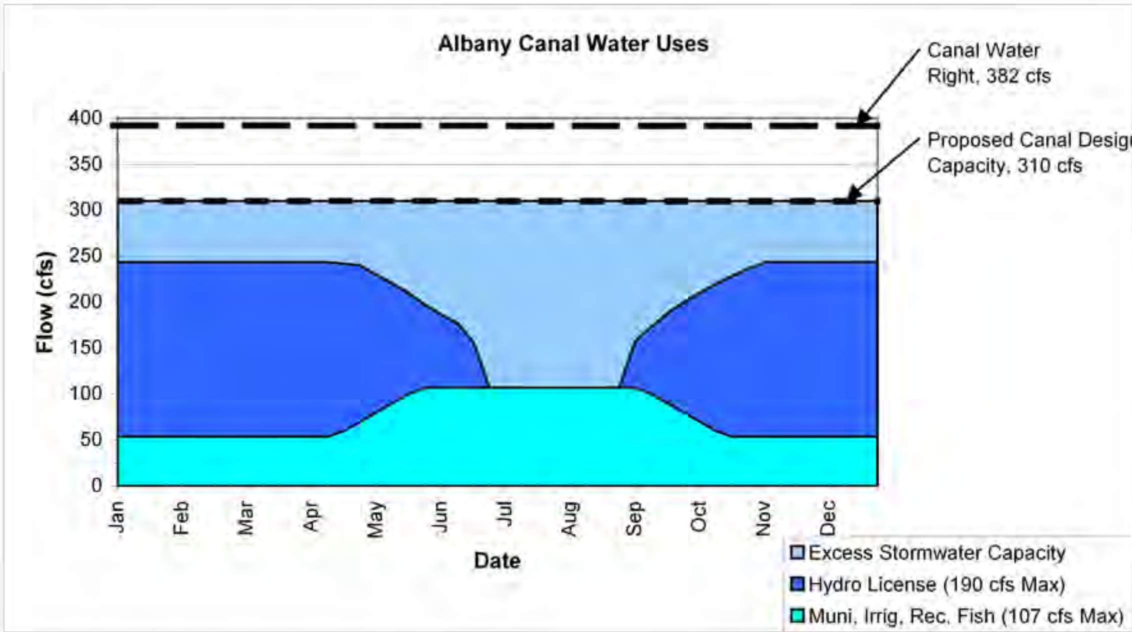
The FERC license also stipulates that the City cannot continue to draw water for hydropower use if the flow in the South Santiam River falls below 1,100 cfs. According to historic hydrology data, the mean daily flow in the South Santiam River is typically less than 1,100 cfs from about mid-June through mid-August. Therefore, hydropower flows are anticipated to drop below the 190 cfs limit, and the plant will likely cease hydropower generation in the dry summer months.

Seasonal Canal Flow Summary

As indicated in the discussion above, there is a "wet" and "dry" seasonal use pattern associated with Canal flows. The wet season typically extends from November to May. The dry season typically runs from July through September, and a transition period spans the gap. The wet season represents the highest flows in the Canal, with hydropower and interim stormwater flows at their highest levels.

Figure 7-2 illustrates typical patterns of Canal flows between the "wet" and "dry" season, as categorized by use. This figure helps to illustrate variations in flow by season, and shows the capacity available on a seasonal basis for interim stormwater conveyance. Data shown is approximate, with the intent of illustrating the varying uses by season allowed through existing water rights and does not depict actual water consumption.

Figure 7-2 Canal Flow Summary



Based on hydropower flow limitations during the summer months, it is unlikely that peak municipal demands will coincide with peak power production needs. ***Figure 7-2*** also illustrates the reduced Canal summer flows, and the fact that higher capacities are only necessary in the winter months.

A 310-cfs Canal design value was utilized throughout the FERC licensing process, and is considered a reasonable design value for Canal analysis in this plan. The 310-cfs value was developed based on concurrent uses of water rights which, as shown in ***Figure 7-2,*** is not likely to occur. However, the difference between typical winter flows shown in ***Figure 7-2*** and the 310-cfs design capacity will allow for an interim stormwater conveyance allowance until lateral inflows are re-routed.

ANALYSIS METHODOLOGY

The goals of the Canal analysis include:

- Identifying areas needing improvement,
- Develop recommendations to address needs, and
- Recommend an approach to quantify and prioritize recommended improvements.

The approach utilized for the Canal evaluation includes:

- Inspect the Canal and develop an inventory of Canal features, structures, and issues;
- Categorize the findings into related groups;

[36] Hydro - Albany Hydroelectric Project Development Cost Analysis Study: A FERC Minor License, FERC Project No. 11509-000, Harza Engineering Company, April 17, 2001.

- Identify and develop solutions for areas needing improvement; and
- Prepare recommendations and a plan to address Canal needs.

Canal Inspection and Inventory

A field inspection was conducted between April 30 and May 18, 2001, in order to assess the existing condition of the Canal. The inspection included walking the length of the Canal with City's operations staff and collecting field data using hand written notes and a GPS location device. Over 95% of the Canal was inspected during the inventory. The remaining 5% (approximately 1 mile) was not inspected due to dense bank vegetation, private fences without access, or other property access concerns. Collected data includes identification and location of all utility, culvert and bridge crossings; flow control structures; Canal access or limited access points; right-of-way (ROW) encroachments; bank damage and excessive bank vegetation; observed sedimentation areas; and all inflow and outflow locations.

Inspection data was compiled into a GIS database. Tabular results (provided in ***Appendix B***) provide an inventory of all flow control features, water quality issues and Canal accessibility issues identified during the Canal inspection. Information for each feature includes: survey record number, nearest station, survey photograph identification number, field notes listing possible flow constraints, source of surface water, potential Canal leakage source, dimensions, etc.

Analysis Procedure

Once the inspection data was compiled into a GIS database, the results were queried to identify points of interest relative to Canal management goals. The following categories were used to analyze Canal needs:

- Flow control,
- Ensure Canal capacity,
- Channel restoration and water quality, and
- Canal accessibility.

Analysis results for each of the categories are provided in the Inspection and Analysis Results section of this chapter.

Flow Control

Flow control features include control gates, weirs, check dams, and other structures that directly influence Canal capacity and flow control. Information was collected for each flow control feature including the type, purpose, operational notes, digital photographs, and sketches illustrating general configuration and dimensions where appropriate. This information is summarized in the GIS database and is included in ***Appendix B***.

In order to identify any potentially flow-limiting Canal features, the structures were examined for their ability to pass a design flow of 310-cfs. As described earlier in this chapter, the 310-cfs flow accounts for irrigation, municipal, and hydropower water rights, and accommodates some surface runoff during the "wet" season. Limited data was available to perform a thorough hydraulic analysis of these structures; however, analyses suitable for a cursory "screening" were performed.

Ensure Canal Capacity

Ensuring Canal capacity involves providing an adequate slope and channel cross section along the entire length of the Canal, ensuring that culvert and bridge crossing are not undersized, and rerouting uncontrolled lateral inflows. Each bridge crossing and Canal culvert was reviewed based on field data and the open channel analysis programs, Heastead's "FlowMaster" and "CulvertMaster"[37]. As with the flow control structures, limited data was available to perform a thorough hydraulic analysis; however, analyses suitable for a cursory "screening" were performed.

Sediment removal quantities to provide a Canal capacity of 310-cfs were based on previous studies, including a review of the *1997 Draft Erosion and Sediment Control Plan*[38], and the *2000 Hydroelectric Project Cost Analysis Study*[39].

Channel Restoration and Water Quality

Field inspection results were queried in order to identify channel restoration needs and subsequent water quality concerns. Locations of lateral inflows, groundwater seepage, local erosion and sedimentation, bank damage, debris, and excessive bank vegetation were considered potential channel restoration needs in this plan.

Canal Accessibility

In order to maintain the Canal, the City needs as unlimited access as practical along its entire length. Limited access points and locations of right-of-way (ROW) encroachments were recorded during the Canal inventory. Items such as general debris, heavy bank vegetation, and encroachments of private property into the Canal ROW such as fences, decks, retaining walls, sheds, and other similar obstructions were recorded.

INSPECTION & ANALYSIS RESULTS

Section Overview

This section provides the results of the inspection, and analyses of the various Canal features relative to flow control, Canal capacity, channel restoration and water quality, and Canal access. Information collected during the Canal inventory for each feature includes its type and purpose, operational notes, digital photographs, sketches illustrating general configuration and dimensions where appropriate.

Flow Control

Table 7-1 provides a summary of the major flow control structures, beginning at the upstream end of the Canal. The locations of these structures are shown on ***Figures 7-1 and 7-3***.

A description of each control structure including its purpose, condition, instrumentation and controls, and estimated capacity is provided. Referenced conditions of control structures are based on visual observations at the time of the field inspection. The level of effort for these observations was limited in scope and consistent with that required for a planning level

[37] Haestad Methods, Inc. Copyright. FlowMaster V.6.0. CulvertMaster V.2.0.

[38] Harza Engineering Company, *Draft Erosion and Sediment Control Plan*, Albany Hydroelectric Project FERC No. 11509-000-OR, January, 1997.

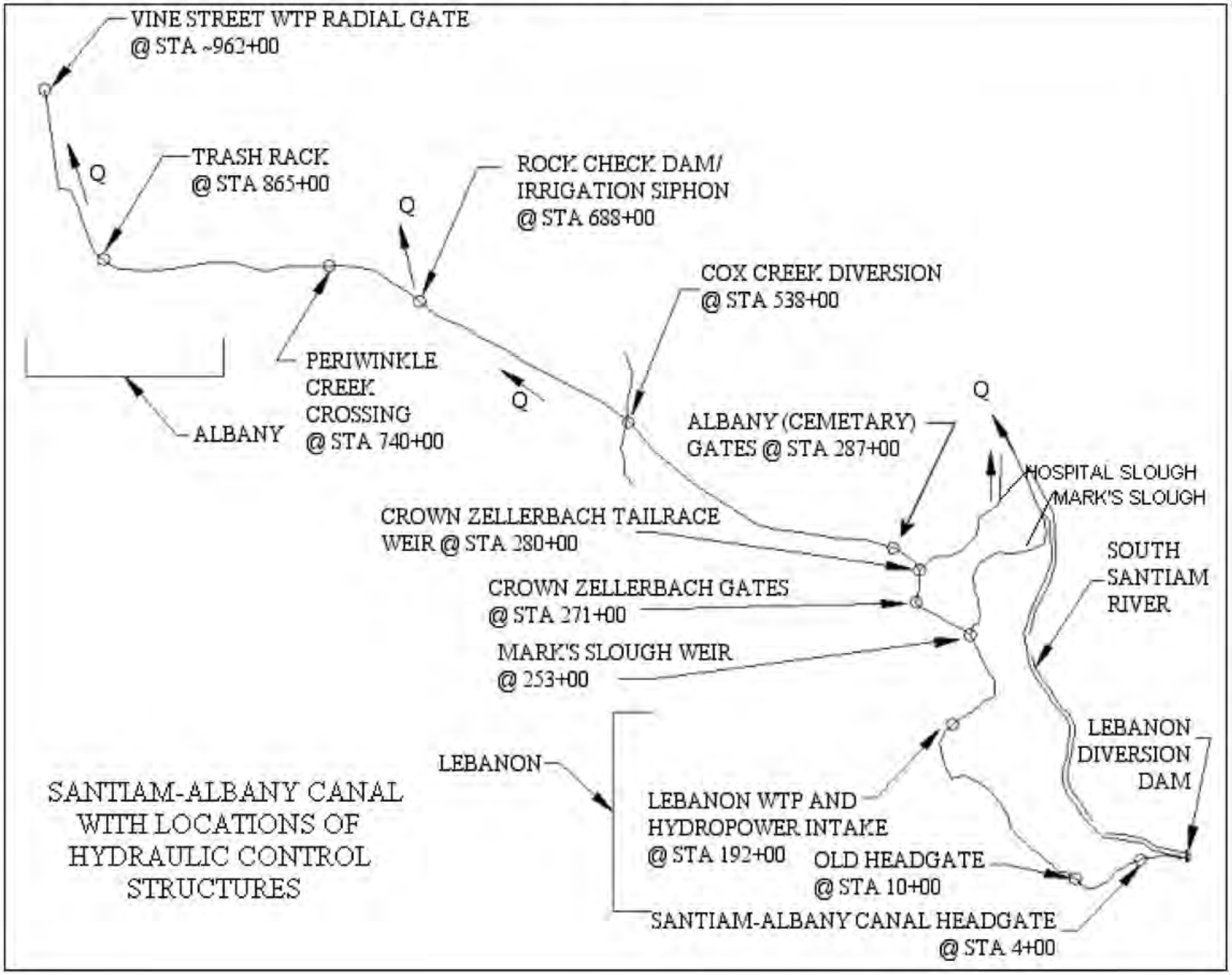
[39] Harza Engineering Company, *Albany Hydroelectric Project Development Cost Analysis Study*, A FERC Minor License, FERC Project No. 11509-000, April 17, 2000.

document. Core samples or other tests were not completed, and a detailed structural evaluation is recommended as part of the pre-design process for recommended improvements.

Table 7-1: Existing Flow Control Structures

Station	*Description*
*	Lebanon Diversion Dam (located on South Santiam River)
4+00	Santiam-Albany Canal Headgate
10+00	Old Headgate
192+00	Lebanon WTP and Hydropower Facility
253+00	Mark's Slough Weir
271+00	Crown Zellerbach (CZ) Gates (also known as Lebanon Gates and Skip Gates)
280+00	CZ Tailrace Weir (at Hospital Slough)
287+00	Albany Gates (also known as Hospital Gates and the Cemetery Gates)
538+00	Cox Creek Diversion
688+00	Rock Check Dam / Irrigation Siphon
740+00	Periwinkle Creek Crossing
865+00	Trash Rack
962+00	Vine Street WTP Radial Gate

Figure 7-3: Existing Control Feature Location Map (Not to Scale)



Lebanon Diversion Dam

The Lebanon Diversion Dam is owned and operated by the City of Albany. It was constructed in 1925 to replace an older wooden dam located downstream of the current location that was destroyed during a flood. The dam, located at South Santiam River Mile 20.8, spans approximately 435 feet across the river (***Photograph 7-1***). The structure raises the water surface elevation to divert South Santiam River flow into the Canal entrance, located approximately 325 feet upstream on the south bank of the South Santiam River as shown in ***Photograph 7-1*** and ***Photograph 7-2***. The existing Canal entrance at Station 0+00 is unscreened, and has a log boom spanning it's entrance to help prevent floating debris from entering the Canal.

Photograph 7-1: Lebanon Dam on South Santiam River (RM 20.8)

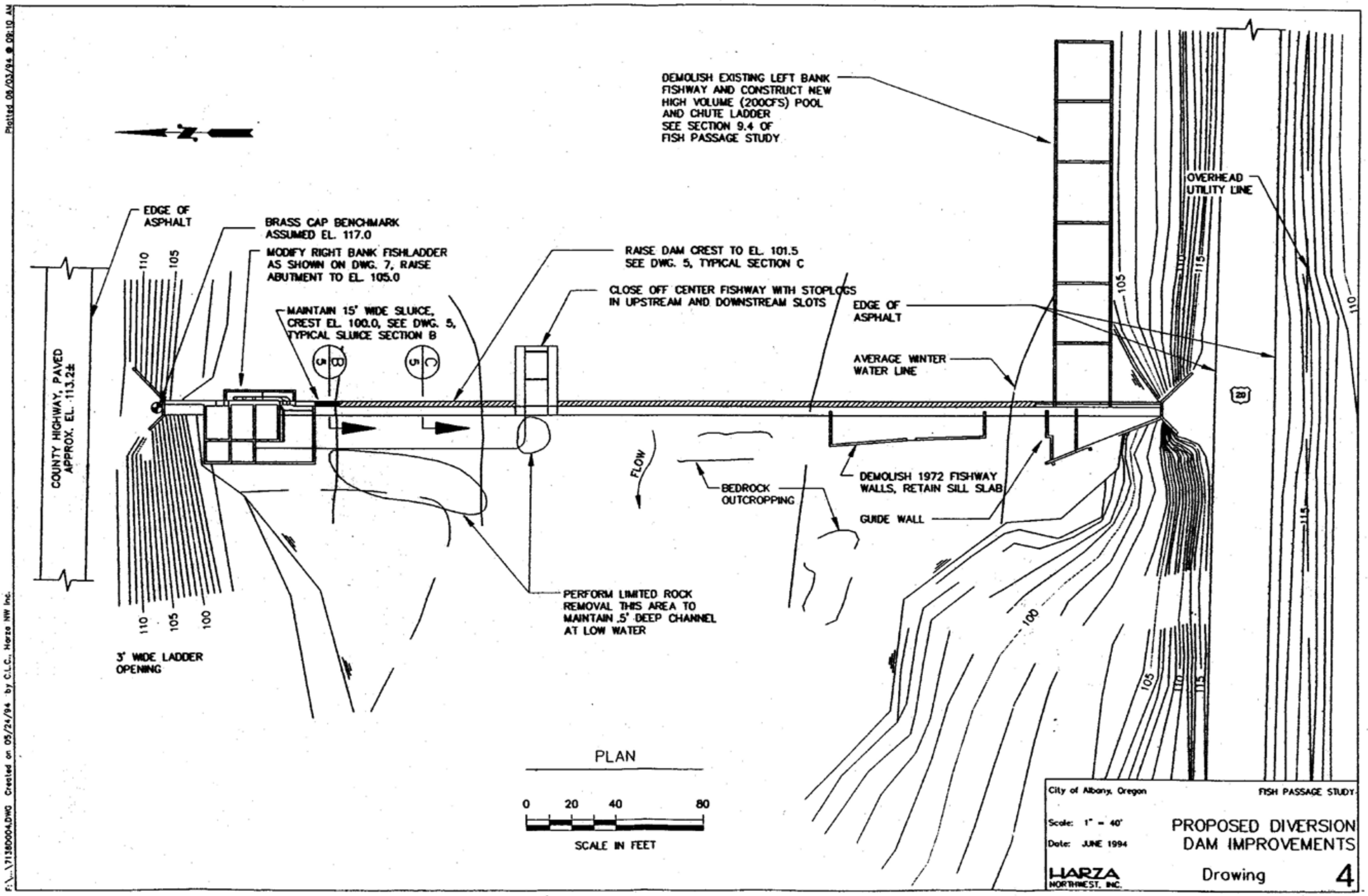
Photograph 7-2: Canal Point of Diversion and Logboom (RM 20.9, Canal Sta 0+00)

The dam raises the water surface approximately 5-feet over the original water surface elevation, which can be increased to about 7-feet through the use of two foot high timber flashboards that are used during low flow periods. The flashboards are typically installed annually in May / June (when low flows allow safe installation), and remain in place until high flows in the fall cause the flashboard system to fail (typically between October and December).

The concrete structure has undergone several modifications over its life. A single fish ladder was originally positioned near the center of the dam, and three other fish ladders have been constructed over the years near the north and south banks. The fish ladders do not meet current fish passage standards, and recommended modifications to the structure have been identified. Fish ladder improvements are part of an overall diversion improvement project that includes a concrete cap on the top of dam to replace the need for flashboards and a fish screen that will be installed at the entrance of the Canal to prevent fish entrainment.

The City has already obtained funding for these diversion improvement projects and is preparing to begin the design and construction process. These improvements are therefore not included in recommended Canal improvement projects, as they are funded outside of this process. ***Figure 7-4*** shows the proposed dam improvements that were licensed by FERC as part of the approved hydroelectric project (FERC No. 11509-000-OR).

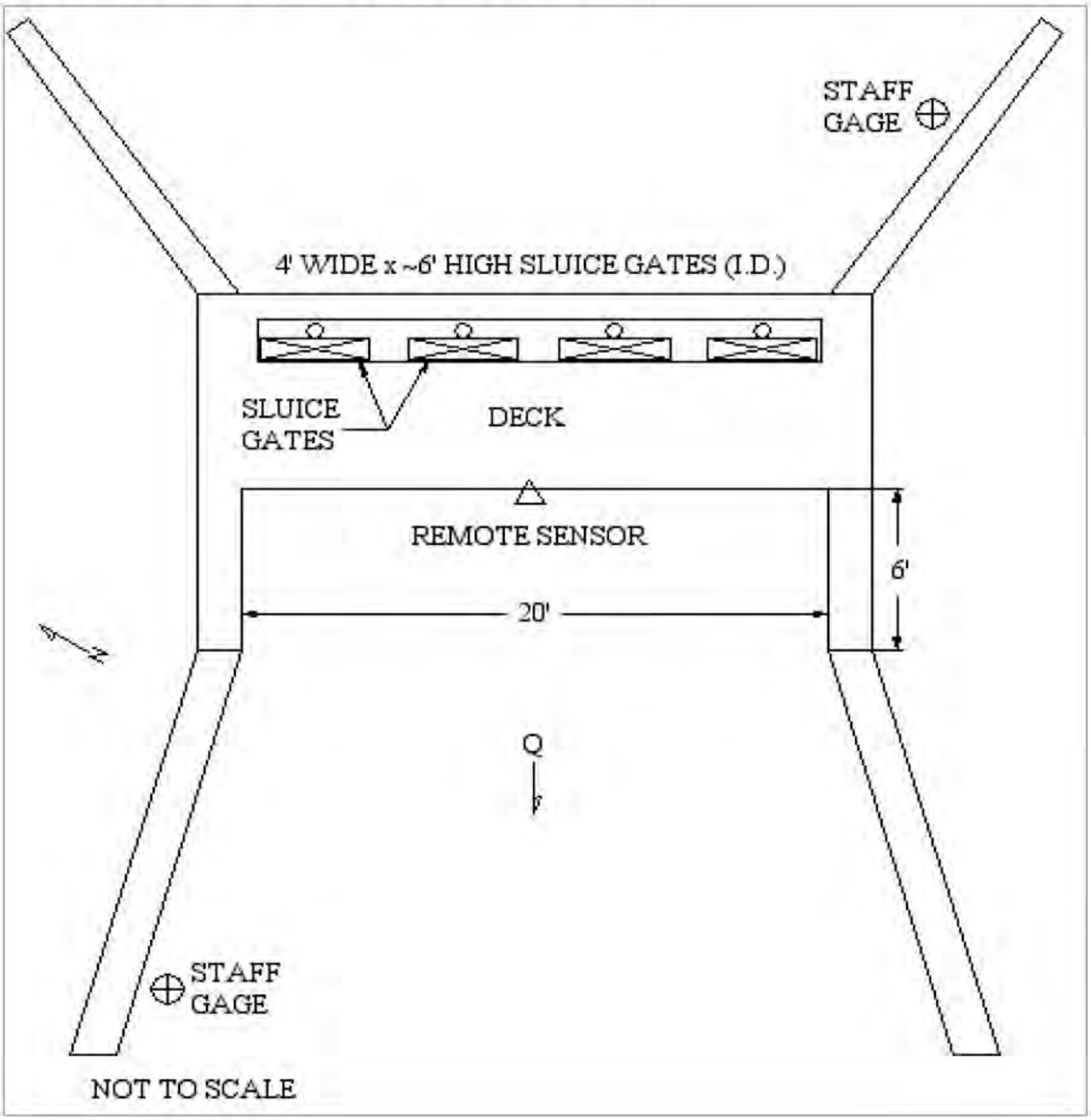
Figure 7-4: Diversion Dam Improvements approved by the FERC as a feature of the Hydroelectric Project



Santiam-Albany Canal Headgate (Sta 4+00)

The Canal headgate structure is owned and operated by the City of Albany. It was constructed in 1924 to replace the old headgate located 600 feet downstream of the current location. This structure controls flows entering the Canal from the South Santiam River and protects the Canal from flooding when the river level is high. ***Figure 7-5*** provides a plan view of the structure configuration, and ***Photographs 7-3 through 7-5*** provide three views of the structure.

Figure 7-5: Canal Headgate Plan View (Not to Scale)



Photograph 7-3: Upstream View of Canal Headgates (looking downstream)

Photograph 7-4: View of Upstream Staff Gage on South Wing Wall of Headgate

Photograph 7-5: View of Canal Headgates Remote Sensor Control Box on Deck Railing

The headgate regulates flows through four manually operated sluice gates. The sluice gates are located in parallel across the upstream side of the structure. Each of the four gates is 4-feet wide and 6-feet tall. The structure is cast-in-place concrete and spans 20-feet across the Canal channel. The top deck is 9.5-feet wide, allowing gated pedestrian access across the Canal. The deck is about 20-feet above the channel invert and supports the mechanical gate controls. Instrumentation includes staff gauges located on the upstream and downstream sides of the structure, and a remote level sensor (pressure transducer) located on the downstream side of the structure. This sensor continuously records Canal levels and transmits the data to the Vine Street Water Treatment Plant (WTP) in Albany via phone lines.

The headgate is manually controlled by City staff, and is adjusted as needed to maintain desired flows. Depending on flow conditions, adjustments can lag 12-20 hours before flow changes are observable at the Vine Street WTP. During flood events, the gates are closed to prevent South Santiam River flow from entering the Canal. However, even when the gates are closed, surface water and other inflows to the Canal downstream of the headgates have caused flooding.

With a potential open area of about 96 square feet in the full open position (4 times 24 sq ft), a design flow of 310-cfs would result with a through gate velocity of about 3.2 feet per second (fps). This calculated velocity is a reasonable value for the Canal based on previous velocity data provided to MWH by the City[40]. A local headloss of about 0.2 feet would be expected through the gates at this velocity. Depending on the velocity of flows in the Canal at the gate, there could be additional headlosses if the local velocity through the gate openings is greater than the ambient velocity in the Canal.

While the headgate structure is still a functioning facility, Albany staff is experiencing ongoing and increasing maintenance problems, particularly with the gate seals, guides, and operators. Gate binding has been reported, and the seals are old and unable to fully stop flow into the Canal. A brief visual inspection indicates that the structure is in stable condition, but would not likely meet current personnel safety standards with respect to walkways and handrail protection.

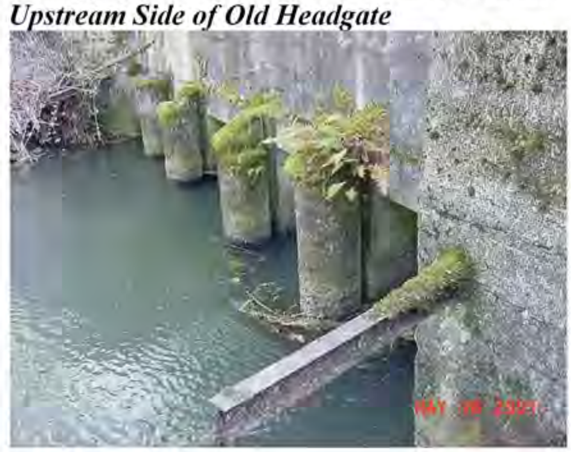
[40] Albany-Lebanon Santiam Canal Cross Section Report, Field Data provided to Harza by City of Albany, various measurement dates from Summer of 1996.

Old Headgate (Sta 10+00)

The old headgate falls within Linn County right of way as it is used as a 20-foot wide bridge for Old Headgate Road. The old headgate was constructed in the 1890's as part of the Santiam-Lebanon Canal project but was abandoned as an active control structure in 1924 with the construction of the new headgate. The original purpose of this structure was to control flows into the Canal from the South Santiam River. The sluice gates have since been removed and the old headgate controls flows only by the limited open space formed by the old sluice gate openings. ***Photographs 7-6 and 7-7*** provide an overview of the structure.

Photograph 7-6: Top View of the Old Headgate

Photograph 7-7: Abandoned Gate Guides on Upstream Side of Old Headgate



The old headgate is a cast-in-place concrete structure about 20-feet high. The structure spans 26 feet across the channel, and wing-walls extend up both banks of the Canal for a total structure length of about 60 feet. Hydraulically, the structure acts as a box culvert with five openings; four are 5-feet high and 4-feet wide, and the center opening is 5-feet high by 6-feet wide. The original sluice gates have been removed from the structure, however the gate guides still remain. The USGS maintains a gauging station (Gage #14187600) on the downstream side of the structure.

This facility is estimated to have a flow capacity of 252 cfs with two feet of water surface clearance[41]. Based on this information, an open channel analysis was conducted during this study that indicates a capacity of 366 cfs with the Canal water surface at the top of the openings. This structure is no longer an active control feature. No obvious flaw or areas of concern were noted during the visual inspection, other than the potential to trap debris, which could limit flows.

[41] CH2M, *An Engineering Report on the Santiam-Albany Canal*, July 1966

Lebanon WTP and Hydropower Intake (Sta 192+00)

This structure, operated and maintained by the City of Albany, was constructed in the 1920's to divert water into the powerhouse and raise the water surface elevation behind the structure to divert water into the Lebanon WTP intake.[42] The structure consists of a 29-foot wide trash rack, an eight-foot diameter morning glory weir, four non-operational sluice gates (closed), and a six-foot wide stop log weir that is used as a sluiceway. Historically, water flowing through the morning glory weir turned a turbine (removed in 1968)[43] that was used to generate power and excess flow was routed through the sluice gates and over the stop logs. Water pooled behind the structure, as it does today, allowing for water diversions to the Lebanon WTP. Today, Lebanon's WTP intake houses a trash rack and rotating screen. ***Photographs 7-8 and 7-9*** show an overview of the facility, and a schematic view of the structure and resulting flow routing is provided in ***Figures 7-6 and 7-7***. During the field inspection it was noted that the concrete was showing signs of cracking, with moss and grasses growing in several locations along the structure.

Photograph 7-8: Overview of intake at Lebanon WTP (Looking Downstream from WTP Intake)

[42] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999

[43] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999

Figure 7-6: Lebanon Hydropower Intake and WTP Diversion Structure, Plan View (Not to Scale)

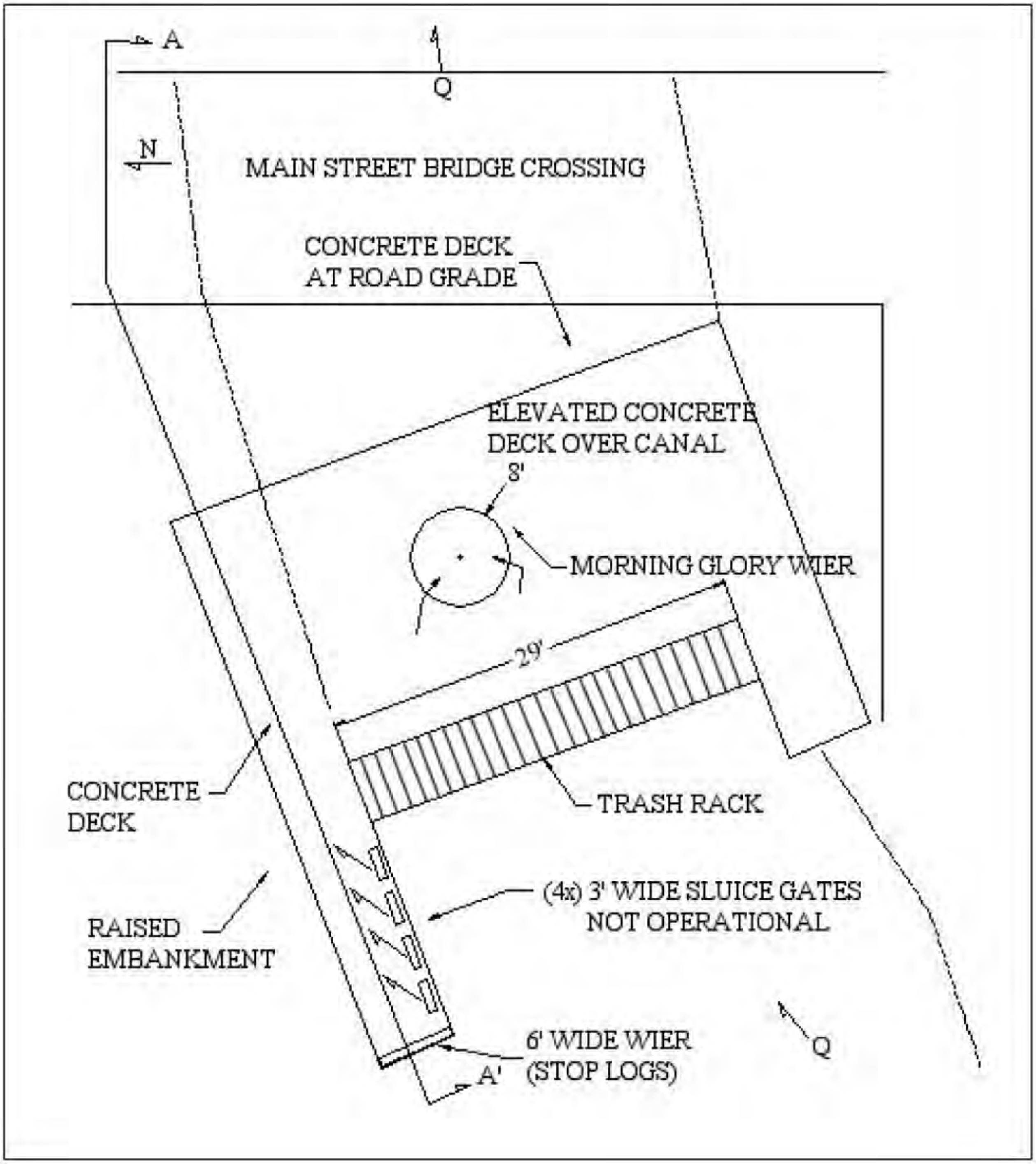
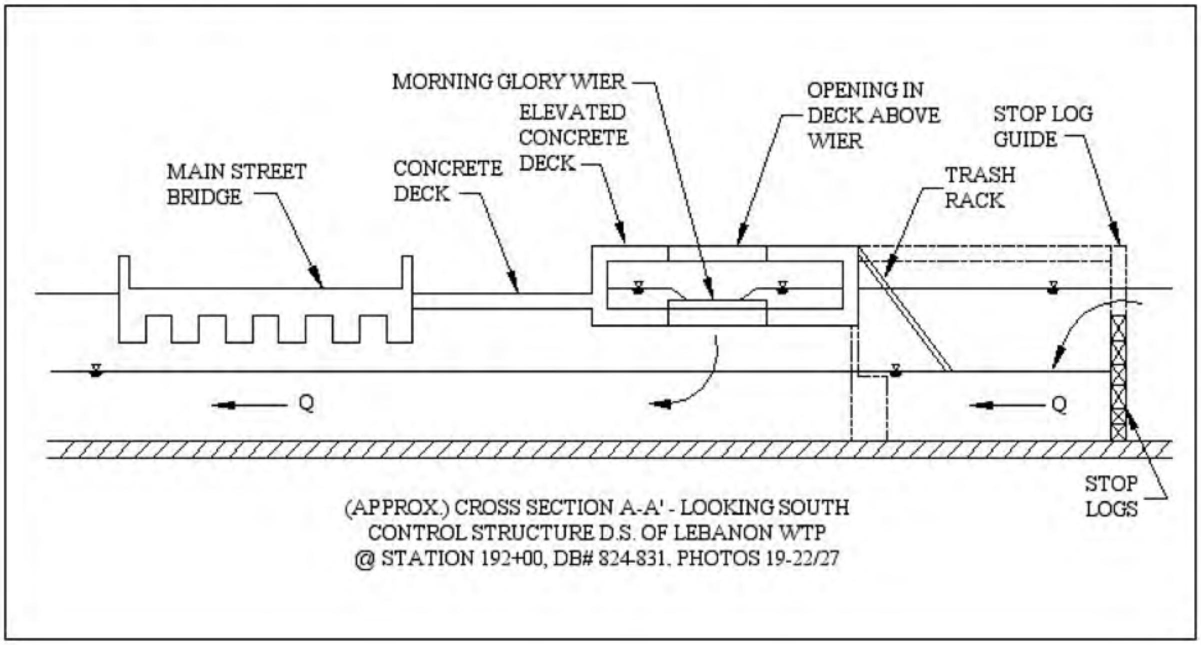


Figure 7-7: Lebanon Hydropower Intake and WTP Diversion Structure, Section A-A' (Not to Scale)



Photograph 7-9: Non-Operational Sluice Gates (Looking upstream)

Currently, flows cannot be controlled with this structure since the sluice gates are non-operational. Due to complex flow routing, lack of as-builts, and unobservable interior changes, the ability of this structure to pass 310-cfs was not evaluated. However, based on visual observations in the field at flow rates substantially less than 310-cfs, it is likely that modification will be needed in order to pass the design flow rate. Gates and sluiceways could be repaired and opened, or the concrete area routing flow through the morning glory weir could be removed to provide more capacity. With the abandonment of the hydroelectric plant, the purpose of this facility has been changed from its original intent and the morning glory weir is no longer required.

Mark's Slough Weir (Sta 253+00)

The Mark's Slough weir structure is owned and operated by the City of Albany. It was constructed around 1910 by the Willamette Pulp and Paper Company on a flume connecting the Canal to Mark's Slough. They had complaints from the public about mill waste and sought a form of disposal by building a flume southeast of the mill to deliver mill waste to Mark's Slough. The structure was to provide overflow from the Canal to this flume.[44] This structure is composed of eleven, 9-foot wide, 5-foot high stop log overflow weirs paralleling the Canal. The weir allows excess flow to spill from the Canal, and return to the South Santiam River at river mile 16.8 via Mark's Slough. Today, the structure is used to control flows delivered to Mark's Slough for downstream water rights. ***Photograph 7-10*** provides a picture of the weir structure, and ***Figure 7-8*** provides a plan and section view of the weir structure.

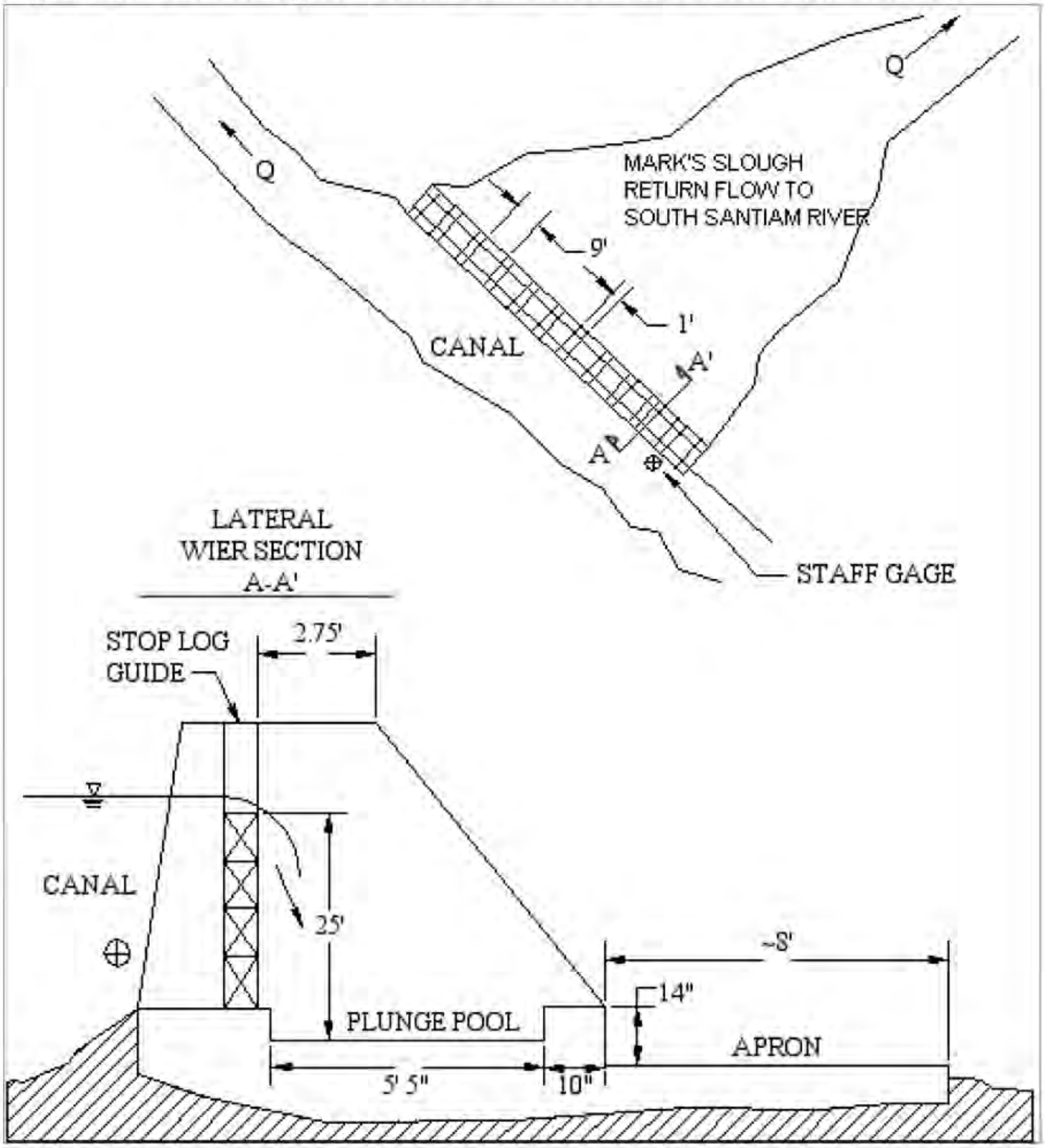
Photograph 7-10: Overflow Weirs at Mark's Slough Diversion.

As currently configured the weir limits the maximum water surface elevation of the Canal in this location. During high flow conditions the weir structure and Mark's Slough provides limited flood relief to this reach of the Canal. Adjustments to raise or lower the crest elevation of this structure must be made manually by City staff.

This is a simple gravity structure that appears to be structurally sound based on a brief visual observation. Gaps were noted in some of the stop logs, as was leakage around and through many of the stop logs. This structure is sited such that it would be feasible to route excess flow from the Canal to Mark's Slough by manually removing stop logs, or automatically in the future with the addition of automated gates in place of stop logs. No information on the capacity of Mark's Slough during various seasons and flow regimes is known. The Slough's capacity should be evaluated prior to installing automated gates for flood protection at this location. Given the configuration parallel to the Canal, this structure is not a restriction to flow in the Canal.

[44] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999

Figure 7-8: Mark's Slough Diversion Plan View and Weir Section (Not to Scale)



Crown Zellerbach (CZ) Gates (Sta 271+00, also known as Lebanon Gates, and Skip Gates)

The CZ Gates are currently owned and operated by the City of Lebanon. This structure was constructed some time after the 1940's[45] by the Crown Zellerbach Corporation. The gates are used to raise the water surface elevation behind the structure to allow for upstream withdrawals into a local fire pond, and to back up water to allow for upstream diversions from the Canal to Mark's Slough.

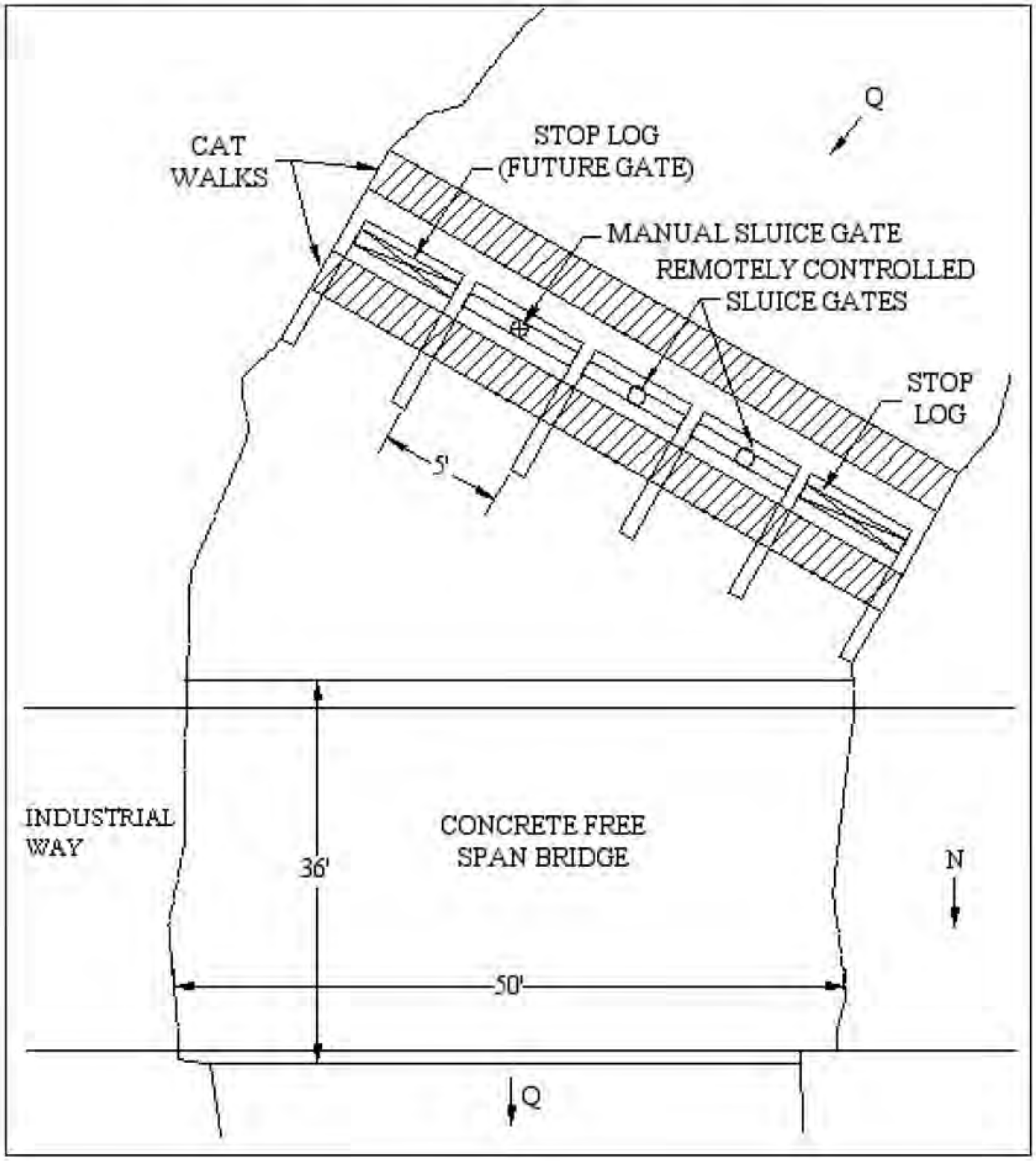
The CZ Gates consist of five, 5-foot wide, control bays. Two of the bays contain remotely operated sluice gates, operated based on a level sensor located upstream of the facility. One bay has a manually controlled sluice gate, and the remaining two contain stop logs, as shown in ***Photograph 7-11*** and ***Figure 7-9***.

Given the five 5-foot wide gates, and that the gates are roughly 7' tall, the structure has the potential to provide approximately 175 square feet of conveyance area with the stop logs removed. Based on the 310-cfs design flow, calculated velocity through these gates would be 1.8 fps which is reasonable based on previous velocity measurements. With a velocity of 1.8 fps a head loss of less than 0.1 feet could be expected at this structure. The above analysis indicates that this structure would not be a flow constriction at 310-cfs.

Photograph 7-11: Upstream Side of CZ Gates.

[45] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999

Figure 7-9: CZ Gates Flow Control Facility (Not to Scale)



CZ Tailrace Weir (Sta 280+00, at Hospital Slough)

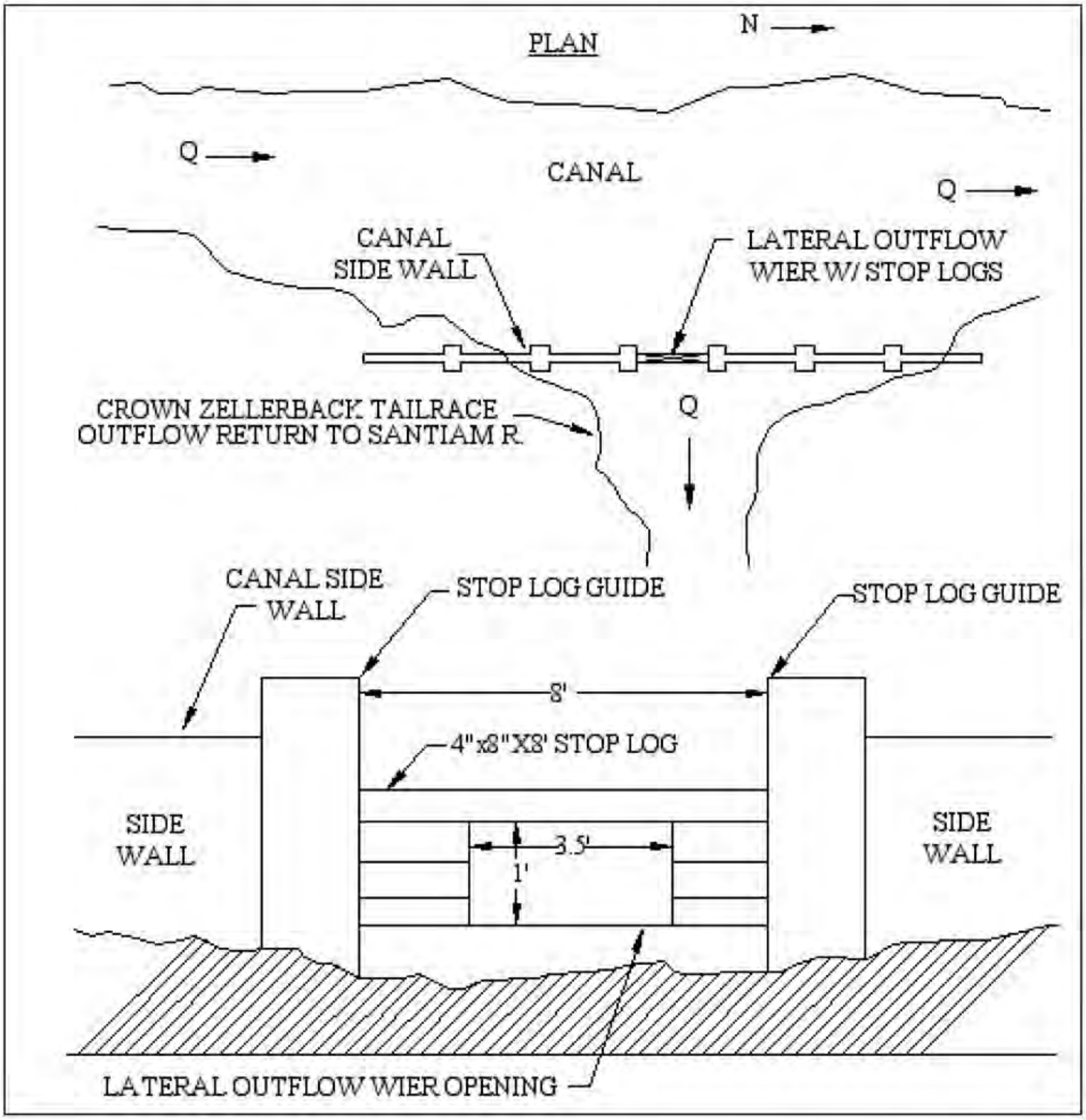
The CZ tailrace weir structure is owned and operated by the City of Albany. The weir structure and tailrace (Hospital Slough) were probably built between 1914 and 1936 while owned by the Crown-Willamette Corporation.[46] The weir structure allows Canal water to be diverted to Hospital Slough, which drains to the South Santiam River at approximately RM 15.3.

The structure consists of three, 8-foot wide stop log weirs held by concrete piers paralleling the Canal. The weir is currently configured with a 3.5-foot wide by 1-foot high orifice as shown in ***Photograph 7-12*** and ***Figure 7-10***.

Photograph 7-12: Overflow Weir to CZ Tailrace (Hospital Slough)

[46] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999

Figure 7-10: CZ Tailrace Weir, Plan View and Weir Profile (Not to Scale)



As currently configured the weir and orifice influences the maximum water surface elevation of the Canal at this location. During high flow conditions, the facility provides an overflow spill to Hospital Slough, which helps mitigate flooding along downstream reaches of the Canal. Given the structure's parallel alignment to the Canal, it does not restrict Canal flows below 310-cfs.

Albany Gates (Sta 287+00, also known as Hospital Gates, and the Cemetery Gates)

According to the Santiam-Albany Canal Historic Document Record, the Albany Gates may have been constructed in 1923.[47] These gates, owned by the City of Albany, span approximately 25-feet across the Canal, as shown in ***Photograph 7-13*** and ***Figure 7-11***.

Photograph 7-13: Upstream Side of Albany Gates

The structure houses twin, manually operated, sluice gates and two stop log bays. Each sluice gate is 4 foot wide by 7.5 foot tall in the fully raised position. Each stop log bay is approximately 7.5-feet wide by 11-feet tall. Restriction of flow by these gates causes water to be diverted back to the South Santiam River through the CZ tailrace.[48] The gates are used to control the Canal water surface elevation to allow gravity withdrawals upstream of the structure, and are modulated to control flooding by routing flow to the CZ tailrace. Instrumentation includes level sensors, located upstream and downstream of the gates that send data to the Vine Street WTP.

With the stop logs removed this structure has the potential to provide a flow area of about 225 square feet. Calculated velocity through the structure at 310-cfs is about 1.4 fps, which is a reasonable value for the Canal and would not cause significant headloss or a flow restriction.

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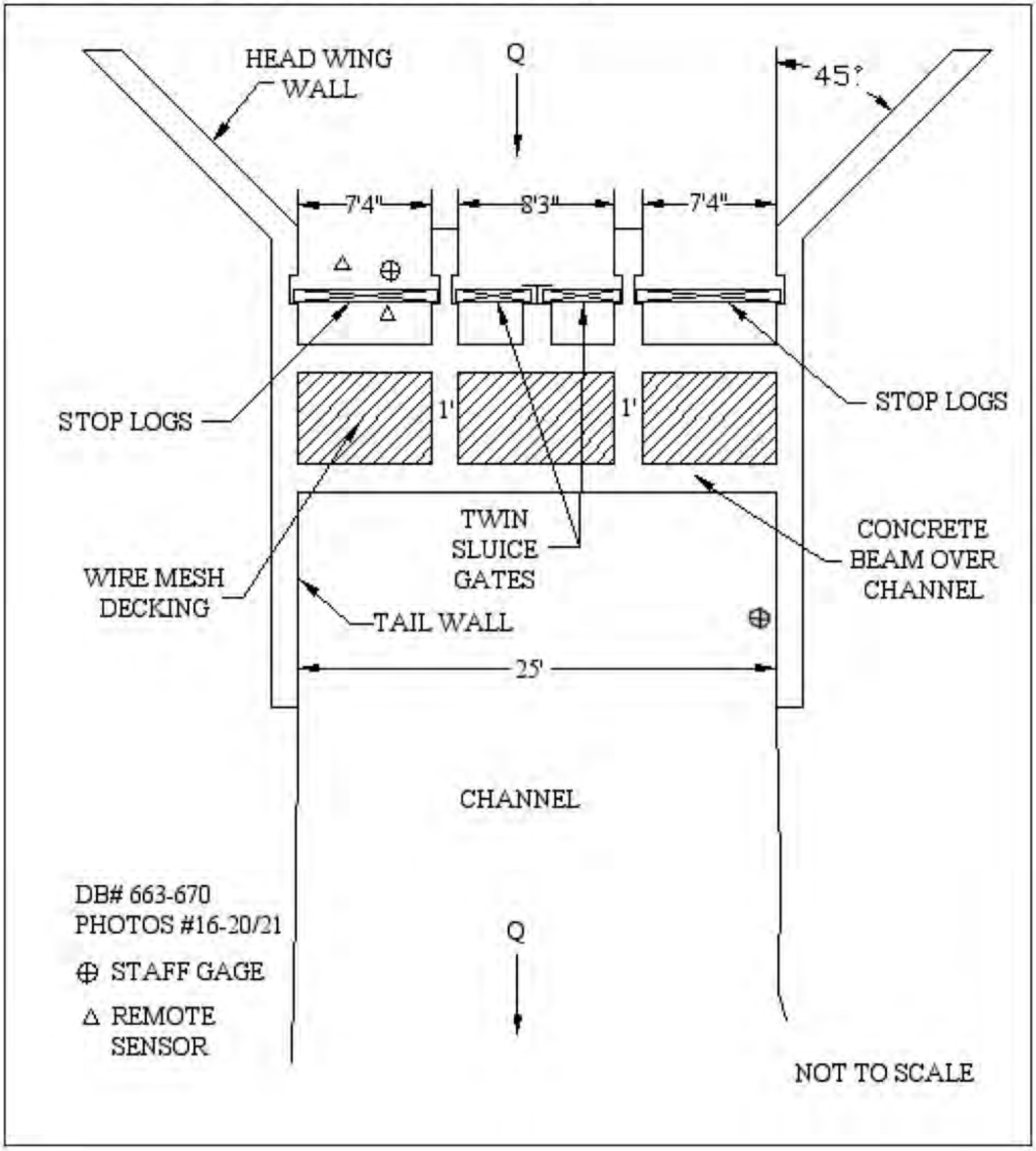
[47] Archeological Investigations Northwest, Inc., *Albany Santiam Canal-Historic Documentation Record,* May, 1999
[48] CH2M, *An Engineering Report on the Santiam-Albany Canal*, July 1966

Figure 7-11: Plan View of Albany Gates (Not to Scale)



Cox Creek Diversion (Sta 538+00)

The Cox Creek Diversion is owned and operated by the City of Albany. The Canal crosses over Cox Creek in a rectangular concrete channel, as shown in ***Photograph 7-14***. A 4-foot high by 10-foot wide box culvert conveys Cox Creek under the concrete Canal channel, perpendicular to the Canal flow as shown in ***Photograph 7-15***. A plan view of the crossing structure is shown in ***Figure 7-12***.

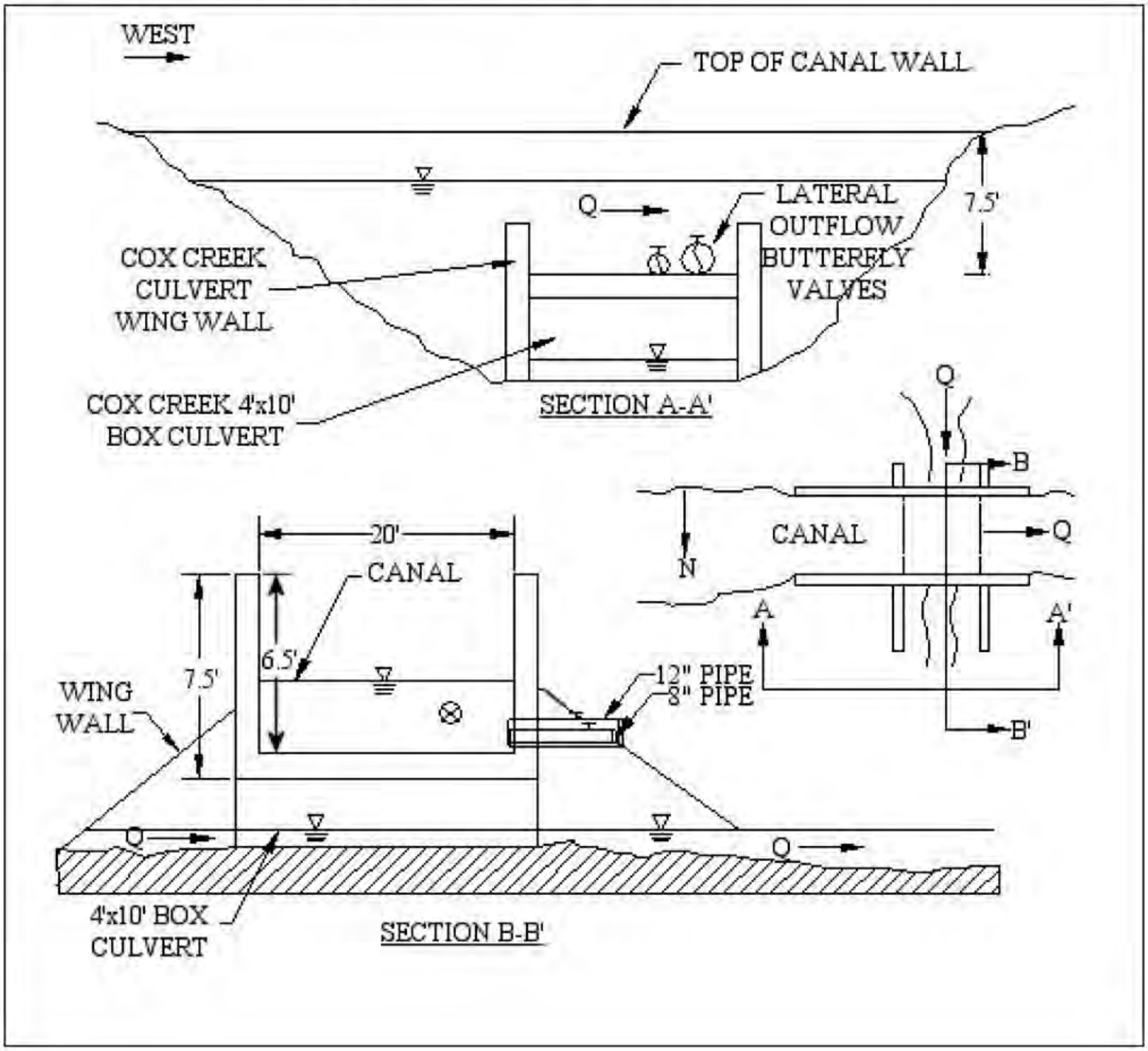
Photograph 7-14: Canal at Cox Creek Crossing Looking Upstream

Photograph 7-15: Cox Creek Culvert Passing Under Canal (Note Butterfly Valves)

Two manually operated butterfly valves (12-inch diameter and 8-inch diameter) are located on the north side of the concrete channel to provide an outlet to Cox Creek. Water is released to augment low summertime flows in Cox Creek, allowed by a 7-cfs water right. Cox Creek supplies water to Timber Linn Lake, Swan Lakes, and Waverly Lake in the City of Albany.

Assuming one-foot of freeboard in the Canal channel structure, an area of approximately 110 square feet (20' wide by 5.5' deep) is available in the Canal channel. A velocity of 2.8 fps would be necessary to pass the 310-cfs design flow. This velocity is consistent with those calculated for other structures, and is a reasonable value.

Figure 7-12: Cox Creek Crossing and Section Views (Not to Scale)



Rock Check Dam (Sta 688+00)

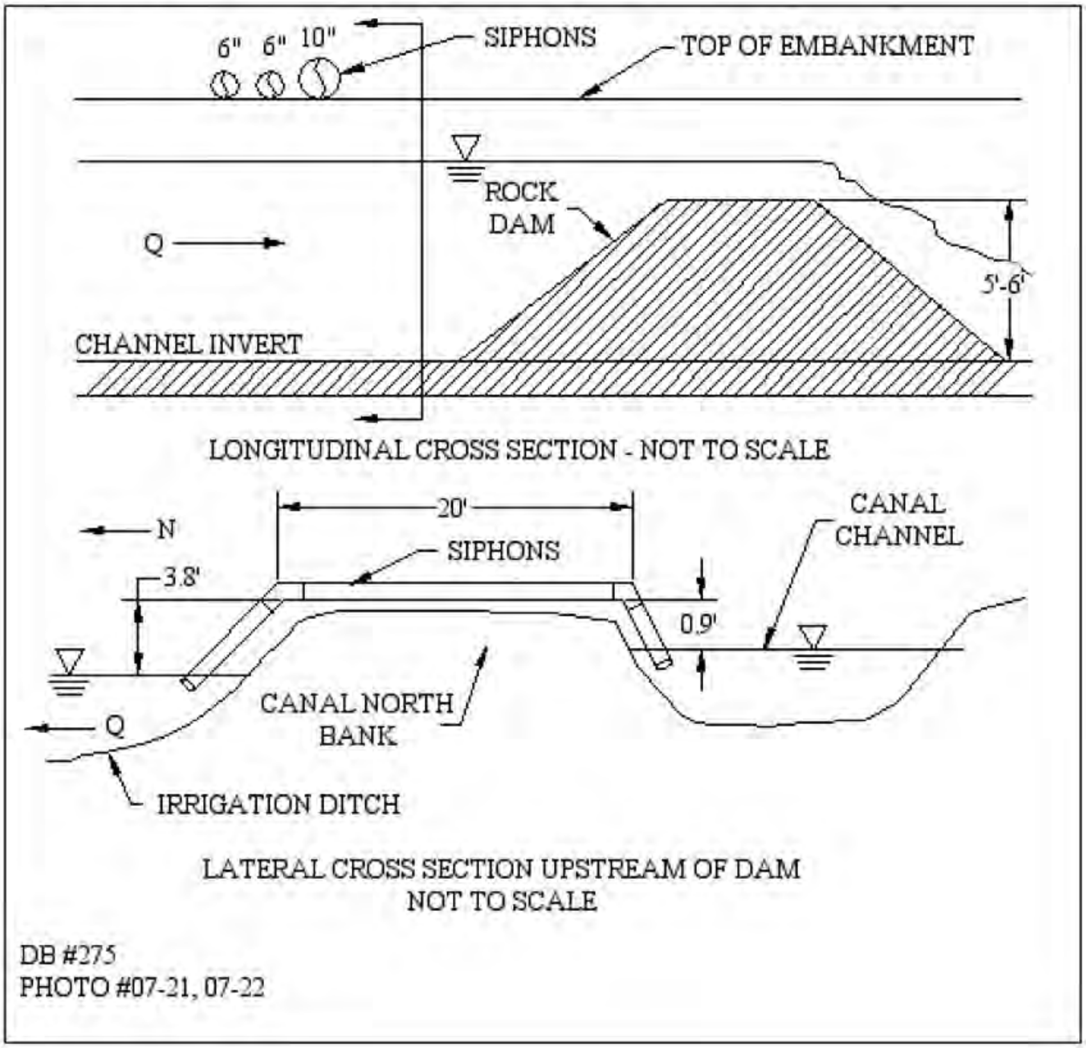
A 5-foot high rock check dam was built in the Canal at station 688+00 between 1966 and 1971. The purpose of the check dam is to raise the water surface elevation behind the structure to allow for a siphoned withdrawal from the Canal to the Grand Prairie Water District's irrigation ditch. Water is withdrawn through three siphon pipes, two 6-inch outside diameter (OD), and one 10-inch OD. ***Photographs 7-16 and 7-17*** and ***Figure 7-13*** provide an overview of the facility.

Photograph 7-16: Rock Check Dam in Channel

Photograph 7-17: Irrigation Siphon Pipes

Based on its current configuration, the rock dam could cause localized flooding at high flow rates. Based on an estimated depth of 2-feet of flow over the rock dam's approximately 30-foot width, a conveyance area of 60 square feet would be available to pass the 310-cfs. This area would result in a calculated velocity of 5.2 fps, which would raise the water surface nearly 0.5-feet due to velocity head. This is a higher velocity than the other structures which indicates a restriction point in the Canal.

Figure 7-13: Rock Dam and Irrigation Siphon (Not to Scale)



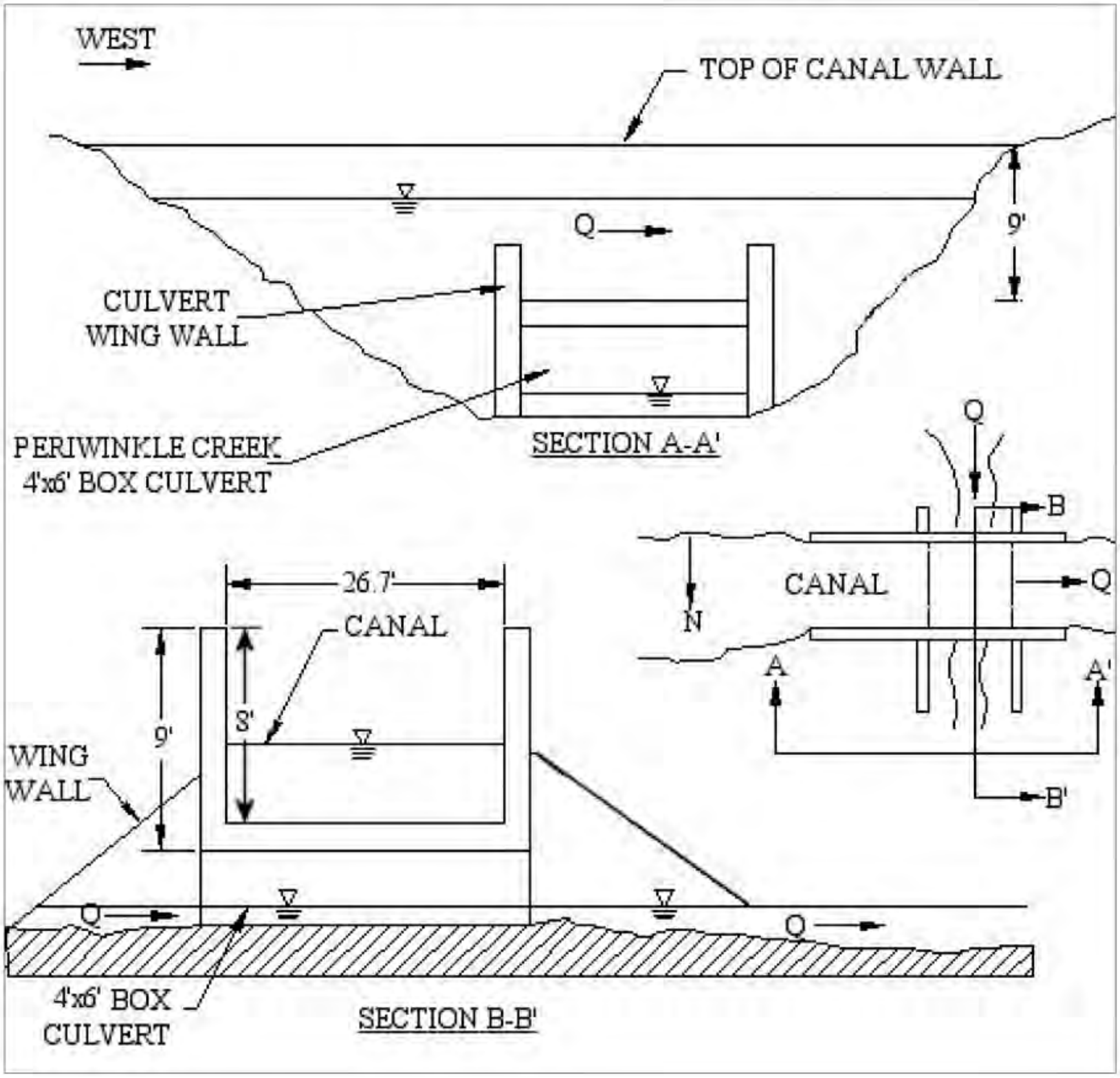
Periwinkle Creek Crossing (Sta 740+00)

The City of Albany owns and maintains the Periwinkle Creek Crossing. The Canal crosses over Periwinkle Creek in a rectangular concrete channel, as shown in ***Photograph 7-18***. A 4-foot high by 6-foot wide box culvert conveys Periwinkle Creek under the Canal channel, perpendicular to the Canal flow. A plan view of the crossing structure is shown in ***Figure 7-14***.

Photograph 7-18: Periwinkle Creek Crossing

Assuming one-foot of freeboard in the Canal channel structure, an area of approximately 187 square feet (26.7' wide by 7' deep) is available in the Canal channel. A velocity of 1.7 fps would be necessary to pass the 310-cfs design flow, and is a reasonable value.

Figure 7-14: Periwinkle Creek Crossing and Section Views (Not to Scale)



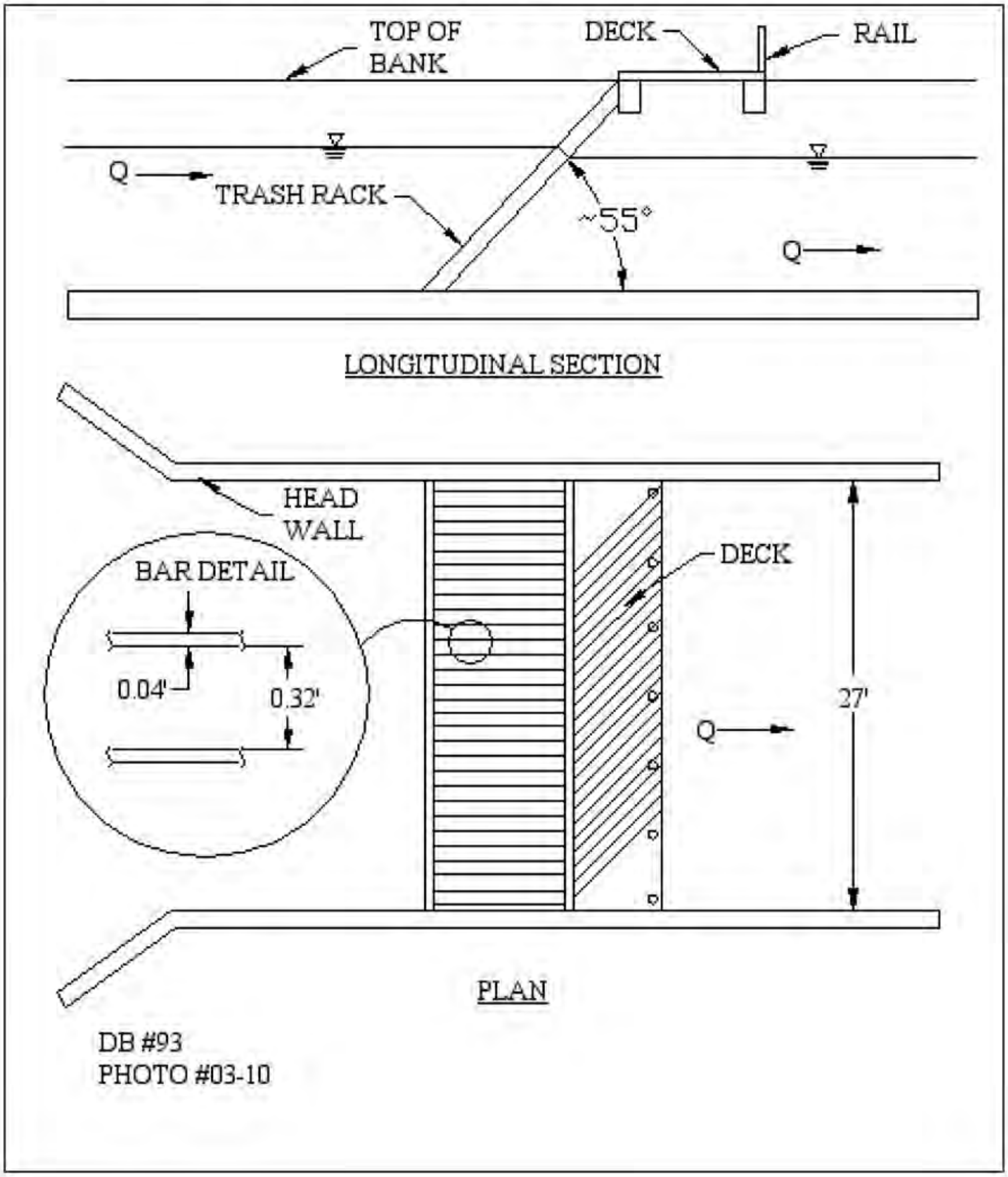
Trash Rack (Sta 865+00)

The City of Albany owns and maintains a trash rack on the Canal just south of 34th Avenue to collect large debris. The trash rack spans 27-feet across the width of the Canal and is comprised of ½-inch thick flat bars spaced 3.8-inches apart. ***Photograph 7-19*** and ***Figure 7-15*** provide an overview of the structure.

Photograph 7-19: Upstream Side of Trash Rack Looking Towards East Side of Canal

This structure controls flows by limiting the flow area within the Canal section. Based on an open area through the trash rack of approximately 135 square feet (27-feet wide by 5-feet deep), a velocity of 2.3 fps would be expected through the trashrack at the 310-cfs design flow. This is a reasonable design value for trashrack structures, and would not result in more than 0.1-feet of headloss. As long as the trashrack is kept clean from debris, it does not appear that this structure is a constriction to flow.

Figure 7-15: Trash Rack Section and Plan Views (Not to Scale)



Vine Street WTP Radial Gate (~Sta 962+00)

The Vine Street Water Treatment Plant (WTP) radial gate is owned and operated by the City of Albany. The 8-foot automated radial gate shown in ***Photograph 7-20*** was installed in 1991 as part of an upgrade to the Vine Street WTP. It is used to raise the water surface elevation in the Canal to divert water into the Vine Street WTP intake, located approximately 65 feet upstream from the gate. The WTP intake houses a 12-foot diameter rotating screen and sprayer that can be seen in the background of ***Photograph 7-20***. Flow spilling under the gate discharges to the Calapooia River approximately 200 feet downstream at Station 964+00. No deficiencies were observed or noted by City operations personnel during the Canal inventory.

Assuming that the gate is fully open and not influencing flow, and that there is 1' of freeboard on the Canal sidewalls, results in approximately 54 square feet of conveyance area. Further assuming, normal flow conditions, a slope of 0.001 ft/ft, and a Manning's n of 0.015 (conservative for concrete channel) results in a capacity of 315-cfs.

Photograph 7-20: Radial Gate at Albany WTP, Looking Upstream

Existing Flow Instrumentation

Currently, the only instrumentation on the Canal are remote depth sensors as listed in ***Table 7-2***. This data supplements data provided by the USGS stream gage located at the old headgates.

Table 7-2: Existing Remote Sensor Locations

Station	*Survey Record Number*	*Notes*
004+00	1028	Remote sensor on downstream side of headgate structure
011+00	1015	USGS remote stream gage sensor, ID Number 14187600, Near Lebanon
271+00	697	Remote sensor, used for automatic gate control at Crown Zellerbach gates
287+00	669	Remote sensor, upstream side of Albany Gates
287+00	670	Remote sensor, downstream side of Albany Gates
441+00	512	Remote sensor by SE corner of Langmack Rd. bridge, upstream side
548+00	395	Remote sensor on upstream abutment on south side of Red Bridge Road crossing
787+00	160	Remote sensor (level gage) on upstream end of Waverly Dr. culvert head wall on south side
962+00	432	Remote Sensor at radial gate downstream from WTP intake

Ensure Canal Capacity

In addition to flow control structures, the Canal's ability to convey a design flow rate of 310-cfs can be limited by bridges and culverts and by open channel capacity. Bridges and culverts have the potential to control and/or restrict flow in the Canal, as they are limited in capacity under high flow conditions. Based on the limited data collected during the Canal inventory, a preliminary hydraulic analysis was performed for each of the bridges and culverts to identify structures that may be undersized for the 310-cfs design flow. Hydraulic analysis performed as part of this plan did not include an evaluation of the Canal's open channel capacity. Recent studies performed by Harza Engineering and Kramer, Chin, and Mayo (KCM) were used to estimate sediment removal quantities necessary to achieve the design capacity.

Ensuring Canal capacity also involves the removal of uncontrolled lateral inflows. These inflows present a flooding risk to the Canal and should be re-routed to historic drainage ways.

Bridge Analysis and Results

52 bridges were analyzed in total, 11 west of Interstate-5 and 41 east of Interstate-5. Bridges located east of Interstate-5 (~Station 760+00) were evaluated utilizing an open channel analysis based on Manning's equation, with Heastead's Flowmaster[49] computer program.

[49] Haestad Methods, Inc. Copyright. FlowMaster V.6.0.

Input parameters included the cross sectional area at the bridge opening, estimated channel slope, a Manning's coefficient value of 0.050 (representing a channel with a clean bottom and brush on sides), and a water surface elevation corresponding with the bottom edge of the bridge girders. Bridges located west of Interstate-5 were evaluated based on data from the KCM *HEC-RAS study*[50], and preliminary HEC-RAS assessment performed by Harza during the Albany *Hydroelectric Project Development Cost Analysis Study*[51].

Table 7-3 summarizes analysis results. Bridge crossings with an estimated capacity within 10 percent of the design flow of 310-cfs were considered to have adequate capacity due to the preliminary nature of the data collected and the ability to remove sediment to increase the flow area. The analysis identified two locations, a private driveway bridge at Station 117+00 and the Franklin Street Bridge at Station 137+00, with suspect capacity and the need for more detailed analysis and possible replacement.

Culvert Analysis and Results

Similar to the bridge analysis, a hydraulic analysis was performed to identify possible locations with limiting culvert capacity for each of the 12 culverts conveying the Canal. The 6 culverts located east of Interstate-5 were evaluated utilizing an open channel analysis based on Manning's equation, with Heastead's CulvertMaster[52] computer program. Input parameters for this evaluation included: culvert dimensions (depth and width), a Manning's coefficient value of 0.028, a water surface elevation corresponding with the top of culvert elevation, culvert length, and slope. Evaluation of the 6 culverts located west of Interstate-5 were based on the findings reported in KCM's *Canal Hydraulic Study*[53], and preliminary HEC-RAS assessment performed by Harza during the *Albany Hydroelectric Project Development Cost Analysis Study*[54]. The results of this analysis are summarized in ***Table 7-4***. The results show that surcharging may be required in order to achieve a capacity of 310-cfs for six culverts. Culverts were identified as potentially being under capacity if a 310-cfs flow rate resulted in less than one-foot of freeboard. Based on limited survey data, it appears that adequate freeboard exists to provide the required surcharging in all locations except for the KGAL Road culvert at Station 455+00.

When solving for capacity under the surcharging scenario, it is necessary to provide the tailwater elevation for that particular discharge. Because the tailwater elevation at 310-cfs is not known for these culverts, it is not possible to determine the actual headwater elevation required to pass 310-cfs. Instead, the required surcharge or head differential was estimated. The surcharge is the difference between the upstream and downstream water levels under fully submerged conditions (i.e. full pipe flow). Once a culvert is fully submerged, the required head differential to pass a given flow will remain the same regardless of the actual depth.

[50] KCM, *Canal Hydraulic Study* – 3rd Avenue to I-5 (Project WC-98-5). Prepared for City of Albany, March, 1999.

[51] Harza Engineering Company, *Albany Hydroelectric Project Development Cost Analysis Study*, A FERC Minor License, FERC Project Jo. 11509-000. April 17, 2000.

[52] Haestad Methods, Inc. Copyright. CulvertMaster V.2.0.

[53] KCM, *Canal Hydraulic Study* – 3rd Avenue to I-5 (Project WC-98-5). Prepared for City of Albany, March, 1999.

[54] Harza Engineering Company, *Albany Hydroelectric Project Development Cost Analysis Study*, A FERC Minor License, FERC Project Jo. 11509-000. April 17, 2000.

[illegible]	Old Headgate Road	1012	N/A	N/A	110	>310	N/A	this location. The capacity of this structure is discussed i the flow control structures section of this Chapter.
038+00	Mobile Home Park	997	0.0008	0.050	556	>310	N	Adequate capacity.
100+00	River Road	972	0.0009	0.050	411	>310	N	Adequate capacity.
107+00	private driveway	969	0.0009	0.050	241	>310	N	Adequate capacity.
109+00	private driveway	966	0.0009	0.050	304	>310	N	Adequate capacity.
110+00	private driveway	965	0.0009	0.055	280	>310	N	Adequate capacity.
111+00	private driveway	961	0.0009	0.050	317	>310	N	Adequate capacity.
112+00	private driveway	960	0.0009	0.050	244	>310	N	Adequate capacity.
114+00	private driveway	956	0.0009	0.050	245	>310	N	Adequate capacity.
117+00	**private driveway**	**952**	**0.0007**	**0.050**	**124**	**245**	**Y**	**Further investigation is recommended.**
120+00	Mountain River Road	-	0.0005	0.050	205	>310	N	Adequate Capacity. This bridge constructed after initial field inspection was completed.
133+00	private driveway	929	0.0005	0.050	176	294	Y	Estimated capacity within 10% of 310 cfs. Assume adequate capacity.
137+00	**Franklin Street**	**917**	**0.0005**	**0.050**	**152**	**250**	**Y**	**Further investigation is recommended.**
144+00	Timber RR	905	0.0013	0.050	188	>310	N	Adequate capacity.
156+00	Main St.	875	0.0003	0.055	259	>310	N	Adequate capacity.
165+00	2nd St.	843	0.0003	0.050	271	>310	N	Adequate capacity.
176+00	E St.	859	0.0003	0.050	209	>310	N	Adequate capacity.
189+00	2nd St.	842	0.0003	0.050	219	>310	N	Adequate capacity.
193+00	Main St.	823	0.0007	0.050	209	>310	N	Adequate capacity.
197+00	RR	815	0.0001	0.050	852	>310	N	Adequate capacity.
199+00	Park St.	814	0.0007	0.050	207	>310	N	Adequate capacity.
201+00	Oak St.	811	0.0007	0.050	307	>310	N	Adequate capacity.
205+00	Grove St.	803	0.0007	0.050	288	>310	N	Adequate capacity.
210+00	Williams St.	795	0.0007	0.050	327	>310	N	Adequate capacity.
211+00	Grant St.	791	0.0007	0.050	339	>310	N	Adequate capacity.
215+00	Hiatt St.	785	0.0007	0.050	366	>310	N	Adequate capacity.
223+00	E. Ash St.	769	0.0015	0.050	452	>310	N	Adequate capacity.
234+00	private driveway	754	0.0004	0.050	211	>310	N	Adequate capacity.
237+00	Woods RV Park	747	0.0004	0.050	258	>310	N	Adequate capacity.
249+00	Wheeler Rd.	719	0.0004	0.050	437	>310	N	Adequate capacity.
259+00	N. Williams St.	706	0.0004	0.050	274	>310	N	Adequate capacity.
267+00	Timber RR	701	0.0004	0.050	373	>310	N	Adequate capacity.
272+00	Industrial Way	687	0.0116	0.050	364	>310	N	Adequate capacity.
316+00	Hwy 20	638	0.0018	0.050	379	>310	N	Adequate capacity.
332+00	Gore Rd	623	0.0018	0.050	422	>310	N	Adequate capacity.
441+00	Langmack Rd	510	0.0025	0.050	208	>310	N	Adequate capacity.
479+00	Tallman Rd	472	0.0018	0.050	178	>310	N	Adequate capacity.
548+00	Red Bridge Rd	390	0.0006	0.050	141	280	Y	Estimated capacity within 10% of 310 cfs. Assume adequate capacity.
610+00	Leichty Property	345	0.0105	0.050	230	>310	N	Adequate capacity.
749+00	Three Lakes Rd	207	0.001	0.050	248	>310	N	Adequate capacity.
757+00	I-5 N	191	-	-	-	>310	-	Adequate capacity based on KCM study.
758+00	I-5 S	190	-	-	-	>310	-	Adequate capacity based on KCM study.
803+00	Columbus St.	141	-	-	-	~310	-	KCM study does not identify any capacity constraints at 310 cfs.
907+00	RR Bridge	61	-	-	-	~310	-	KCM study states that this bridge can pass 310 cfs with t bottom of the bridge deck submerged.
939+00	12th Ave.	43	-	-	-	~310	-	KCM study states that this bridge restricts flow at 310 cf without raising the Canal banks.
942+00	11th Ave.	38	-	-	-	~310	-	KCM study states that this bridge restricts flow at 310 cf without raising the Canal banks.
945+00	10th Ave.	33	-	-	-	~310	-	KCM study states that this bridge restricts flow at 310 cf without raising the Canal banks.
953+00	7th Ave.	20	-	-	-	~310	-	KCM report states that this bridge can pass 310 cfs with the water surface against the bridge deck.
956+00	6th Ave.	18	-	-	-	~310	-	KCM report states that this bridge can pass 310 cfs with the water surface against the bridge deck.
959+00	5th Ave.	13	-	-	-	~310	-	KCM report states that this bridge can pass 310 cfs with the water surface against the bridge deck.
962+00	4th Ave.	9	-	-	-	>310	-	KCM report states that this bridge can pass 310 cfs.

OTE: Capacities shown in this table are based on limited survey data and are only intended to provide an initial screening of potentially under capacity dges. Information contained in this table is approximate and should not be used for design or other uses not outlined in this Water Facility Plan.

otnotes

Pedestrian bridges were not evaluated.

Slopes used to develop capacity estimates were based off 2' contours unless otherwise noted in ***Appendix B***.

lysis Results

Manning's n	Assumed Slope[1] (ft/ft)	Length (ft)	Type	Dimensions	Capacity w/ No Surcharge (cfs)	Req'd Surcharge @ 310-cfs (ft)	Freeboard @ 310-cfs (ft)	Conclusion
0.028	0.0077	35	CMP Pipe-Arch	7.6' x 11.8'	199	0.7	2.5	Adequate Capacity
0.028	**0.0136**	**73**	**CMP Pipe-Arch**	**6.6' x 9.8'**	**131**	**1.5**	**0.7**	**Less than 1' of freeboard, further investiga**
0.028	0.0015	110	CMP Pipe-Arch	7.6' x 11.8'	297	0.9	3.6	Adequate Capacity
0.028	0.0024	63	CMP Pipe-Arch	7.6' x 11.8'	142	0.7	3.8	Adequate Capacity
0.028	0.0177	60	CMP Pipe-Arch	7.6' x 11.8'	232	0.7	6.6	Adequate Capacity
0.028	0.0071	55	CMP Pipe-Arch	7.6' x 11.8'	232	0.7	7.2	Adequate Capacity
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.
-	-	-	-	-	>310	-	-	Adequate Capacity based on KCM study.

le are based on limited survey data and are only intended to provide an initial screening of potentially under capacity
his table is approximate and should not be used for design or other uses not outlined in this Water Facility Plan.

stimates were based on invert elevations unless otherwise noted in ***Appendix B***.

Bridge and Culvert Analysis Limitations

The bridge and culvert analyses and results presented in this plan are subject to many variables. Open channel hydraulic analysis is highly dependent on the slope of the Canal, and slopes calculated based on 2-foot contours from topographic maps likely do not represent local variations in slope near the structures. Additionally, no analysis of potential backwater affects was performed for the crossings. Because a more thorough hydraulic analysis requiring additional data was outside the scope of this plan, the hydraulic analyses presented above was used to screen the bridges and culverts to identify those that may not be able to convey the full 310-cfs design flow. Conclusions presented in this plan should be used to identify locations that may be under capacity and warrant further investigation and possible replacement.

Canal Open Channel Capacity / Sediment Removal

Some reaches of the Canal have overtopped during periods of high surface runoff, and past analyses have documented concern for the Canal to have adequate capacity to convey 310-cfs without overtopping[55]. Additional analysis of the Canal capacity was outside the scope of this plan; however, identification of sediment removal needs and potential channel modifications are a significant issue regarding the Canal.

The most recent capacity analysis was performed in support of the FERC hydropower license application, and is documented in Chapter 3 of Harza's 2000 *Hydroelectric Project Development Cost Analysis Study*[56] for three design flows, including the design flow used in this plan of 310 cfs. In the feasibility study, sediment removal and dredging needs were estimated based on quantities and bid tabs provided by the City for previous dredging efforts. The results of this analysis were presented as a range of cost estimates to obtain the desired capacity of 310-cfs.

In addition to the overall design flow value, other factors will influence the amount of dredging necessary. Instrumentation to provide real-time measurement of Canal flows, in combination with automation of gates on the control structures to both control flows within the Canal and allow spill of excess water will help to reduce dredging needs identified in the 2000 study. Ongoing maintenance and upkeep of the Canal that addresses local sediment issues will also help to increase its capacity outside of additional dredging projects.

During the 2000 study, the "expected" value was provided as the most probable effort required to meet dredging needs. However, based on the factors mentioned above and observations made during the field inspection the "optimistic" estimate of $1,500,000 used in the feasibility study has been used as an allowance for sediment removal in this plan.

Additional channel restoration may be needed west of Interstate-5 to pass a design flow rate of 310-cfs. The hydraulic study performed by KCM in 1999[57] identified that flooding would likely occur between 7th Avenue and Pacific Boulevard, downstream of the railroad bridge, and around the trash rack at a 310-cfs flow rate. The study listed lowering the water surface

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[55] Harza Engineering Company, Albany Hydroelectric Project Development Cost Analysis Study, A FERC Minor License, FERC Project No. 11509-000, April 17, 2000.

[56] Harza Engineering Company, Albany Hydroelectric Project Development Cost Analysis Study, A FERC Minor License, FERC Project No. 11509-000, April 17, 2000.

[57] KCM, *City of Albany, Canal Hydraulic Study – 3rd Avenue to I-5 (ProjectWC-98-5),* March, 1999

elevation, raising the banks, or a combination of the two to increase the capacity of the Canal in these reaches. An allowance for raising the Canal banks to increase channel capacity is discussed in the Recommended Improvements section of this chapter.

Lateral Inflows

Lateral inflows to the Canal include both point and non-point sources, that have a significant effect on Canal flows. Point sources include storm drains, gutter drains, established drainage ditches, and similar discrete points conveying surface water to the Canal. Non-point sources include overland runoff and seasonal watercourses draining into the Canal along its banks. During the flood event in 1996, the headgates were closed but flows in excess of 400 cfs were experienced at the Waverly Drive culvert.[58] This high flow rate resulted in flooding at12th, 20th, and 40th Avenues and just west of Interstate-5. These flows were entirely due to uncontrolled lateral inflows. Immediately following the flood, the City began working on identifying and eliminating lateral inflows. Although drainage from an estimated 680 acres has been re-routed, an estimated 1,600 acres still drains directly into the Canal as shown in ***Table 7-5***, and on ***Figure 7-1***. Areas draining into the Canal have been approximated based on drainage facility plans for Albany and Lebanon and limited topographic information available for unincorporated areas adjoining the Canal. The identified drainage areas only represent large tracts of land draining to the Canal and do not incorporate all lateral inflows identified in the Canal inspection. This approach, while adequate for facility planning efforts, should be updated as field survey data is collected to better define drainage patterns and the extent of each basin.

Table 7-5: Lateral Inflow Acreage Draining into Canal

Beginning Station	*Ending Station*	*Length Along Canal (ft)*	*Total Acres Draining to Canal*[59]	*Running Total of Acres Draining to Canal*
0+00	30+00	3,000	190	190
135+00	225+00	9,000	280	470
315+00	420+00	10,500	660	1,130
440+00	455+00	1,500	20	1,150
465+00	510+00	4,500	90	1,240
665+00	695+00	3,000	50	1,290
695+00	740+00	4,500	120	1,410
750+00	760+00	1,000	60	1,470
760+00	780+00	2,000	130	1,600
Total	--	***39,000***	***1,600***	***1,600***

[58] KCM, *City of Albany, Canal Hydraulic Study – 3rd Avenue to I-5 (ProjectWC-98-5)*, March, 1999

[59] Source: City of Albany GIS Map, Figure 7-1

Removal of lateral inflows will require diversion of ditch/culvert inflows to cross-over culverts to re-establish historic drainage patterns. This will be difficult in some areas due to topography and upstream conditions. The 2001 field inspection identified 196 locations of lateral inflows including drainage ditches, drainage pipes, and sheet runoff as shown *Table 7-6*.

Table 7-6: Characterization of Lateral Inflows

Type of Inflow	*Headworks through City of Lebanon*	*Farmlands (between Lebanon and Albany)*	*Within the City of Albany*	*Total*
Drainage ditches	8	31	12	***51***
Drainage pipes	25	3	7	***35***
Sheet runoff	47	48	15	***110***
Total	***80***	***82***	***34***	***196***

Drainage pipes observed during the inventory ranged from small roof drains to large stormwater runoff pipes, and sheet runoff ranged from residential yards to farm fields. All point-source inflows identified in the City of Albany are from private property. Additional detail and specific locations of the noted Canal inflows are provided in the Canal inventory data in ***Appendix B***.

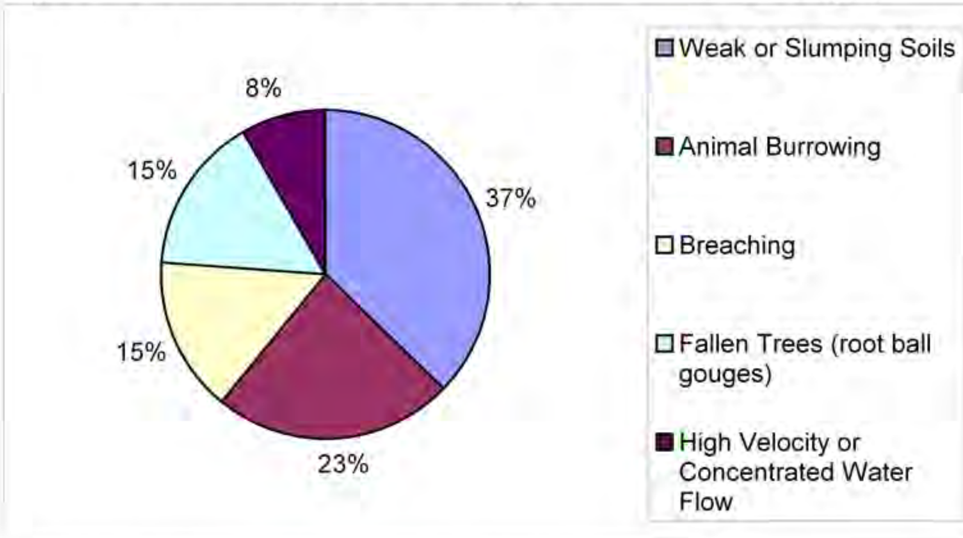
Channel Restoration and Water Quality

In addition to the overall capacity issues with the Canal, local issues associated with erosion, scour, observable local sedimentation and debris were recorded during the Canal inventory. Bank erosion and sedimentation were the most frequent problems noted during the Canal inventory. Based on data from the inventory, bank damage was classified into five categories as follows:

- Weak or slumping soils
- Animal burrowing
- Breaching
- Fallen trees (root ball gouges)
- High velocity or concentrated water flow

Figure 7-16 provides a summary of relative percentages for each category of recorded bank damage.

Figure 7-16: Summary of Bank Damage Observations By Category**



*** Based on 201 recorded data points noting bank damage.*

As shown in the ***Figure 7-16***, 60 percent of the local bank damage is the result of bank failures caused by weak soils and animal burrows. The severity of damage from animal burrows was not quantified since they are often small and/or obscured by dense brush. There is no practical way to address the problem of animal burrowing except to repair damage due to large burrows as part of routine maintenance when these areas sluff into the Canal or create other obvious problems.

Local Sedimentation and Bank Erosion Problems

During the field inspection several types of erosion and sedimentation were observed. Erosion was seen in areas with high velocities or concentrated flows, along the bank at the water surface (due to small amounts of wave action and current), the outside of Canal bends, and at locations with debris or tree falls.

Sedimentation patterns observed at the inside of Canal bends, and scour along the outside of bends is an expected channel forming pattern. These areas of erosion eventually lead to sedimentation as the eroded soils are transported from areas with higher velocities and settle in areas of lower velocities.

Where property constraints do not require maintaining the existing Canal width, these types of local sedimentation problems can be minimized by simply widening the channel and laying the side slopes back at a shallower angle. In Canal reaches with limited right-of-way, or other property or physical constraints, the use of structural solutions will be required to maintain the Canal capacity and minimize sedimentation problems. Typical structural engineered solutions include use of rip-rap, bioengineered slopes, or concrete structures (both cast-in-place and pre-cast concrete).

Evidence of bank slumping, or creeping, was noted where trees along the Canal banks have curved trunks. In many cases where trees indicate bank creep, it appears that most of the movement occurred long ago and that the bank is now relatively stable. This is evident by the shape of the upper bank and by it not having scarps or cracks showing rotation or lateral spreading. In these cases no action is recommended unless tree trunks are in the water. Trees whose trunks are in the water are recommended to be removed as they represent a potential obstruction in the Canal.

Trees growing on the Canal banks are both a benefit and a maintenance issue. They shade the Canal helping maintain water quality, and provide a means for bank stabilization, but are also the cause of many bank erosion problems. Fallen trees and tree limbs can cause two general types of problems related to erosion and sedimentation. When a tree falls, the pulled-up root ball creates a large gouge in the bank. This scenario loosens soils and increases erodibility, as well as increases the surface area that Canal water is in contact with. Additionally, when fallen trees or other debris form along the bank, flows are diverted around the debris and are concentrated toward the opposite bank, which can cause local scour and back-eddies that can lead to sedimentation.

This plan recommends removing all fallen trees and any other brush or debris in the Canal. Once the obstruction is removed bank damage should be repaired and related sedimentation should be removed. Removal of sediment from the toe of the bank should consider overall bank stability factors like the angle of repose for a given soil type, and the mean velocities of the channel for the given stretch.

Water Quality

Analysis of the Canal's water quality is included in *Chapter 8 – Vine Street Water Treatment Plant*. The Canal's headwaters, originating in the South Santiam River, are considered to have good water quality (low turbidity, temperature, etc). However, there are a number of locations along the Canal where contaminants and urban runoff from developed areas can potentially be introduced, as reported in the *Source Water Assessment Report for the City of Albany* completed in 2002[60]. As noted in *Chapter 8 – Vine Street Water Treatment Plant*, the City routinely monitors the quality of raw water, testing for potential contaminants and is capable of removing most contaminants through the treatment process. This raw water monitoring has not found significant levels of contaminants. However, due to urban storm water runoff and drainage of farm fields adjacent to the Canal, water quality at the Vine Street WTP is generally more turbid and may be less pristine than the water quality in the South Santiam River at the Canal's diversion point.

Uncontrolled lateral inflows are the biggest factor in increased turbidity and pollutant levels during storm events. With the headgates closed during the flood event of 1996, turbidity levels at the Albany WTP were four times greater than those at the Lebanon WTP. This observation points to sediment loading from farm field drainage as the probable source.

Groundwater seepage can be another source of pollutants that reduces raw water quality. Seepage into and out of the Canal was observed at several locations during the field inspection. Most of these locations don't present an immediate concern for water quality except for a 2,500-foot stretch where the Canal runs by Cheadle Lake (located southeast of Lebanon, Sta 70+00 to 95+00). Seepage from Cheadle Lake presents a water quality concern as this lake receives discharge from a wood and pulp mill. The Canal and the lake are separated by a relatively narrow berm and the water surface elevation of the Canal is 3 to 5 feet below the water surface elevation of the lake.

[60] Source Water Assessment Report: City of Albany, Oregon - January 15, 2002, prepared by Oregon DEQ and OHDWP.

Water temperature is another factor that affects water quality. Canal temperatures can influence algal and microbial loading, as well as impact the "treatability" of the water. Trees and brush that line the Canal help to keep the water shaded, moderate water temperatures, reduce algae blooms, and provide bank stabilization (a reduction in erosion and turbidity). However, trees and brush must be maintained to prevent flow impedance.

Canal Accessibility

Maintaining access along the length of the Canal is very important for the City and adjacent landowners. Canal access allows the City to routinely inspect and maintain the Canal as well as respond to emergency conditions. These are essential activities to ensure reliable supply at the Vine Street Water Treatment Plant. Two primary causes for limited access are:

- Encroachments into Canal right-of-way (ROW), including fences, decks, retaining walls, sheds, and other obstructions, and
- Heavy bank vegetation.

Resolution of encroachment issues will require coordination with property owners, and will likely be a costly and time-consuming process. Therefore, the City should prioritize access needs and approach them accordingly, as funds become available.

Locations of encroachments and heavy bank vegetation encountered during the field inspection are recorded in the inventory data provided in ***Appendix B***. A summary of the limited access areas is provided in ***Table 7-7.***

Table 7-7: Summary of Limited Access by Type

Access Limitation Type	*Approximate Length Along Canal (ft)*	*Approximate Length Along Canal (mi)*
Encroachments	10,500	1.99
Heavy Bank Vegetation	22,200	4.20
Encroachments and Heavy Bank Vegetation	12,100	2.29
Total	***44,800***	***8.48***

RECOMMENDED IMPROVEMENTS

Based on the data collected during the Canal inventory, and on the analyses presented in the Inspection and Analysis Results section, recommendations for the Canal have been grouped into four categories:

- Update control structures,
- Ensure Canal capacity,
- Channel restoration, and
- Improve Canal access.

Suggested actions and recommended improvement projects are identified below for each category, along with supporting data and estimated improvement costs where appropriate.

Update Control Structures (Project Number C1)

Flow control improvements are recommended to improve existing control capabilities and to provide automated instrumentation and control. Improvements to existing gate structures, addition of new control gates, and automation and instrumentation of the gates will provide a flexible system to minimize flooding along the Canal.

Improvements identified for existing and recommended control structures are discussed below. A hydraulic study, defining the available capacity, is recommended for each of the proposed drainage channels receiving excess flow from the Canal.

Lebanon Diversion Dam

The Lebanon Diversion Dam creates the hydraulic head necessary to supply flow into the Canal at the headgates. Several improvements to the diversion dam were included in the FERC hydroelectric project license to address fish passage concerns described in the Inspection and Analysis Results section of this chapter. Suggested improvements include a recommended fixed crest replacement of the existing flashboard system, in conjunction with fish ladder modifications to improve upstream fish passage over the structure. Fish ladder modifications include abandonment of the two center ladders, reconfiguration of the north bank fish ladder, and replacement of the south bank fish ladder with a high-flow (200 cfs) pool-and-chute fishway. The dam's structural condition will be further evaluated during the design of improvements.

The Canal entrance will be screened to prevent both juvenile and adult salmonid and resident species from entering the Canal system. A flat-plate screen oriented parallel to the South Santiam River has been approved by both the FERC and resource agencies as part of the hydroelectric project licensing effort.

Funding for these improvements has already been secured and the design and construction process has begun. Consequently, no money is included in this plan for dam or fish screen improvements.

Santiam-Albany Canal Headgate (Sta 4+00)

Initially this structure was considered a separate improvement project from the dam and fish screen improvements discussed above. However, as the dam and fish screen project progressed the decision was made to include improvements to the headgate in the diversion dam project scope. Consequently, improvements to the Canal headgate are included with the design and construction project described for the diversion dam and fish screen. Therefore, no recommended improvements are included in this plan for the headgate.

Goals for improving and/or replacing this structure include:

- Gain hydraulic efficiency of the intake (which may decrease the amount of head necessary for the dam in order to take the full design flow at the intake),
- Improve flow control and allow provisions for headgate automation,
- Provide a structure with a new 50 year design life with decreased annual maintenance needs, and
- Address safety concerns for operation and maintenance of the facility.

An evaluation will be performed to consider the best value solution on a life cycle cost basis for the structure. Alternatives under consideration include retrofitting the existing structure, replacement of the existing structure, and constructing a new headgate integral to the fish screen at the Canal entrance to both control inflow to the Canal and help to provide a uniform flow distribution across the fish screen.

Lebanon WTP and Hydropower Intake (Sta 192+00)

Although this structure was not evaluated for its ability to pass a design flow rate of 310-cfs, its capacity is suspect based on visual observations in the field at flow rates substantially less than 310-cfs. $300,000 is included in this plan as an allowance to evaluate the capacity of this structure and make the required modification to achieve a capacity of 310-cfs. Improvements may include repairing the inoperable sluice gates or improving the concrete control structure. Because there is not an overflow channel upstream of the structure, improvements for this location will likely not include automated gates as recommended for other structures.

Mark's Slough Weir (Sta 253+00)

This structure diverts flow into Mark's Slough, a drainage channel leading to the South Santiam River. The addition of automated gates is recommended for this structure that would provide a means to release excess flow or stormwater into Mark's Slough.

Recommendations for improvements include general structural and safety inspections, and addition of automated control gates tied into a central flow monitoring and control system for the Canal. The number of gates should be developed based on the available excess capacity of Mark's Slough, and on the anticipated flood relief necessary in the Canal. A hydraulic study is recommended to both quantify the anticipated excess flow amounts, and to confirm hydraulic capacity downstream of the gates in Mark's Slough under high surface runoff conditions.

In order to estimate costs, the addition of three 9-foot wide gates was assumed. These gates could be retrofitted into the existing stop log guides. Additional improvements are recommended for personnel safety and access, along with any structural repairs noted during further analysis. The estimated cost of these improvements is $520,000, not including communications costs for an integrated system. Communication costs for all major flow control facilities are discussed in aggregate following the recommended improvements for individual control structures.

An alternate means to retrofit automated gates for this facility would be replacement of a section of the structure with a rubber dam or Obermeyer Weir type inflatable, overflow weir gate. Estimated costs for this alternate approach would be similar to the gate option.

Crown Zellerbach (CZ) Gates (Sta 271+00, also known as Lebanon Gates, and Skip Gates)

No improvements are proposed in this plan for the CZ Gates other than the potential to integrate the instrumentation and control of the existing gates into Albany's central control system. Albany and Lebanon would need to develop a cooperative management agreement to integrate these systems since this structure is owned by the City of Lebanon. Estimated costs to integrate the gate automation and controls are provided in the Instrumentation and Controls section.

CZ Tailrace Weir (Sta 280+00, at Hospital Slough)

Based on the ability to spill excess Canal flow into Hospital Slough, the addition of automated gates tied into a central Canal flow monitoring and control system is recommended for the CZ Tailrace Weir. Additional hydraulic analysis should be performed to determine available capacity and required relief conditions, and to confirm hydraulic capacity downstream in the Hospital Slough channel.

Structural and safety inspections and addition of two automated control gates are recommended for this facility. Final configuration and facility recommendations will also likely include improvements to personnel safety and access, general structural repair, and miscellaneous metalwork. Estimated project costs are $350,000, not including communications costs for an integrated system.

Albany Gates (Sta 287+00, also known as Hospital Gates, and the Cemetery Gates)

Operation of this facility can be automated with the addition of electric actuators on the two existing sluice gates, plus the addition of two additional automated sluice gates. In addition to a structural and safety inspection, a review of the existing mechanical equipment for both operational performance and compliance with a future Canal flow monitoring and gate automation system is recommended for the Albany Gates.

Allowances in the cost estimate were provided for structural upgrades, gate replacement, and safety improvements to meet current safety standards. Hydraulic and hydrologic analyses are recommended to better quantify anticipated design flows and spill capacity to the Crown Zellerbach tailrace (Hospital Slough) under high runoff conditions. The estimated cost of these recommendations is $480,000, not including communications and hydrologic analyses.

Cox Creek Diversion (Sta 538+00)

The addition of automated sluice gates tied into the Canal level monitoring and flow control system are recommended for this structure. These proposed gates could spill excess Canal flow into Cox Creek, and could be calibrated to provide an automated release of the minimum in-stream flows. The gates would be constructed in the side wall of the concrete channel conveying the Canal.

A hydraulic and hydrologic analysis should be performed on Cox Creek downstream of the release point to verify capacity to accommodate excess Canal flows during high runoff periods. A structural and safety inspection is also recommended for this facility prior to investing City funds into its improvement.

Cost estimates for improvements to this structure were based on the addition of two automated sluice gates with necessary structural retrofits, and an allowance for safety grating and structural upgrades to the facility. The estimated cost of these improvements is $400,000, not including communications costs for an integrated system.

Rock Check Dam (Sta 688+00)

The rock dam is a known constriction to Canal flows that could cause flooding to the surrounding lands during high runoff periods. The goal for improvements to this diversion is to mitigate potential Canal flooding by removing the rock dam while maintaining a functioning diversion to the Grand Prairie Water District.

One solution may be to remove the rock dam but maintain a gravity diversion through the addition of a gate control structure between the Canal and the Grand Prairie Water District's irrigation ditch. The gate could be automated and tied into a Canal level monitoring and control system. However, before this option is pursued a vertical control survey is needed.

An alternate solution would be to remove the rock dam and provide a pumped diversion for the water district. Additional study is recommended to determine whether a gravity diversion with automated gates is more economical than a pumped option over the life of the structure. Because this structure serves a private water district, the City should also explore cost sharing opportunities with the Grand Prairie Water District. A budget allowance of $200,000 is provided in this plan to accommodate construction of the control gate alternative or for removal of the dam and installation of a pump and related pipelines. The allowance does not account for communications costs for an integrated system.

Vine Street WTP Radial Gate (~Sta 962+00)

This gate was installed as part of the 1991 Vine Street WTP upgrade, and has no known deficiencies. Other than connecting the automated controls into a Canal flow control system, no improvements are recommended for this structure. Costs to integrate the recommended control system are discussed in the automation section.

Addition of a New Control Structure

This plan recommends the City construct at least one additional control structure, designed to provide flood relief from the Canal to other drainage channels. Locating a control structure close to the east end of the Urban Growth Boundary ***(Figure 7-1)*** would be a logical site, as it would allow for flood relief downstream of all the large inflow areas and before the Canal enters the City. Consideration should be given to locating the structure near Interstate-5 and diverting flow to Oak Creek. This structure would help reduce flooding in Albany during major storm events as observed during the floods of 1996. Evaluation of downstream impacts on Oak Creek should be completed before selecting this site as a diversion point.

A cost estimate of $700,000, not including communication costs for an integrated system, is identified to construct a new control gate within the Canal at the east end of the Urban Growth Boundary.

Instrumentation and Control for Canal Flow Control Structures

Instrumentation and controls will be a key component of regulating Canal flows in the future. Automation of gates is recommended for several of the control structures discussed above. Costs of mechanical gate actuators are included in the cost estimates for each control structure where applicable. Instrumentation and control costs referenced in this section include controller circuits, programming, radio antennas for a telemetry based system, and related components necessary to have a full and functioning telemetry control system. Addition of this type of system would allow all major structures to be controlled from a central location. Feasible locations for a central control facility would be either the Vine Street Water Treatment Plant, or the Scravel Hill Water Treatment Plant. By automating and centralizing the control of these structures, the City's ability to react and control emergency conditions would be significantly increased.

Currently, remote depth sensors are located in the Canal as listed in ***Table 7-2***, in addition to data provided by the USGS stream gage located at the old headgates. This plan recommends developing flow-rating curves for the existing gages, and transmitting this data to a central control facility. Correlating water surface elevations to flows will help determine the Canal's sensitivity to adjustments made at each control structure and assist in investigations to identify and eliminate lateral inflows.

The estimated cost to utilize the existing sensors, provide transmission of signals to the Vine Street WTP via radio telemetry using the City's existing 800 MHz licensed control system and connect automated gates at each facility is summarized in ***Table 7-8***.

Table 7-8: Cost Estimate for Automation of Canal Flow Control Structures

Automate Control Structures	*Cost*
Equipment and Installation (for 9 Sites)	$300,000
Master Station (equipment & installation)	$50,000
Develop Rating Curves (9 sites)	$100,000
Total	***$450,000***

Hydraulic Analysis Allowance for Receiving Drainage Channels

The discussion above included recommendations to modify several of the control structures to divert excess Canal flow into historic drainage channels. However, there is not any recent data available regarding the capacity of the proposed receiving channels and there is concern that they may be capacity limited during high flow periods. This plan recommends the City further analyze these drainage courses prior to modifying any control gates. A $500,000 budget allowance is recommended to address initial analyses.

Flow Augmentation

Another goal identified during the Canal analysis was to convert the City's surplus hydropower water right for use in augmenting urban stream flows for fish and other beneficial uses. The City has submitted a transfer application to the Water Resource Department to allocate the excess hydro water rights to the six streams listed in ***Table 7-9***. Diversion locations are shown in ***Figure 7-1*** at the approximate stations shown in ***Table 7-9***. An allowance of $100,000 is provided in this plan to configure manual control release facilities at each of the identified sites.

Table 7-9: Potential Sites for Flow Augmentation

Station	***Name / Drainage Course***	***Flow Amount (cfs)***
423+00	Burkhart Creek	15
538+00	Cox Creek	20
740+00	Periwinkle Creek	20
838+00	Oak Creek	15
924+00	Cathey Canal	13
950+00	8th Avenue Canal	2
	Total	***85***

Ensure Canal Capacity (Project Number C2)

Based on the analysis of bridge, culvert, and sedimentation issues provided in the Inspection and Analysis Results section, the following recommendations are offered to ensure the Canal will be able to convey the 310-cfs design flow.

Recommended Bridge Improvements

Based on the hydraulic analysis used to screen the bridge inventory for potentially undersized bridges, additional study is recommended for the facilities discussed below. Cost estimate allowances are provided for each facility based on improvements that may be necessary to achieve a capacity of 310-cfs and MWH's experience with similar improvement projects. All costs should be re-examined based on additional study.

<u>Sta 117+00: Private Driveway Bridge</u>

Based on the preliminary analysis presented earlier in this chapter, the private driveway bridge located at Station 117+00 is potentially under capacity. A cost allowance of $100,000 is recommended to provide a new bridge if necessary.

<u>Sta 137+00: Franklin Street Bridge</u>

The Franklin Street Bridge is a concrete structure with deep girders. In order to minimize disruption to the road's grade, a $300,000 cost allowance to construct a new, thinner profile bridge is recommended.

Recommended Culvert Improvements

Based on the hydraulic screening presented earlier in this chapter one culvert may cause a restriction at 310-cfs.

Sta 455+00: KGAL Road Culvert
For the KGAL culvert, a required surcharge of 1.5-ft was calculated to pass the 310-cfs design flow. However, this culvert has very limited freeboard. A cost allowance of $300,000 is recommended to replace this culvert with a bigger structure, likely a concrete box culvert.

Recommended Sediment Removal to Achieve 310-cfs Capacity
Removal of sediment in the Canal will be required to achieve the 310-cfs design flow. In addition to permitting requirements and property access issues, a detailed analysis of the quantity, location, methods of sediment removal, methods of treatment, and location of sediment disposal sites, is outside the scope of this plan. However, the previous 2000 Harza

Cost Analysis Study addressed[61] these issues. Therefore, as described earlier in this plan, an allowance of $1,500,000 has been recommended based on the 2000 Harza Cost Analysis Study[62] and results of the field inspection. This figure is intended only to obtain the initial 310-cfs flow. Local sediment removal issues are discussed further under Channel Restoration.

Raise Canal Banks
The KCM study used to evaluate bridges and culverts west of the Interstate-5 crossing stated that flooding would likely occur between 7th Avenue and Pacific Boulevard, downstream of the railroad bridge and around the trash rack at a 310-cfs flow rate. The study further stated that to increase the flow capacity in areas where the Canal cannot convey 310-cfs, water surface elevation must be lowered, bank elevations must be raised, or a combination of the two. A budget allowance of $400,000 has been included in this plan to raise Canal banks.

Lateral Inflow Reduction
As estimated in the Inspection and Analysis Results section of this chapter, approximately 1,600 acres drain into the Canal based on our very rough assessment. Drainage from this area creates a flooding risk during periods of high runoff. Removal of lateral inflows will require diversion of ditch/culvert inflows to cross-over culverts to re-establish historic drainage patterns. This will require separation of the City of Lebanon's stormwater system from the Canal and will be difficult in other areas due to topography and upstream conditions.

In addition to reducing flooding risks, elimination of these inflows will reduce sedimentation problems in the Canal by removing sediment sources. Sediment reduction will help maintain Canal capacity and improve water quality. Any re-routing of lateral in-flows would have to weigh Canal benefits against potential upstream and downstream impacts on the drainage way. Hydraulic analysis should be completed to ensure that flooding problems are not being created by re-routing storm runoff. A budget allowance of $300,000 has been included in this plan for the City to continue their work on removal of lateral inflows.

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[61] Harza Engineering Company, *Albany Hydroelectric Project Development Cost Analysis Study*, A FERC Minor License, FERC Project No. 11509-000, April 17, 2000.
[62] Harza Engineering Company, *Albany Hydroelectric Project Development Cost Analysis Study*, A FERC Minor License, FERC Project No. 11509-000, April 17, 2000.

Channel Restoration (Project Number C3)

Channel restoration work is recommended to rehabilitate the Canal and protect water quality. Items addressed for this category include local sediment removal, debris removal, bank repair, excess vegetation and fallen tree removal to maintain capacity, and water quality issues.

Removal of local sediment deposits was only considered necessary if they were caused by dumped or accumulated debris, or by local erosion and subsequent sediment deposition. Removal of sediment naturally deposited due to meanders and weak slumping soils is not generally recommended.

Water quality issues identified under this category include further investigating the effects of Cheadle Lake on Canal water quality. A flow net analysis is recommended to assist in estimating seepage into the Canal from Cheadle Lake to help determine if further investigation is warranted.

An allowance of $1,000,000 is provided for channel restoration and ensuring water quality based on an evaluation of needs identified in the Canal inventory for local sediment removal, debris removal, local bank repair, tree removal, and preliminary investigation into potential water quality issues from Cheadle Lake.

Improve Canal Access (Project Number C4)

Actions recommended under this category include removing excessive bank vegetation and securing legal and physical access along the Canal where practical, such as commissioning a right-of-way survey and removing right-of-way encroachments. A cost allowance of $500,000 is proposed to begin the process of improving Canal access.

SUMMARY

Table 7-10 presents a summary of recommended Canal improvement projects. Engineering estimates of probable cost include a 20 percent contingency allowance, and a 15 percent allowance for engineering/administration fees. The methodology used to prepare cost estimates is described in *Chapter 11 - Basis of Cost Estimates*. Staging of improvements is discussed in *Chapter 12 - Recommended Plan*.

Table 7-10: Recommended Canal Improvement Projects

Project Description		*Cost Estimate*
Update Control Structures (Project Number C1)		
Lebanon WTP and Hydropower Intake (Station 192+00)		$300,000
Mark's Slough (Station 253+00)		$520,000
CZ Tailrace (Hospital Slough, Station 280+00)		$350,000
Albany Gates (Station 287+00)		$480,000
Cox Creek (Station 538+00)		$400,000
Rock Dam and Siphon (Station 688+00)		$200,000
New Control Gate (Oak Creek, Station 755+00)		$700,000
Communication for all Structures		$300,000
Master Station		$50,000
Develop Rating Curves for Remote Sites		$100,000
Hydraulic Analysis Allowance for Receiving Drainage Channels		$500,000
Flow Augmentation Allowance		$100,000
	Sub-Total	*$4,000,000*
Ensure Canal Capacity (Project Number C2)		
Private Driveway Bridge (Station 117+00)		$100,000
Franklin Street Bridge (Station 137+00)		$300,000
KGAL Road Culvert (Station 455+00)		$300,000
Sediment Removal		$1,500,000
Raise Canal Banks		$400,000
Lateral Inflow Removal		$300,000
	Sub-Total	*$2,900,000*
Channel Restoration (Project Number C3)		
Allowance to Repair Bank Damage, Remove Debris and Excess Bank Vegetation, Complete preliminary Cheadle Lake Seepage Analysis		$1,000,000
	Sub-Total	*$1,000,000*
Improve Canal Access (Project Number C4)		
Allowance for Removing Encroachments, Securing ROW, and Removing Heavy Bank Vegetation		$500,000
	Sub-Total	*$500,000*
	Total	***$8,400,000***

CHAPTER 8 – VINE STREET WATER TREATMENT PLANT

INTRODUCTION

This section summarizes the evaluations, conclusions and recommendations for the Vine Street Water Treatment Plant (WTP) based on a historical performance evaluation, a capacity audit (both hydraulic and process) and a facilities review. More detailed information used to complete the evaluations and develop recommended improvements can be found in ***Appendix C***. A list of recommended improvements including estimated costs are summarized at the end of this chapter. Recommended staging for these improvements is included in *Chapter 12 - Recommended Plan*.

HISTORY AND ROLE OF THE VINE STREET WTP

The Vine Street WTP is located at 300 Vine Street. The WTP was built in 1912 as part of a water supply and hydroelectric power system. A brief history of the plant's improvements is shown below:

- 1912 – Vine Street WTP constructed, including two settling basin, six rapid sand filters (filters 1-6);
- Late 1940's – Raw water pumps and a flocculator-clarifier (Accelator No.1) were added;
- Mid 1960's – One of the original sedimentation basins was converted into two rapid sand filters (Filters 7 and 8);
- Mid 1970's – Backwash ponds were added;
- Late 1970's – A settling tank (contact basin) was added;
- 1984 – The City of Albany purchased the water system from Pacific Power and Light. This purchase included the Vine Street WTP, the Santiam–Albany Canal, hydropower facilities, pump stations, reservoirs and pipelines;
- 1991 - Albany completed an expansion and upgrade of the Vine Street WTP. This expansion added two additional filters (filters 9 and 10), converted the solids contact basin to a clarifier (Accelator No. 2) and other improvements to ensure continued regulatory compliance and improved system reliability, and
- 1995 – Albany upgraded and expanded backwash holding lagoons and sludge drying beds for improved solids handling and to meet regulatory requirements.

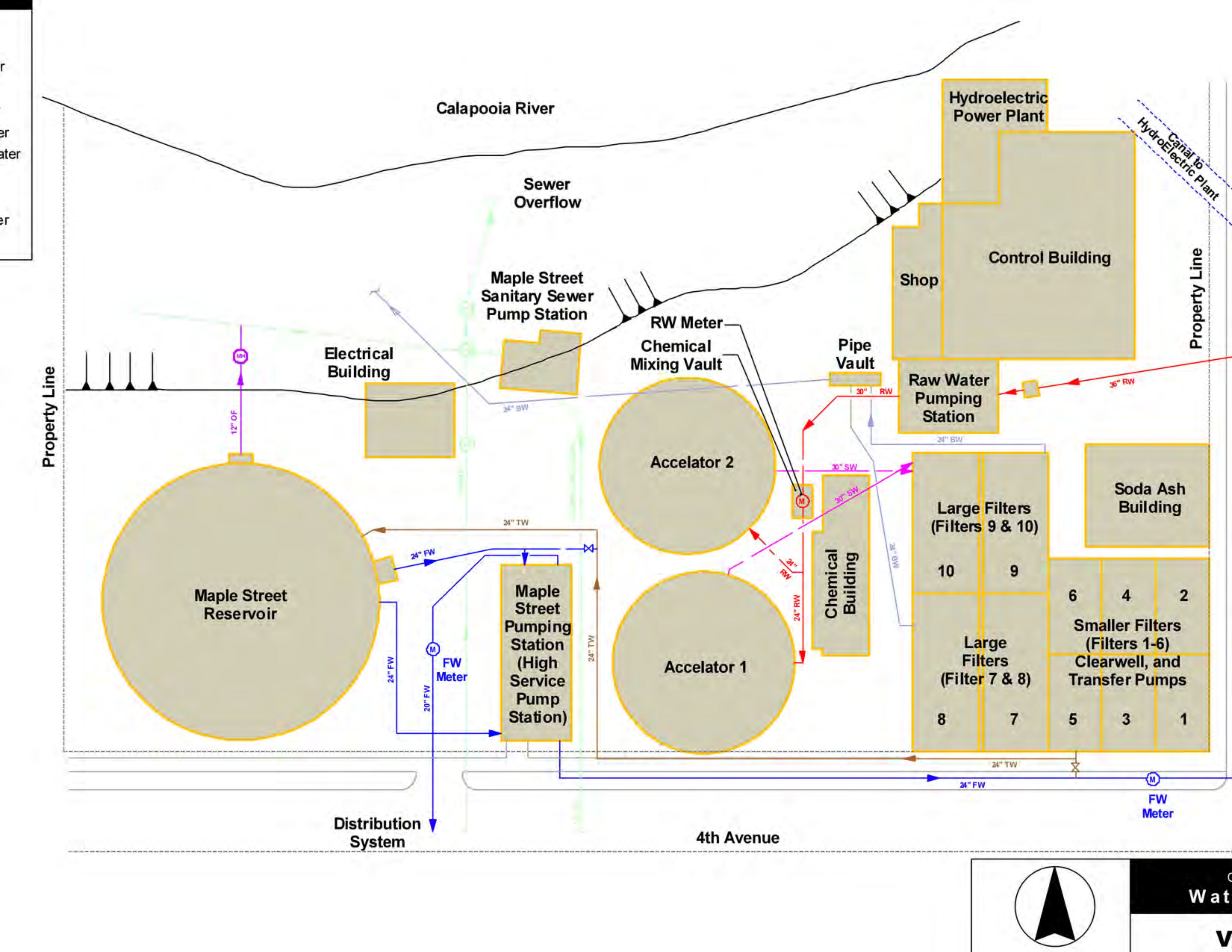
The Vine Street WTP is currently Albany's sole source of water. Historically, the maximum day flow produced by the plant has been approximately 16 MGD. A maximum day demand of 17 MGD was reported in July 1998, but is not considered a reliable peak production value because of errors in metering finished water production. ***Figure 8-1*** is a plan-view layout of the WTP in its current configuration, and ***Figure 8-2*** is a process flow schematic of the plant indicating key processes, chemical addition points and sample locations.

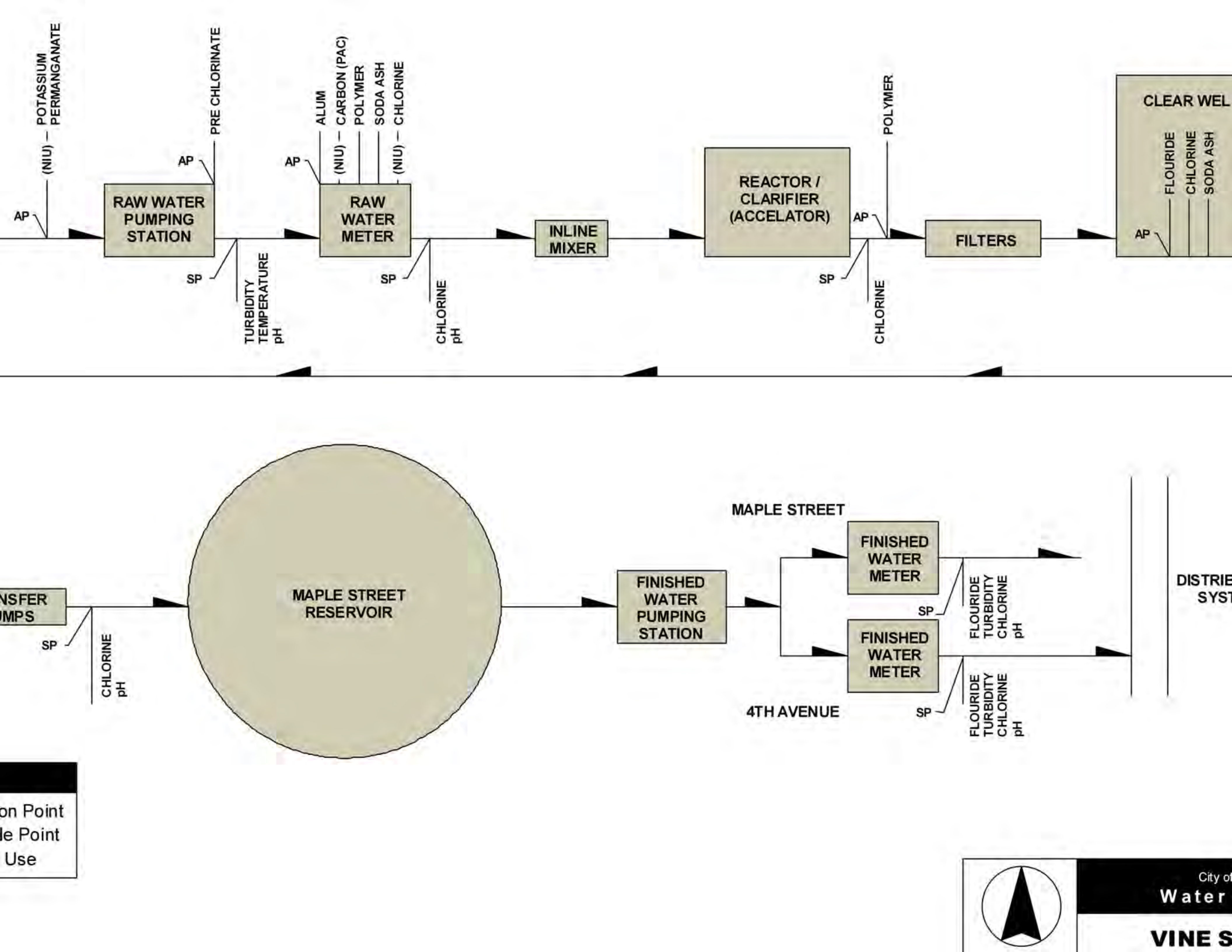
In 2006, a new 12 MGD Joint Water Project (JWP) developed as a cooperative endeavor by the Cities of Albany and Millersburg, is scheduled to come on-line. This project includes construction of a new WTP, the Scravel Hill WTP, supplied by the Santiam River (see *Chapter 9 - Joint Water Project*). Albany will be allocated 10 MGD of the Scravel Hill WTP capacity with Millersburg receiving the remaining 2 MGD capacity. Following construction of the JWP, the Vine Street WTP will, initially, play a lesser role in meeting the City's water needs and the Scravel Hill WTP will become the City's primary water source under normal operating conditions. As population and demands grow, the Vine Street WTP will need to be operated more frequently to meet these demands. At full development of the urban growth boundary (UGB), Albany will rely on the Vine Street and Scravel Hill plants to provide a capacity of 20 MGD each to meet a projected maximum day demand of 40 MGD.

RAW WATER QUALITY

Raw water enters the Vine Street WTP from the Santiam-Albany Canal (Canal). The Canal originates from a diversion along the South Santiam River and traverses approximately 18 miles through the City of Lebanon, unincorporated parts of Linn County and the City of Albany to the Vine Street WTP. The Canal was first used as a drinking water source in the 1880's and as a hydroelectric generation source in 1881[63]. The City of Lebanon also uses water from the Canal as a potable water supply, and several property owners along the Canal have South Santiam River water rights, primarily for irrigation purposes. The Canal terminates via a discharge to the Calapooia River adjacent to the WTP. The Canal is discussed in detail in *Chapter 7 - Canal Evaluation*.

[63] Albany-Santiam Canal Historic Document Record, Archeological Investigations NW, May 6, 1999.


Calapooia River
Sewer Overflow
Maple Street Sanitary Sewer Pump Station
Electrical Building
Hydroelectric Power Plant
Control Building
Shop
Canal to HydroElectric Plant
Property Line
Raw Water Pumping Station
36" RW
30" RW
Pipe Vault
RW Meter
Chemical Mixing Vault
Accelator 2
Accelator 1
Chemical Building
24" RW
30" SW
24" BW
Soda Ash Building
Large Filters (Filters 9 & 10)
10
9
Large Filters (Filter 7 & 8)
8
7
Smaller Filters (Filters 1-6) Clearwell, and Transfer Pumps
6
4
2
5
3
1
24" TW
24" FW
FW Meter
Maple Street Pumping Station (High Service Pump Station)
20" FW
Maple Street Reservoir
12" OF
Distribution System
4th Avenue
Wat



AP
(NIU) — POTASSIUM PERMANGANATE
RAW WATER PUMPING STATION
AP
PRE CHLORINATE
SP
TURBIDITY
TEMPERATURE
pH
AP
ALUM
(NIU) — CARBON (PAC)
POLYMER
SODA ASH
(NIU) — CHLORINE
RAW WATER METER
SP
CHLORINE
pH
INLINE MIXER
REACTOR / CLARIFIER (ACCELATOR)
AP
POLYMER
SP
CHLORINE
FILTERS
CLEAR WEL
AP
FLOURIDE
CHLORINE
SODA ASH
NSFER
UMPS
SP
CHLORINE
pH
MAPLE STREET RESERVOIR
FINISHED WATER PUMPING STATION
MAPLE STREET
FINISHED WATER METER
SP
FLOURIDE
TURBIDITY
CHLORINE
pH
FINISHED WATER METER
4TH AVENUE
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FLOURIDE
TURBIDITY
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DISTRIE
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The Vine Street WTP has historically produced treated water that meets all drinking water regulations. However, the Canal flows through areas subject to potential contamination; there are a number of locations along the Canal where contaminants and urban runoff from developed areas can be introduced, especially during rainfall and runoff events. The City routinely monitors the quality of raw water by testing for potential contaminants and is capable of removing nearly all contaminants through the treatment process. This raw water monitoring has not found significant levels of contaminants. Due to urban stormwater runoff and drainage of farm fields adjacent to the Canal, water quality at the Vine Street WTP may be less than the water quality in the South Santiam River at the Canal's diversion point.

Raw water quality data between 1994 and 2000 were analyzed for five key parameters:

- Turbidity,
- Temperature,
- pH,
- Alkalinity, and
- Organic content.

These parameters are typically of most importance when evaluating a treatment plant's overall performance. A summary of findings for these parameters is shown below and a more detailed analysis of these parameters is presented in ***Appendix C***.

Turbidity

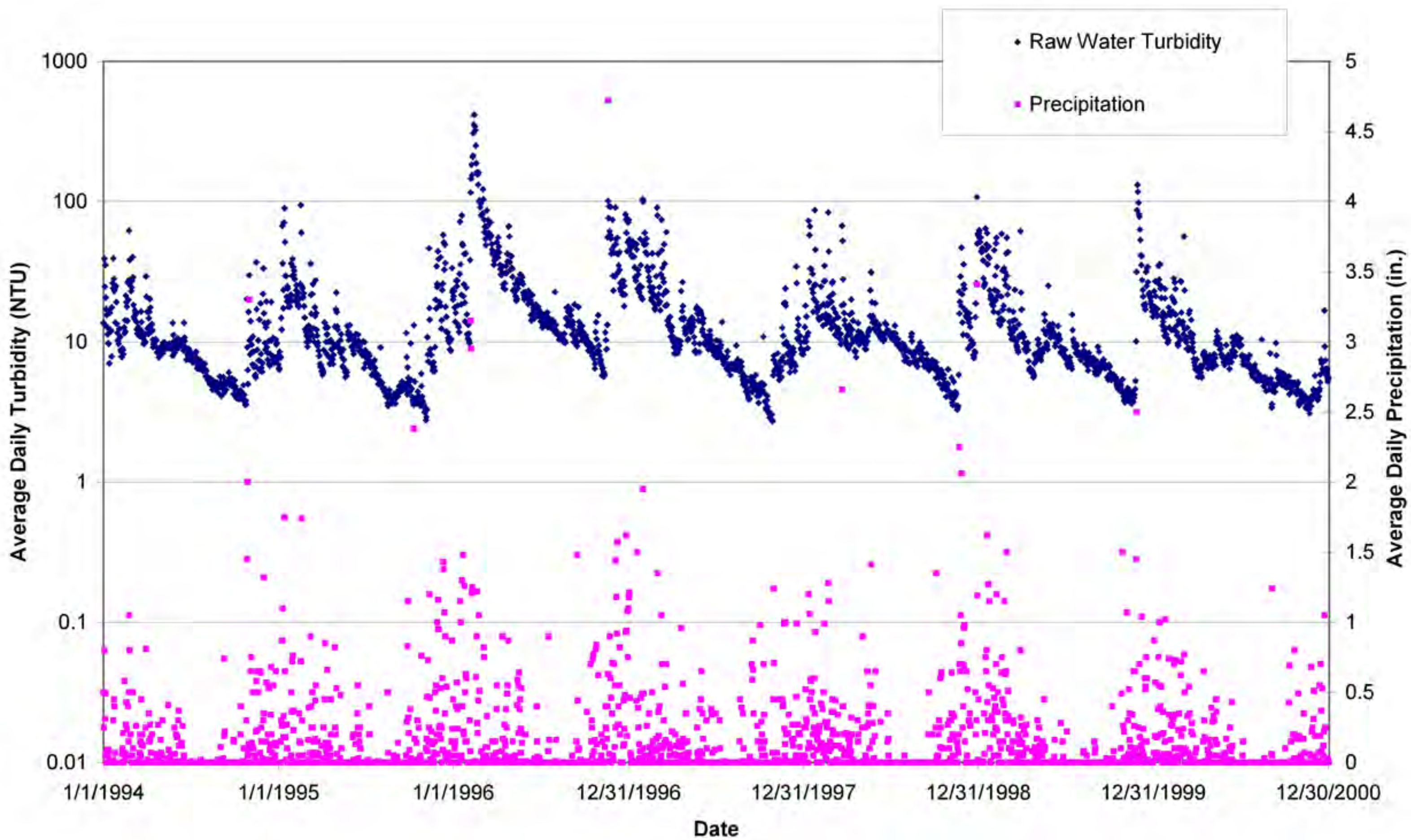
Raw water turbidity is a direct measurement of particulate matter, including bacteria and viruses and is probably the single most important water quality parameter considered when evaluating the treatability of water. Turbidity, reported in nephelometric turbidity units (NTU), is measured by passing visible light through a water sample. The percent of light passing through the sample is related to the water's turbidity. Lower turbidity water is typically easier to treat and usually requires lower chemical doses for optimum coagulation and filtration.

The raw water turbidity from the Canal has historically been low and moderately variable during the majority of the year. High rainfall events generally correspond to an increase in Canal turbidity. The lowest turbidity periods occur during the warmer, drier months and the highest turbidity levels during periods of wet weather. Between 1994 and 2000, turbidities averaged less than 20 NTU from May to October and minimum turbidities were as low as 3 NTU. Between September and April, turbidities averaged 60 NTU. Plant staff has recorded instantaneous peak turbidities as high as 800 NTU during particularly harsh winter storms; however these events are short-lived and rare. The highest average daily turbidity was recorded in 1996 as 414 NTU. ***Figure 8-3*** presents the average daily raw water turbidity, as well as the observed daily precipitation from 1994 to 2000.

Temperature

Raw water temperature also plays an important role in water treatment because it affects the rate of chemical reactions, floc settling and filter performance. Higher temperature water typically requires lower chemical doses and offers better floc formation, settling and filtration characteristics.

Figure 8-3: Albany WTP: 1994-2000 – Average Daily Plant Turbidity and Precipitation



During the seven-year period of record (1994 through 2000), wintertime low average temperatures were approximately 45°F (7.2°C) and summertime highs average temperatures were approximately 65°F (18.3°C). The lowest observed temperature was 34.2 °F (1.2 °C) in December 1998. The highest measured temperature was 69.3 °F (20.7 °C), in July 1998.

pH

The pH of the raw water from the Canal typically varies between 6.5 and 8.0 throughout the year with normal average values between 7.0 and 7.5. pH is important in water treatment because of its impact on coagulation performance and also on the corrosive characteristics of the water supply. A pH in the range of 6.5 to 7.0 is typically considered optimum for alum coagulation. pH values in excess of 7.0 are considered optimum for overall corrosion control. Corrosion control typically governs the pH, with some sacrifice in coagulation performance.

Historically, pH peaks in late summer/early fall, possibly due to increased microbiological activity in the Canal. As dissolved oxygen concentrations rise and carbon dioxide levels decrease with photosynthesis, pH will become less acidic. Historic minimums occur in the winter months, presumably due to heavy rainfall events. The most acidic raw water pH was 6.63 in November 1994. The least acidic pH was 7.8 in early September 1996.

Alkalinity

Alkalinity is important in water treatment because of its impact on coagulation performance as well as its impact on corrosivity and pH stability. Alkalinities above 20 mg/L as $CaCO_3$ are considered adequate for alum coagulation and also offer improved pH stability in the distribution system.

Alkalinity is not measured regularly at the Vine Street WTP; however, data was collected from 1994 to 2000. Alkalinity in the Santiam-Albany Canal typically ranges from 18 to 24 mg/L as $CaCO_3$. Raw water alkalinity has not been measured with enough frequency to establish seasonal alkalinity trends. Increased monitoring of alkalinity is recommended for improved process control.

Organic Content

The concentration of organic matter in the raw water can affect its treatability. It can also influence chlorine demand and the creation of disinfection by-product (DBP). As the concentration of organic matter in the water increases, the required alum dose and the required chlorine dose usually increase as well. In addition, higher concentrations of organic matter usually result in higher levels of DBPs in the distribution system; the natural organic matter reacts with free chlorine to form the harmful by-products.

Total Organic Carbon (TOC) is a general measure of the organic matter present in the raw water. This parameter is sometimes used as an indicator of Trihalomethane (THM) and Haloacetic Acid (HAA) formation potential. TOC is also important as existing regulations intended to minimize DBP formation require a percentage removal of TOC depending on the raw water TOC concentration (see TOC discussion in *Chapter 6 - Water System Regulatory Review* for discussion).

Table 8-1 presents the results of the recent TOC sampling at the Vine Street WTP. The sample collected in October, 2002 was not included as part the following analysis, as it was shown to be a statistical "outlier" (at 95-percent confidence interval), and likely the result of sampling error. The data suggests that the TOC concentration in the raw water is relatively low compared to other U.S. water supplies, and comparable to other Pacific Northwest supplies. Results from this analysis indicate TOC removal efficiencies between 42 and 73 percent, and averaging 55-percent over the entire sampling period. During December 2002 and January 2003, when raw water TOC levels exceeded 2.0 mg/L (requiring enhanced coagulation for at least 35-percent TOC removal), TOC removal efficiencies averaged 64-percent. More data is required to understand the seasonal variability and the Vine Street Plant's TOC removal efficiency; Albany should continue to monitor its raw and finished water TOC to demonstrate continued compliance with TOC regulations at the plant.

Table 8-1: Raw Water TOC Sampling Results

Sample Date	*Raw Water TOC (mg/L)*	*Finished Water TOC (mg/L)*	*Removal Efficiency (%)*
February 5, 2002	1.20	0.70	42%
March 5, 2002	1.20	0.70	42%
April 9, 2002	1.48	0.55	63%
May 7, 2002	1.21	0.54	55%
June 4, 2002	1.28	0.60	53%
July 16, 2002	1.19	0.66	45%
August 13, 2002	1.58	0.42	73%
September 10, 2002	0.75	0.37	51%
October 8, 2002 [1]	0.87	0.87	NA
November 26, 2002	1.35	0.61	55%
December 17, 2002	3.28	1.10	66%
January 14, 2003	2.43	0.92	62%
		Average removal efficiency	***55%***

[1] October 8, 2002 test is not included based on intermediate removal percentages

Capacity Evaluation

The capacity of the Vine Street WTP was evaluated from two perspectives; hydraulic and process capacity. Each treatment train/support system has its own process capacity relative to certain design or operating criteria/parameters and may be independent of other unit processes. The hydraulic capacity is related to the piping and flow control systems that limit the ability of the water to flow through the interconnected system. Both capacities are discussed below and in more detail in ***Appendix C.*** The analysis concludes that the plant's firm capacity is approximately 16 MGD.

Hydraulic Capacity

The raw water pump station, individual treatment processes and the high pressure pump station were designed for, and are capable of handling an ultimate capacity of 20 MGD. To understand and analyze the hydraulics associated with the plant piping, a hydraulic computer model was created to identify potential capacity-limiting pipes within the WTP. This hydraulic model is separate from the hydraulic model of the distribution system and was developed to identify restrictions in unit processes and piping within the Vine Street WTP. In order to fully develop the model and calibrate its accuracy, a full-scale hydraulic "stress" test was conducted on August 15, 2001, at incremental flows from 16 to 20 MGD. Key hydraulic and water level parameters were measured during the test to calibrate the model. ***Figure 8-4*** shows the individual components considered for the model.

The analysis identified a "hydraulic bottleneck" effectively limiting the plant's hydraulic capacity to 16 MGD. This hydraulic bottleneck results from a combination of two constraints at the WTP: air entrainment between the accelators (clarifiers) and the filters, and the pipe/open channel configuration between filters 1-6 and filters 7-10 (see ***Figure 8-4***). This bottleneck currently limits the flow to the six smaller filters, (resulting in approximately 2,400 gpm (3.5 MGD) reduction in capacity). Although the Vine Street WTP can produce up to 20 MGD for short periods of time, this rate cannot be sustained because of the bottle neck. To achieve a sustained 20 MGD plant capacity, this hydraulic bottleneck must be eliminated. More detailed discussion of the hydraulic model developed for the WTP is included in ***Appendix C***.

Process Capacity

Each of the key treatment plant processes were evaluated for their ability to meet current and possible future conditions. This evaluation is based on past performance of the Vine Street WTP and observations made at similar plants.

The chemical feed systems have performed reliably and with few problems since the 1991 plant expansion and upgrade. They have adequate capacity for the 20 MGD design plant capacity and for severe water quality events that may occur during the winter months. It is expected that these systems will continue to provide reliable service with routine maintenance.

The clarifiers (accelators) have a total design process capacity of 16.5 MGD (with an overflow rate at approximately 2.5 gpm/sf). Accelator performance at 20 MGD was observed during the plant stress test in August 2001. At 20 MGD (with overflow rates at or above 3.0 gpm/sf), water quality performance was slightly compromised. Installation of settled water turbidity monitors on the effluent of each accelator is recommended to improve process control and to assist in establishing "baseline" performance data. If additional treatment efficiency is required as demands increase at the Vine Street WTP, the City may want to investigate the use of alternative coagulants such as cationic polymer coagulant aids and proprietary alternative alum coagulants to optimize coagulation and clarification. These alternative coagulants would also reduce soda ash usage and solids production.

The filters have been able to produce excellent filtered water quality under all seasonal and flow conditions experienced to date. The smaller filters (filters 1-6) experience some hydraulic and operating performance limitations during the high flow periods each summer. As previously discussed, these limitations result from a hydraulic bottleneck in the settled water piping system that limits the maximum flow that can be delivered. Also, the smaller filters have a lower submergence (< 5 feet) over the top of the filter media and experience air binding problems when the headloss increases, thereby shortening filter runs below recommended minimum lengths during these periods. While the combined rated capacity of the six smaller filters is approximately 6,000 to 6,300 gpm (8.6 to 9.1 MGD at 5 gpm/sf maximum filtration rate), the maximum output of these filters is approximately 3,600 to 3,900 gpm (5.2 to 5.6 MGD), resulting in a 3.0 to 3.5 MGD reduction in overall plant capacity. Even if the hydraulic bottleneck to the six small filters is eliminated, it is unlikely that the capacity of these filters can be further increased unless the air binding problem, discussed below, is addressed.

To address the air binding problems in the smaller filters, media submergence must be increased. This can be achieved by rebuilding the filter underdrains using a "gravel-less" underdrain system to lower the top of the media by at least one foot or more. Also, changes in the media configuration are recommended to reduce the headloss development rate in the upper portion of the media.

It is important that the filter to waste (FTW) system be capable of passing the filter's maximum flow rate to maintain a relatively constant flow through the media and minimize the potential for filter breakthrough and loss of media. The existing FTW piping for the six small filters is only 4-inch diameter, significantly smaller than the filter effluent piping. It is highly unlikely that these pipes can carry the current maximum flow (600 gpm) for each filter and maintain a "seamless" transition from FTW to filtration mode. Nor will they be capable of carrying the maximum filter flow (1,000 gpm) anticipated following the removal of the hydraulic bottleneck. Thus, the FTW piping systems for each of the six small filters should be upsized to accommodate the ultimate maximum filter flow.

Automatic backwash improvements are also recommended to optimize performance and to reduce risk of damage to the filter underdrains. This requires improvements to the instrumentation and control logic at the WTP. In addition, the current backwash pump configuration does not provide adequate redundancy. A new backwash pump with variable frequency drive should be installed to replace the smaller constant speed backwash pump. This pump will provide redundancy for both the large and small filter backwash systems. These improvements are included in the recommend improvements section of this chapter.

The Vine Street WTP's residual solids are received, handled and treated in a series of lagoons and drying beds located on a plateau below the plant. The existing lagoons and drying beds will require some improvements to ensure long-term performance. These improvements will help maintain compliance with the National Pollutant Discharge Elimination System (NPDES) permit (discussed in *Chapter 6 – Water System Regulatory Review*). Improvements include reshaping and relining the existing ponds and installation of new sumps in the two dewatering lagoons. When the plant is used for summer "peaking" only, minimizing the on-site solids inventory is recommended. Alternative solids handling strategies, including discharging to the wastewater treatment plant (WWTP) were investigated, but routine discharge to the WWTP was not found to be economically feasible given wet weather capacity of the sewer system, relatively high solids loading and correspondingly high connection fee and sewer service charge.

pipelines (Points 1 & 2). At increased flows (> 16MGD), the air is unable to escape via the existing air-release valve (Point 3), and the existing overflow pipeline (Point 4). The air accumulates in the filter header, limiting the capacity of this pipeline to provide sufficient flow to Filters 1-6. This problem is accentuated by the pipe reduction / open channel configuration between Filters 7-10 and Filters 1-6 (Point 5).

PIPE VAULT

Accelator 1 to Combination
- #1 30-inch pipe entrance
- #2 Pipeline
- #3 90-deg bend
- #4 Pipeline
- #5 45-deg bend
- #6 Pipeline
- #7 30-inch BFV
- #8 Pipeline
- #9 90-deg bend
- #10 Pipeline
- #11 30x36x36-inch Tee (branch)

Accelator 2 to Combination
- #12 30-inch pipe entrance
- #13 Pipeline
- #14 90-deg bend
- #15 Pipeline
- #16 30-inch BFV
- #17 5-deg bend
- #18 Pipeline
- #19 90-deg bend
- #20 Pipeline
- #21 30x36x36-inch Tee (branch)

SW Combining to Overflow
- #22 30x36x36-inch Tee (thru)
- #23 Pipeline
- #24 30x36x36-inch Tee (thru)
- #25 Pipeline
- #26 45-deg bend
- #27 Pipeline
- #28 45-deg bend
- #29 Pipeline
- #30 36x36x36-inch Tee (branch)
- #31 Pipeline
- #32 36x36x36-inch Tee (branch)

Overflow through "New" Filter Pipe Gallery
- #33 Pipeline
- #34 36x20x20 Cross (delivery to F8 & F10-branch)
- #35 Pipeline
- #36 36x20x20 Cross (delivery to F7 & F9-branch)

Piping - Filters 7 -

Filters 7 & 8
- #37 36x20x20
- #38 20-inch FV
- #39 Pipeline (e
- #40 20-inch pi

Filters 9 & 10
- #41 36x20x20
- #42 20-inch FV
- #43 Pipeline (e
- #44 20-inch pi

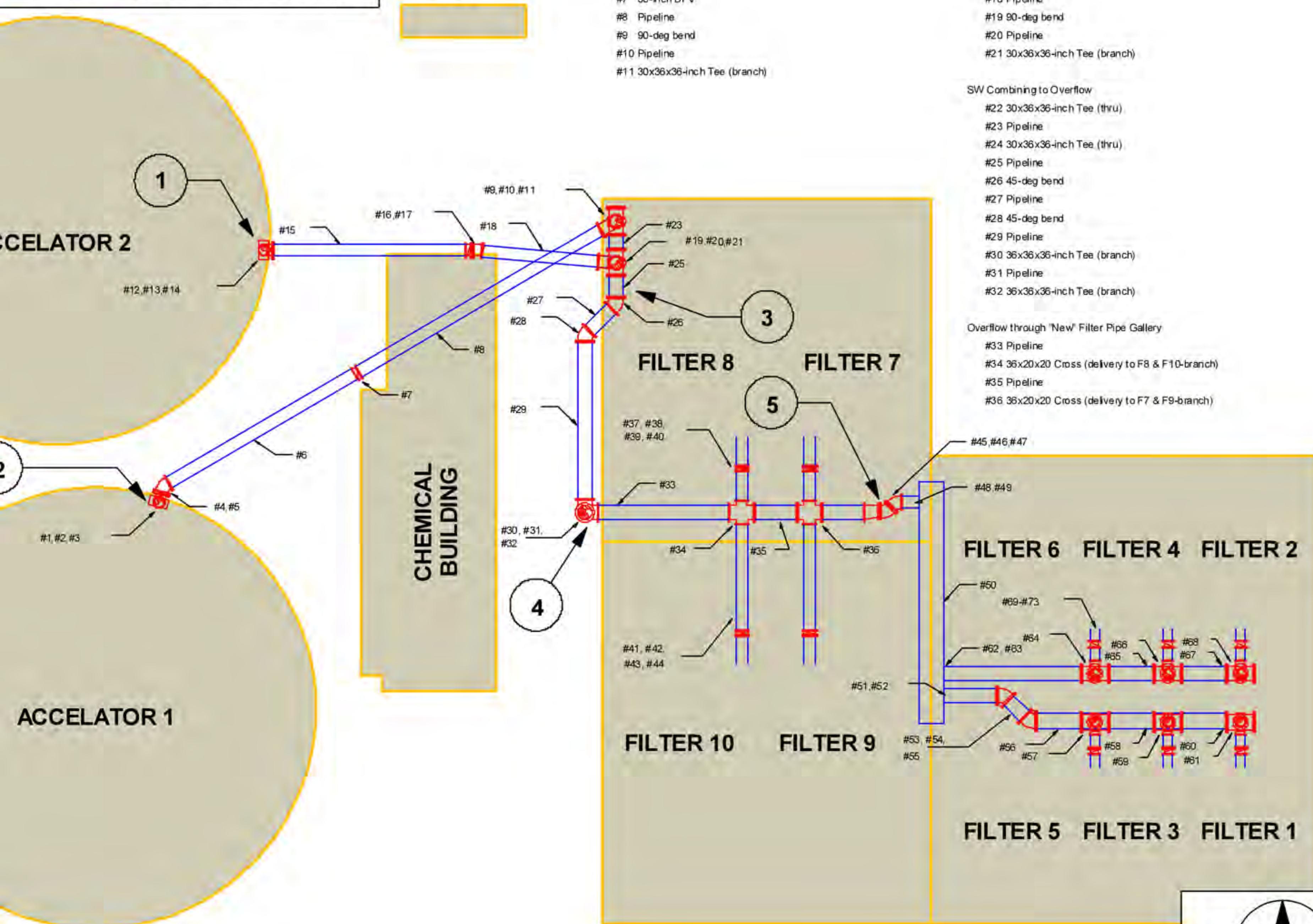
Filter Wat

"New" Filter Pipe G
- #45 36 to 24-in
- #46 15-deg be
- #47 15-deg be
- #48 Pipeline
- #49 24-inch pi
- #50 Channel

- #51 18-inch pi
- #52 Pipeline
- #53 45-deg be
- #54 Pipeline
- #55 45-deg be
- #56 Pipeline
- #57 18x18x10
- #58 Pipeline
- #59 18x18x10
- #60 Pipeline
- #61 18x18x10

- #62 18-inch pi
- #63 Pipeline
- #64 18x18x10
- #65 Pipeline
- #66 18x18x10
- #67 Pipeline
- #68 18x18x10

Filters 1 - 6 (each)
- #69 18x18x10
- #70 Pipeline
- #71 10-inch BF
- #72 Pipeline
- #73 90-deg be
- #74 Pipeline
- #75 10-inch pi



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RECOMMENDED IMPROVEMENTS

Recommended improvements to the Vine Street are based on a review of the reliability, useful life, and repair and replacement needs for the plant's treatment/support processes. A summary of existing and future needs is presented below. Recommended improvements listed below consider the plant's needs within the 2025 planning window.

Instrumentation and Control Improvements (Project Numbers WTP01, 10, 11, 18 and 24)

Five instrumentation and control related improvements are recommended as part of this facility plan. Additional and upgraded monitoring equipment is recommended to assist WTP operators assess and respond to water quality conditions and treatment efficiency. Recommended monitoring equipment includes instrumentation for settled water turbidity, filtered water particle counters and TOC analysis.

Progressive Software Solutions (PS2) performed an audit of the Vine Street WTP's instrumentation and control systems (*City of Albany Water Plant Automation Upgrade Study, 2002*). PS2 recommended that the City replace existing plant control hardware with a new control system. This includes preliminary design, upgrades for both the existing "old" and "new" style remote telemetry units (RTUs) for the distribution system as well as replacement of the main plant programmable logic control (PLC) and upgrades to the system control and data acquisition (SCADA) system at the WTP.

As technology evolves, the SCADA system at the Vine Street WTP will require additional upgrading. During the 2025 planning window replacement hardware and software will be needed to stay current with developing technology. Consequently this plan includes an allowance for a replacement control system at the end of the Vine Street WTP planning window.

Water Quality Monitoring Upgrades (Project Number WTP01)

This project provides two on-line turbidimeters (one for each Accelator's effluent pipeline), ten on-line particle counters (one for each filter) and bench-top water quality analysis capabilities for TOC analysis.

WTP Automation Upgrade, Plant Work (Project Number WTP10)

This project replaces the existing TI PLC system at the Vine Street WTP with a redundant Allen-Bradley PLC5 system. Proposed improvements include preliminary design, programming modifications, PLC/radio hardware and software, electrical installation, and system start-up.

WTP Automation Upgrade, Distribution Work (Project Number WTP11)

Instrumentation and control improvements are recommended in the distribution system to enhance coordination of the WTP with system-wide distribution system needs. This project replaces on-site monitoring and control instrumentation at the following sites:

- 34th Avenue Reservoir and Pump Station,
- Queen Avenue Reservoir and Pump Station,
- Broadway Reservoir,

- North Albany Pump Station,
- Gibson Hill Road Pump Station, and
- Valley View Reservoir sites.

These six units will be configured to utilize communications via the new DataRadio master station installed at the WTP.

Distribution System Pressure Monitoring Improvements (Project Number WTP18)

This project involves installation of five new pressure transmitters on distribution system piping to increase overall system monitoring. The pressure monitors will assist operators better respond to and manage changes in the system as demands and pressures fluctuate. Pressure transmitters should be installed in close proximity to existing wastewater pumping stations to facilitate ease of data acquisition.

Instrumentation and Control Improvements (Project Number WTP24)

Instrumentation and control improvements will need to be replaced at the conclusion of the Vine Street WTP planning period (2025). Costs associated with this project include: preliminary design, upgrades to the remote telemetry units (RTUs), as well as replacement of the main plant PLC and upgrades to the SCADA system at the WTP.

Accelator Improvements (Project Numbers WTP03 and 19)

Two improvement projects are recommended for the accelators, one for each accelator. Plant operators report frequent problems with the bearings/lubricants in the Accelator No. 2. Additionally the tube settlers installed in 1991 are cracking due to aging and exposure to sunlight.

Replace Accelator Settling Tubes (Project Number WTP03)

A thorough inspection of Accelator No. 2, including the mechanical systems and replacement of the tube settlers, is recommended. Sand blasting/re-coating the Accelator's interior during the replacement is also recommended.

Replace Accelator Settling Tubes (Project Number WTP19)

The useful life of the Accelator settling tubes is approximately 20 years. Tube settlers were installed in Accelator No. 1 in 1998. Though there is no indication of tube settler cracking in the Accelator No. 1, these will need to be replaced within the 2025 planning window and should be coordinated with sand blasting and recoating of the Accelator's interior.

Filter Improvements (Projects Numbers WTP02, 13, 20 and 21)

This plan recommends four improvement projects to the rapid sand filters. These improvements include replacement of surface wash system, media, filter to waste piping and replacement of valves and actuators.

The surface wash systems for the smaller filters (filters 1 through 6) should be replaced with stainless steel surface wash agitators due to age and corrosion of the piping. The valves and actuators for all filters are also nearing the end of their useful life and a study investigating leaking through these valves, including a prioritized replacement schedule, is recommended. In addition, the piping and valves for all filter pipe galleries need re-coating.

Normally, the life expectancy of filter media/underdrains is 20-30 years. Media for filters 1-6 and 9 and 10 was installed with the 1991 WTP upgrade. Filter media in filters 7 and 8 was installed in the mid-1960 and has been in service for more than 30 years. During the 2025 planning period, the useful life of the media and gravel support system will be reached and the filter media will need to be replaced or membrane units similar to those planned for the Scravel Hill WTP installed as an alternative.

For the purposes of this study, media replacement is recommended. If media replacement is performed, the filter underdrains for all filters and surface wash systems (for filters 7-10, the four larger filters) should also be re-built to accommodate a slightly deeper media. Replacement of the surface wash systems for filters 1 through 6 has also been recommended as discussed above. New media for filters 1 through 8 should include “gravel-less” block underdrains. Media for all filters should be replaced with granular activated carbon (GAC) over 12-inches of sand for a total bed depth of 30-inches and new fiberglass troughs and stainless steel surface wash agitators (filters 7 through 10) provided.

Backwash/Surface Wash Piping System Improvements (Project Number WTP02)

Surface wash improvements include automatic backwash control capabilities to optimize performance, enhance reliability and reduce risk of filter damage. This project replaces the existing surface wash system and upsizes filter-to-waste system pipes and valves in filter bays 1-6 from four to eight inches. The project includes a redundant 75 hp, 7,500 gpm variable speed backwash pump for filters 7-8 that replaces the existing smaller constant speed backwash pump. Improvements also include valve and piping modifications where necessary, as well as instrumentation and control work to control backwash flow.

WTP Filter Gallery Maintenance (Project Number WTP13)

This project replaces valves and actuators associated with filters 1-8, with the exception of filter-to-waste valves in filters 1-6 that have been accounted for in project WTP02. The project includes re-coating pipelines in both the old and new filter pipe galleries.

Repair/Replace Filter Media/Underdrain System (Project Number WTP20)

This project involves removal and disposal of all existing filter media and underdrains. New underdrains are recommended to be “gravel-less” plastic blocks. Replacement media should be at least 30-inches of dual media (sand and anthracite coal). Existing troughs should be removed and replaced with fiberglass troughs and new surface-wash system piping and agitators installed on filters 7-10 (surface wash improvements to filter 1-6 are included in WTP02).

Add Granular Activated Carbon (GAC) to Filter Media (Project Number WTP21)

This project incorporates GAC in lieu of anthracite coal as part of the filter media improvements. GAC offers greater protection against potential organic contaminants which may be present in the raw water and consequently will increase the reliability and safety of finished water. Costs associated with this project are based on the incremental costs of GAC over anthracite.

Clearwell Improvements (Project Number WTP14)

A single clearwell improvement project is recommended to repair a suspected leak in the clearwell and install a drain line and valve.

Clearwell Repairs (Project Number WTP14)

The clearwell beneath the small filters (filters 1-6) is suspected to be leaking and in need of structural repair. Also, there is currently no way to drain or bypass the clearwell for maintenance. In addition to repairing any leaks, installation of a new drain line and valve are recommended. Alternatively, a sump could be installed in the bottom of the clearwell allowing for a pump to be temporarily lowered into the sump for dewatering. The City should consider deferring clearwell improvements until the Scravel Hill WTP comes on line, or scheduling the repairs during a period of very low demand when the Vine Street WTP doesn't need to operate. The plant needs to be out of service for 1-2 days to make these repairs.

Plant Piping and Valve Improvements (Project Numbers WTP04, 22 and 23)

Three improvement projects are recommended to plant piping and valves at the Vine Street WTP. These improvements provide access for pipeline inspection and maintenance, programmed replacement of valves and actuators and removal of a hydraulic bottleneck in the filter gallery.

Access is required for routine inspection of large diameter (>20-inch) waterlines at the Vine Street WTP. In the past, the finished water suction header from the Maple Street Reservoir has accumulated material buildup limiting the pipelines capacity. Similar buildup of corrosion and material buildup is suspected in the 24-inch steel transfer line, approximately the same age as the filtered water header. There is currently no way to clean these pipes, or inspect other larger diameter pipelines at the WTP.

All valves and actuators at the plant will need to be replaced at least once during the 2025 planning window. The frequency of replacement depends on the rate a valve is exercised; for instance, seals on a modulating valve will need to be replaced more often than those of a standard open/close valve. For planning purposes, we recommend replacement of every large valve and actuator at the WTP with electrically actuated valves over the course of the planning window. The City should include the power requirements for operating these valves as part of the total back up power demand for the WTP.

Plant Pipeline Inspection and Cleaning (Project Number WTP04)

Routine, thorough inspections and cleaning of the large diameter pipelines (> 20-inch) at the WTP are required to minimize the accumulation of material build-up. This project installs several new pipeline clean-outs for introduction and retrieval of pipe cleaning equipment. Once these improvements are installed, a video inspection of all larger diameter pipelines can be completed to determine the degree of corrosion/material accumulation, particularly in the 24-inch steel transfer line.

Valve Maintenance (Project Number WTP22)

Programmed replacement of WTP plant valves and actuators not identified in projects WTP02 and WTP13 is recommended as these facilities reach the end of their service lives. Replacement valves and actuators include raw water isolation valves, check valves, settled water isolation valves, accelator drain valves and backwash inlet/outlet valves.

Plant Hydraulics (Project Number WTP23)

This project removes a hydraulic bottleneck between the Accelators and Filters. This bottleneck currently limits the flow to the six smaller filters (filters 1-6), resulting in approximately 2,400 gpm (3.5 MGD) reduction in capacity. Costs associated with this project include improvements to minimize air entrainment in the settled water pipelines, filter gallery-piping improvements and replacement of the existing open channel between filters 1-6 and filters 7-10 with a closed conduit.

Maple Street Reservoir Improvements (Project Numbers WTP05 and 25)

Two improvements projects are recommended for the Maple Street Reservoir. These projects are needed to repair or replace the baffle curtain in the reservoir and to complete related improvements to minimize short circuiting and increase chlorine contact time.

The Maple Street Reservoir has a capacity of 2 million gallons and is used to equalize production from the Vine Street WTP and to meet chlorine contact time requirements as part of the treatment process. A Hypalon (chlorosulfonated polyethylene) baffle was installed in the reservoir as a part of the 1991 upgrade. This baffle provides a more circuitous route through the reservoir thereby increasing chlorine contact time. Hypalon normally has a material life of 20-25 years.

An inspection of the existing Hypalon baffle was completed in April 2000 and revealed potential sites for “short-circuiting” within the reservoir. This was later confirmed by performing tracer tests through the Maple Street Reservoir. Immediate repair and replacement of the baffle is recommended; cleaning, inspection and re-coating (if warranted) of the reservoir interior during replacement is also recommended. In addition, the useful life of the new baffle is expected to be reached within the 2025 planning window. Therefore, the baffle curtain inside the Maple Street Reservoir will need to be replaced again within the planning window.

Repair Maple Street Reservoir Baffle and Improve Disinfection Performance (Project Number WTP05)

This project includes repair or replacement of the Hypalon baffle and improvements to the inlet and outlet piping, including complete closure of a gap between the baffle and interior reservoir wall that may be contributing to short circuiting and less than expected chlorine contact times.

Replace Maple Street Reservoir Baffle (Project Number WTP25)

The baffle installed in the Maple Street Reservoir as part of project number WTP05 will reach the end of its expected service life near the end of the planning window for the Vine Street WTP. This project involves replacement of the baffle membrane, but is somewhat less extensive than WTP05 because it will not include modifications to the reservoir’s inlet and out piping.

Control Building Improvements (Project Numbers WTP07 and 12)

Two improvement projects are recommended for the Control Building. These projects upgrade the heating, ventilation and air conditioning (HVAC) systems and install security improvements.

Replace/Repair Control Room Building HVAC System (Project Number WTP07)

The Control Building's HVAC system is antiquated, comprised of portable, electric wall-mount units that do not provide efficient climate control; temperature is often too hot in the summer, too cold in the winter. Improvements to update heating and cooling systems in the control building are recommended and may need to be coordinated with improvements to the electrical system to ensure an adequate power supply is available.

WTP Security Upgrade (Project Number WTP12)

This project involves installation of security improvements identified through project number Planning 1, Systemwide Security Assessment. Anticipated improvements include installation of cameras, gate keypads and integration of the new system into the City's SCADA network.

Electrical System Evaluation (Project Number WTP08)

One improvement project is included for evaluation of the plant's electrical system. An evaluation of the electrical system is recommended based on a history of premature failures in pumping system drive units and the potential for using VFDs to address plant pumping needs.

VFD Harmonics Evaluation (Project Number WTP08)

This project involves a thorough evaluation of the plant's electrical system, including the harmonics, transient currents and other factors that may have limited the life of drive units. The study will include an evaluation of the grounding grid and circuit breaker testing.

Regulatory Related Improvements (Project Numbers WTP06, 09, 15, 16 and 17)

Five regulatory improvement projects are recommended to address building, fire, safety and environmental regulatory related needs at the Vine Street WTP.

Chlorine System Safety Improvements (Project Number WTP06)

This plan recommends moving from gas to liquid chlorine (sodium hypochlorite) storage and feed systems at the plant. Although building and fire codes do not require improvements to the chlorine system unless the City makes improvements to the WTP, the plant is in close proximity to residential properties and safety improvements are recommended.

If gas chlorine were retained, the City would be required to install either new "double-lined" chlorine gas containment tanks, or a scrubber (and potentially a new building). Considering the future use of the Vine Street WTP as a peaking and supplementary supply, the most viable alternative is sodium hypochlorite, delivered and stored on site as 12 percent solution. This alternative represents the lowest capital cost investment compared to modifying the existing chlorine gas system. Incorporation of sodium hypochlorite into the WTP would involve the installation of two 6,000-gallon storage tanks, chemical metering and feed facilities.

ADA/OSHA Compliance Upgrade (Project Number WTP09)

Several locations at the WTP do not meet current employee protection standards against falls and accidents. Stairs, steps, ladders and handrails should be improved to meet current codes. In addition, the City should consider complying with the current Americans with Disabilities

Act (ADA) access requirements where practical within the context of retaining the historic character of the structure. A study assessing OSHA and ADA needs and more stringent permitting requirements for the structure based on its historic designation is recommended.

This project involves inspection and recommendations for specific improvements needed to comply with the ADA and OSHA standards. Recommended improvements as a result of this evaluation will likely include handrail, walkway, platform and other improvements needed to comply with these requirements.

Chemical Storage Improvements (Project Number WTP15)

The liquid alum tank on the second floor of the chemical storage building is not protected from leaks should the tank become damaged. Construction of a wall around the base of the alum tank is recommended to contain potential leaks.

Solids Handling (Project Number WTP16)

This project reshapes and relines the two existing backwash ponds to improve storage capacity and minimize leaching. The project also includes installation of a sump on each of the ponds. In addition to improved solids handing at the Vine Street WTP, this project will help ensure continued compliance with NPDES permit requirements.

Seismic Upgrades (Project Number WTP17)

There have been several earthquakes in the Pacific Northwest over the past 10-years that could have severely damaged the plant had they occurred in proximity to Albany. All of the existing plant buildings and piping are susceptible to damage during an earthquake, especially the unreinforced masonry (URM) buildings. The liquid alum storage tank is also considered "at risk" in terms of protection against seismic events, especially considering its location on the 2nd floor of the chemical storage building.

A system vulnerability study is recommended to define the plant's vulnerability to seismic events. Anticipated improvements as part of this project include installation of pipeline restraints and reinforcement of brick masonry structures. In addition, seismic valves and restraints at the Maple Street Reservoir are required as discussed in *Chapter 10 - Distribution System Evaluation*.

High Service Pump Station (HSPS) (Project Numbers PS8, 9 and 10)

The High Service Pump Station pumps finished water from the Maple Street Reservoir to the distribution system. Three improvement projects are recommended for the station. A discussion of the capacity of this pump station is included in the pump section of *Chapter 10 - Distribution System Evaluation*.

Replace HSPS Pump No. 14 (200 HP) (Project Number PS8)

This project replaces pump no. 14, a 200 HP pump, at the HSPS. The *High Service Pumping Evaluation* by Brown and Caldwell recommended replacement of pump no. 14 to allow the pump to operate in tandem with the other pumps at the HSPS. Replacing pump no. 14 will increase reliability and improve efficiency and operation of the HSPS.

HSPS Backup Power Outlet (Project Number PS9)

A backup power outlet for use with a portable generator is recommended for the HSPS to provide limited power and pumping capacity during power outages. The outlet would be configured to allow a quick connection and transition to power generated on site. An evaluation of the HSPS is included in *Chapter 10 – Distribution System Evaluation*.

Analysis of Operating Conditions, including Variable Frequency Drives (VFDs) at the HSPS (Project Number PS10)

This project involves an analysis of operating conditions at the HSPS and includes the possible use of VFDs to address gaps in the range of available flows for the pump station. At the time of the inspection two pumps at the HSPS were equipped with variable frequency drives (VFDs), Pumps No. 12 and No. 15 (300 HP each). VFDs allow a pump to cover a broader range of flow rates and pressure heads by slowing down or speeding up the radial speed of the pump motor. A typical operating range of a VFD drive is from 75 percent to 105 percent of design speed. The flow rate will increase or decrease directly (linearly) with speed and the pressure head will increase or decrease by the square of the speed. Brown and

Caldwell performed an analysis on the HSPS. In their study, the VFD's at the HSPS were found to have a very narrow range of operation from approximately 92 percent to 100 percent. The study also identified gaps in the flow regime at lower flow rates between 1,600 gpm and 3,100 gpm[64].

Since the inspection, the City has removed the VFD's from Pumps No. 12 and No. 15. This plan recommends that the need for VFDs be further evaluated, and if found beneficial, a control strategy using the VFDs should be developed.

Planning Projects (Project Numbers Planning-1 and 2)

Two planning projects are recommended for the Vine Street WTP in addition to the improvements projects listed above. These projects meet regulatory requirements for security of the water system and expected updates of this facility plan.

System-wide Security Assessment (Project Number Planning1)

As discussed in *Chapter 6 - Water System Regulatory Review*, a system wide security assessment is required under the Public Health Security and Bioterrorism Preparedness Response Act amendment to the Safe Drinking Water Act. This project provides funding for preparation of a plan meeting the requirements of the act, including a system vulnerability assessment and follow-up emergency preparedness plan sufficient to meet regulatory requirements. An allowance for improvements identified as a result of this evaluation is included in project number WTP12.

Facility Plan Updates (Project Number Planning 2)

Water facility plan updates are required periodically to reflect changes in expected growth patterns and demands, the regulatory environment and capital improvement needs. On average facility plan updates are completed on 10-year cycles. Two water facility plan updates have been included.

[64] Brown and Caldwell. *City of Albany Water System, High Service Pumping Evaluation*. January, 2001

Table 8-2 presents a summary of recommended WTP improvement projects. Estimates include a 20 percent allowance for engineering/administration fees, and a 20 percent contingency allowance. A greater allowance for engineering, legal and administrative costs has been used for the Vine Street WTP cost estimates due to the inherit complexities associated with plant design and retrofitting an existing historic structure. The methodology used to prepare cost estimates is described in *Chapter 11 - Basis of Cost Estimates*. Staging of improvements is discussed in *Chapter 12 - Recommended Plan*.

SUMMARY AND CONCLUSIONS

The City of Albany's Vine Street WTP has successfully met the City's drinking water needs for over 90 years. Initially, when the new Scravel Hill WTP is brought on-line, the Vine Street plant will be used to augment the Scravel Hill WTP during peak demand periods and to serve as a supply for emergency purposes. As demands increase overtime, the Vine Street WTP will play a larger role in meeting increased demands and will be ultimately relied upon to provide half of the projected maximum day demand at buildout.

Investments in the Vine Street WTP will be required to maximize its remaining useful life, replace outdated equipment and systems, and meet future regulatory requirements. With improvements, the plant can continue to serve the City well beyond the 2025 planning horizon.

Table 8-2: Vine Street WTP Improvement Project Summary and Cost Estimate

Project No.	*Project Title*	*Description*	*Total Project Cost*
WTP01	Water quality monitoring improvements	Bench top UV spectrophotometer	**$84,000**
WTP02	Surface wash improvements	Surface wash, backwash pump, valves & piping	**$329,000**
WTP03	Accelator #2 improvements	Sand blast/re-coat, replace tube settlers	**$213,000**
WTP04	Pipeline cleaning/inspection	Inspect pipeline system, install pipe clean-outs	**$112,000**
WTP05	Disinfection performance improvements	Repair/replace baffle at Maple Street Reservoir	**$115,000**
WTP06	Chlorine system improvements	Sodium hypochlorite storage and feed system	**$140,000**
WTP07	Plant HVAC evaluation	Inspection & report	**$70,000**
WTP08	VFD harmonics evaluation	Inspection & report	**$20,000**
WTP09	ADA/OSHA Compliance Evaluation	Inspection	**$50,000**
WTP10	Automation upgrade, WTP	Hardware/software programming	**$535,000**
WTP11	Automation upgrade, distribution system	Hardware/software programming	**$127,000**
WTP12	WTP security upgrade	Engineering, equipment installation	**$150,000**
WTP13	WTP filter gallery improvements	Pipeline improvements	**$560,000**
WTP14	Clearwell repairs	Crack/leak repairs, install drain pipe	**$70,000**
WTP15	Chemical storage improvements	Alum containment facility	**$28,000**
WTP16	Solids handling	Reshape/reline ponds and sump installation	**$220,000**
WTP17	Seismic restraint upgrades	Reinforce control building process units	**$570,000**
WTP18	Distribution system monitoring equipment	Pressure transmitters, I&C improvements	**$70,000**
WTP19	Accelator improvements	Settling tube replacement	**$210,000**
WTP20	Filter improvements	Install new underdrains, replace filter media	**$682,000**
WTP21	Filter media improvements	Install GAC filter media	**$150,000**
WTP22	Plant valve/actuator replacement	Replace all valves and actuators	**$994,000**
WTP23	WTP filter gallery improvements	Pipeline improvements	**$280,000**
WTP24	Instrumentation and control improvements	Hardware/software programming	**$840,000**
WTP25	Disinfection performance improvements	Replace baffle in Maple Street Reservoir	**$80,000**
PS8	High pressure pump station improvements	Replaces HSPS Pump No. 14	**$75,000**
PS9	HSPS backup power oulet	Installs outlet for backup power from a generator	**$30,000**
PS10	Analysis of HSPS operating conditons	Evaluate station operation and potential use of VFDs	**$55,000**
Planning-1	Security evaluation	System-wide vulnerability assessment	**$150,000**
Planning-2	Water Facility Plan updates	Update of plan	**$600,000**
		Total, Vine Street WTP Improvements	***$7,609,000***

CHAPTER 9 – JOINT WATER PROJECT

INTRODUCTION

This facility plan scope of work initially included an evaluation of alternative water supply sources needed in addition to the Vine Street Water Treatment Plant (WTP). The scope of work included development of alternative water sources and a planning level assessment of water quality, water rights and project feasibility for each source option. These tasks were put on hold and later deleted from the water facility plan as the concept of a regional or Joint Water Project (JWP) progressed.

The JWP will include an intake structure on the Santiam River, raw water transmission lines, construction of the Scravel Hill WTP and finished water lines and will serve as a regional facility serving the Cities of Millersburg and Albany. As noted in *Chapter 8 - Vine Street Water Treatment Plant,* the Vine Street Plant can be expanded to provide Albany with an ultimate capacity of 20 MGD; half of the 40 MGD projected maximum day demand (MDD) capacity required at buildout of the Urban Growth Boundary (UGB). The Scravel Hill WTP will provide Albany with the additional 20 MGD capacity needed to fully meet projected demands.

The JWP stems from a project initiated by the City of Millersburg to design and construct an independent water source supplied by the Santiam River. Millersburg notified the City of Albany of their intent to disconnect from Albany's water system once the new water source was completed. Albany expressed interest in construction of a regional or shared water supply facility that would be consistent with recommendations in the *1988 Albany and Millersburg Water Facility Plan*[65]. A series of discussions concerning a regional water treatment facility began at approximately the same time work was started on this update of the 1988 water facility plan.

This chapter briefly reviews the history of the discussions leading to development of the JWP concept and summarizes work done by the engineering consulting firm of CH2M-Hill to define the scope and cost of the JWP. Intake, raw water transmission, treatment, storage, finished water lines and related improvements that make up the JWP are discussed in this chapter and incorporated as part of the recommended water system improvements in *Chapter 12 - Recommended Plan*.

DEVELOPMENT OF THE ALBANY/MILLERSBURG JOINT WATER PROJECT

The engineering firm of Brown and Caldwell identified the need for an additional water source during development of the *1988 Albany and Millersburg Water Facility Plan*. The 1988 plan considered alternative supplies and recommended development of a second supply on the east side of the UGB, supplied either by groundwater or the Santiam River. Subsequent investments in the distribution system have been based on balancing supply from the existing Vine Street treatment plant and an east end supply source. Lack of a protected, viable deep aquifer and concerns with mixing ground and surface water supplies precluded detailed consideration of a ground water source to meet the projected supply shortfall. Although a second source of supply from the Santiam River was used as a basis for sizing distribution improvements, the 1988 plan did not identify a specific eastside site.

[65] February 1988, Albany and Millersburg Water Facility Plan, Brown and Caldwell Engineering, Inc.

As noted in earlier chapters, Albany currently provides water service to the City of Millersburg. The provisions of service are governed by the terms and conditions of two intergovernmental service agreements. The City of Millersburg began exploring the feasibility of constructing a separate and independent water supply in August 1989[66]. At that time, Millersburg applied to the state Water Resources Department (WRD) for a water right permit from the Willamette and Santiam Rivers. In October 1996 WRD granted a water right permit[67] to Millersburg to withdraw up to 22 cubic feet per second (cfs) or 14.2 million gallons per day (MGD) from either the Willamette or the Santiam Rivers. New rules recently adopted by WRD governing municipal water rights may put these permitted rights in jeopardy.

In 1998 Millersburg retained the engineering firm of CH2M-Hill to update the *Millersburg Water System Master Plan* and determine the feasibility of constructing a separate, independent water supply. CH2M-Hill completed the update of Millersburg's facility plan in January 1999[68]. The updated plan recommended construction of an intake on the Santiam River just downstream of the confluence of the North Santiam and South Santiam Rivers. Millersburg notified Albany in September 1999 of their intent to construct an independent water treatment plant and disconnect from Albany's water system by 2001.

Construction of Millersburg's WTP and related improvements did not progress as rapidly as planned and in August 2001 Millersburg and Albany City Councils began a series of discussions to explore the possibility of constructing a joint water treatment facility that would meet the long-term needs of both communities. These discussions included state and federal regulatory agencies to fully explore water right, land use, fish and wildlife, flood plain and wetland protection issues.

In July 2002 discussions culminated in execution of a cooperative inter-governmental agreement to construct and operate a joint water project serving both communities needs. The agreement includes rules of governance and cost-sharing for construction and operation of the JWP and is included as ***Appendix F***.

The agreement calls for construction of a water treatment plant supplied by the Santiam River. The treatment plant will have an initial capacity of 12 MGD (10 MGD dedicated to Albany and 2 MGD dedicated to Millersburg) and an ultimate capacity of 26 MGD (20 MGD dedicated to Albany and 6 MGD dedicated to Millersburg). Albany benefits by having an increased water supply quantity with additional water rights from Millersburg. Millersburg benefits by having increased water supply security by utilizing Albany's higher priority water rights.

Albany and Millersburg retained CH2M-Hill to assist with development of the JWP and to prepare a series of technical memorandums to frame decisions for the joint City Councils as they considered the scope of the project and cost sharing opportunities. These memorandums are summarized in a document entitled *Albany-Millersburg Joint Water Supply Project Conceptual Design Project Definition*. A copy of the document is included as ***Appendix G.***

[66] State of Oregon, Water Resources Department, Application No. 70055 and 70056 for Permit to Appropriate Surface Water, August 31, 1989.

[67] State of Oregon, Water Resources Department, Permit to Appropriate The Public Waters (municipal), priority date August 31, 1989

[68] January 1999, City of Millersburg Water System Master Plan, CH2M-Hill

THE JOINT WATER PROJECT

The Joint Water Project will be supplied by an intake on the Santiam River located approximately one-quarter mile downstream of the confluence of the North Santiam and South Santiam Rivers. Raw water will be pumped from the intake to a treatment plant located on Scravel Hill. Following treatment, finished water will be conveyed using a shared gravity flow water line from the Scravel Hill Treatment Plant Reservoir to the intersection of Century Drive and Berry Drive. At this point the water line splits with branches to each community's distribution system.

Key facilities included with the Joint Water Project are listed below and shown in a process schematic, ***Figure 9-1***, and on a site map, ***Figure 9-2***. These facilities are discussed in detail in the following sections of this chapter:

- Raw water intake facility,
- Raw water pump station,
- Raw water transmission pipeline,
- Scravel Hill water treatment plant,
- Finished water reservoirs/clear well, and
- Finished water transmission pipelines.

RAW WATER INTAKE FACILITY

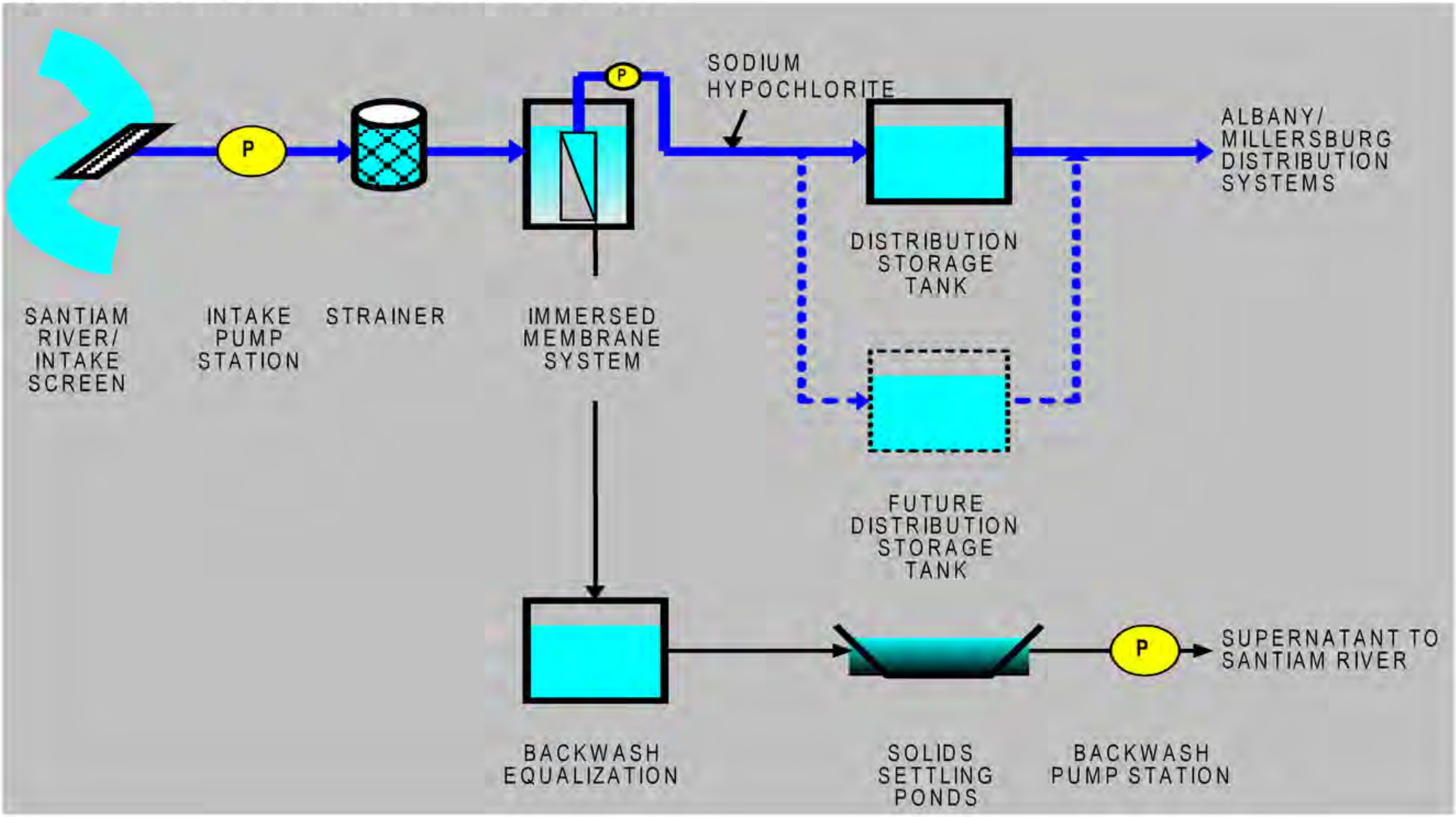
Intake Location

As noted above, the location of the intake structure will be just downstream of the confluence of the North Santiam and South Santiam Rivers at river mile 11.8[69]. CH2M-Hill evaluated the intake site and concluded that the channel bed and bank are stable and well suited for use as an intake.

The intake and raw water pump station will be located on property owned by the City of Albany. The location of the intake and raw water pump station are shown on ***Figure 9-3***. A photograph of the intake site is included as ***Photograph 9-1***.

[69] August 2, 2000, *Wetlands and Waters Determination and Delineation Report for the City of Millersburg Water System*, CH2M-Hill

Figure 9-1: Process Schematic for the Joint Water Project



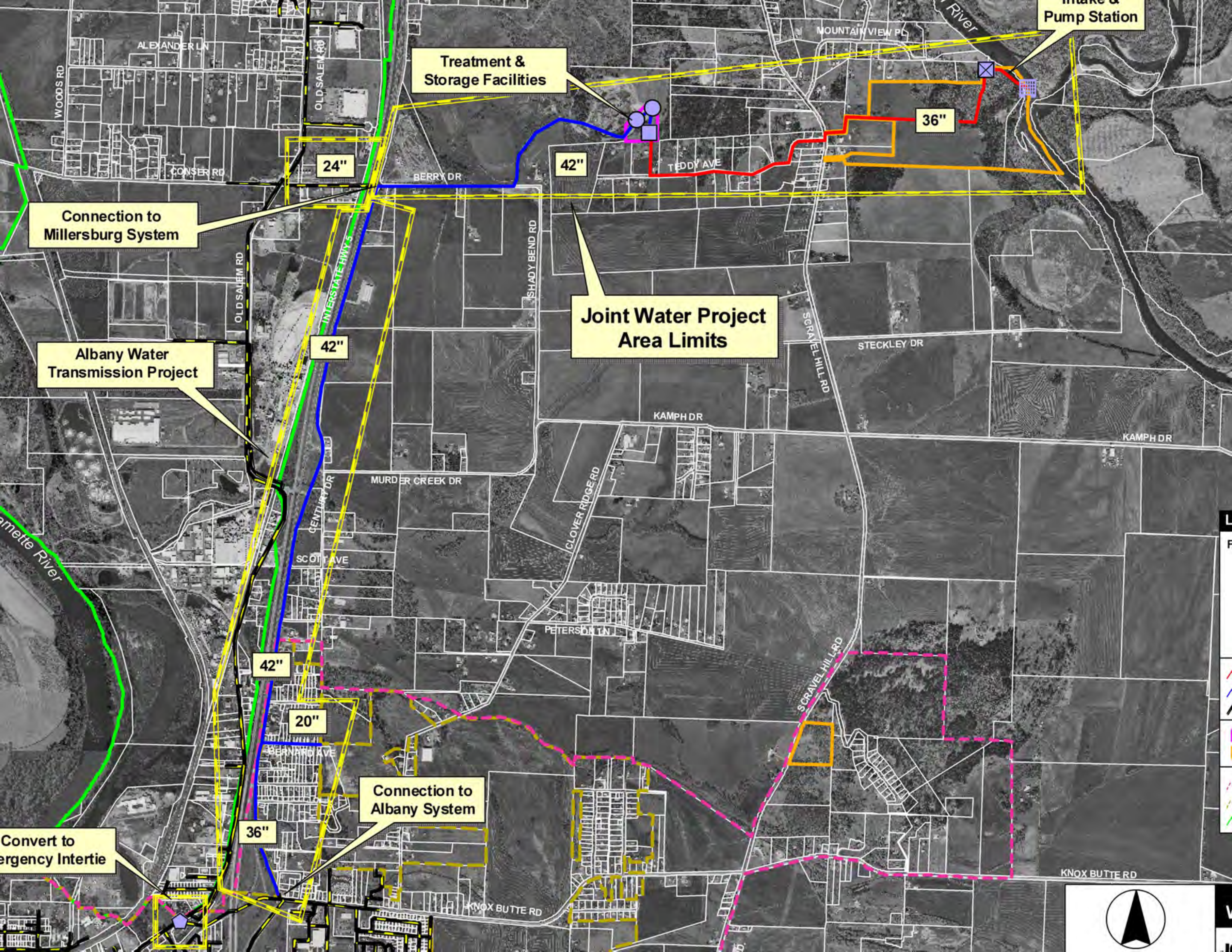
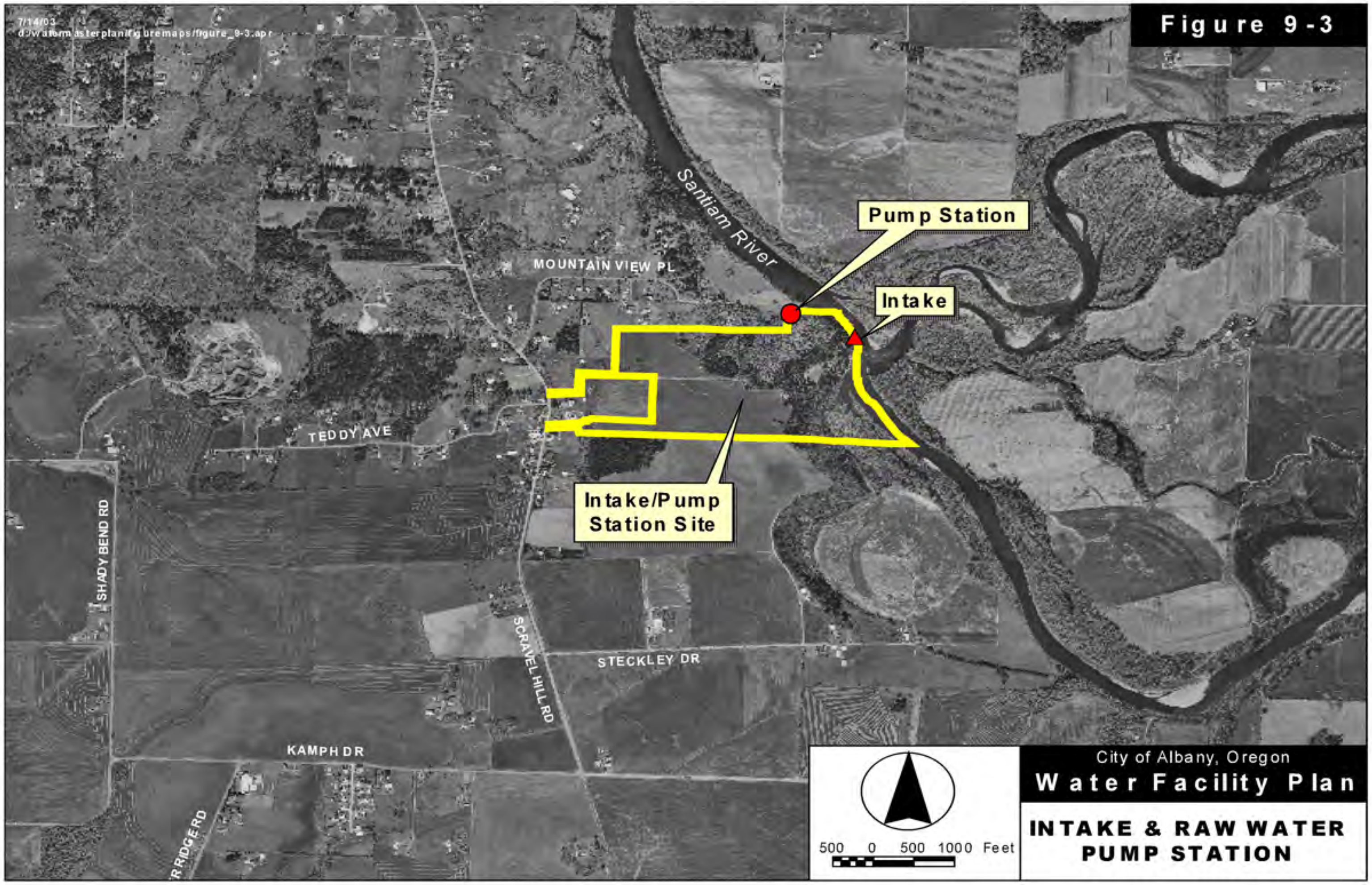

Pump Station
Treatment &
Storage Facilities
Connection to
Millersburg System
Albany Water
Transmission Project
Joint Water Project
Area Limits
Connection to
Albany System
Convert to
ergency Intertie
36"
42"
24"
42"
42"
20"
36"
MOUNTAIN VIEW PL
ALEXANDER LN
WOODS RD
OLD SALEM RD
CONSER RD
BERRY DR
TEDDY AVE
SHADY BEND RD
STECKLEY DR
SCRAVEL HILL RD
KAMPH DR
MURDER CREEK DR
CLOVER RIDGE RD
CENTURY DR
SCOTT AVE
PETERSON LN
BERNARD AVE
KNOX BUTTE RD
INTERSTATE HWY 5
River

Figure 9-3 Intake and Raw Water Pump Station



Photograph 9-1: Photograph of Intake Site (looking upstream on the Santiam River)

The intake will be designed to meet initial and buildout demands for both communities. These demands have been discussed above and are summarized below in ***Table 9-1***.

Table 9-1: Required Santiam River Intake Capacities

Capacity	*Albany*	*Millersburg*	*Combined*
Initial	10 MGD	2 MGD	12 MGD
Buildout	20 MGD	6 MGD	26 MGD

Raw water intake facilities are required by law to be screened so that fish will not be drawn into or held against the water intake inlet screen. The inlet screen design must follow construction guidelines developed by the National Oceanic and Atmospheric Administration, Fisheries Service (NOAA, Fisheries). The conceptual design for this intake uses eight half-round screens connected to four 24-inch gravity conveyance pipelines. The pipelines will run parallel to the shoreline to a point where they can cross the shoreline riparian zone with minimal disruption. An air burst system will be provided to periodically clean the screens. The compressors and associated controls will be located in the raw water pump station.

RAW WATER PUMP STATION

The proposed raw water pump station is located on property purchased by the City of Albany. The relationship of the pump station and intake are shown schematically in ***Figure 9-4.*** The pump station is setback from the riverbank to ensure year-around access and to minimize riparian impacts.

Access to the pump station for routine maintenance will be through Albany's property off of Scravel Hill Road. The raw water pump station structure will be sized to accommodate the ultimate required capacity for buildout of both communities. Pumping, mechanical and related equipment will be installed in phases with initial improvements providing a firm capacity of approximately 20 MGD. Four 4,500 gallon per minute (GPM) (approximately 6.5 MGD each) pumps will be required to provide firm capacity for peak summertime demands. There will be lower initial headloss in the raw water transmission pipeline at initial flow rates because the raw water transmission pipeline is sized for buildout demands of 26 MGD. Consequently, the raw water pump station will meet the Scravel Hill WTP's firm capacity needs of 12 MGD during winter conditions and the 16.5 MGD estimated treatment plant capacity during summer conditions.

RAW WATER TRANSMISSION LINE

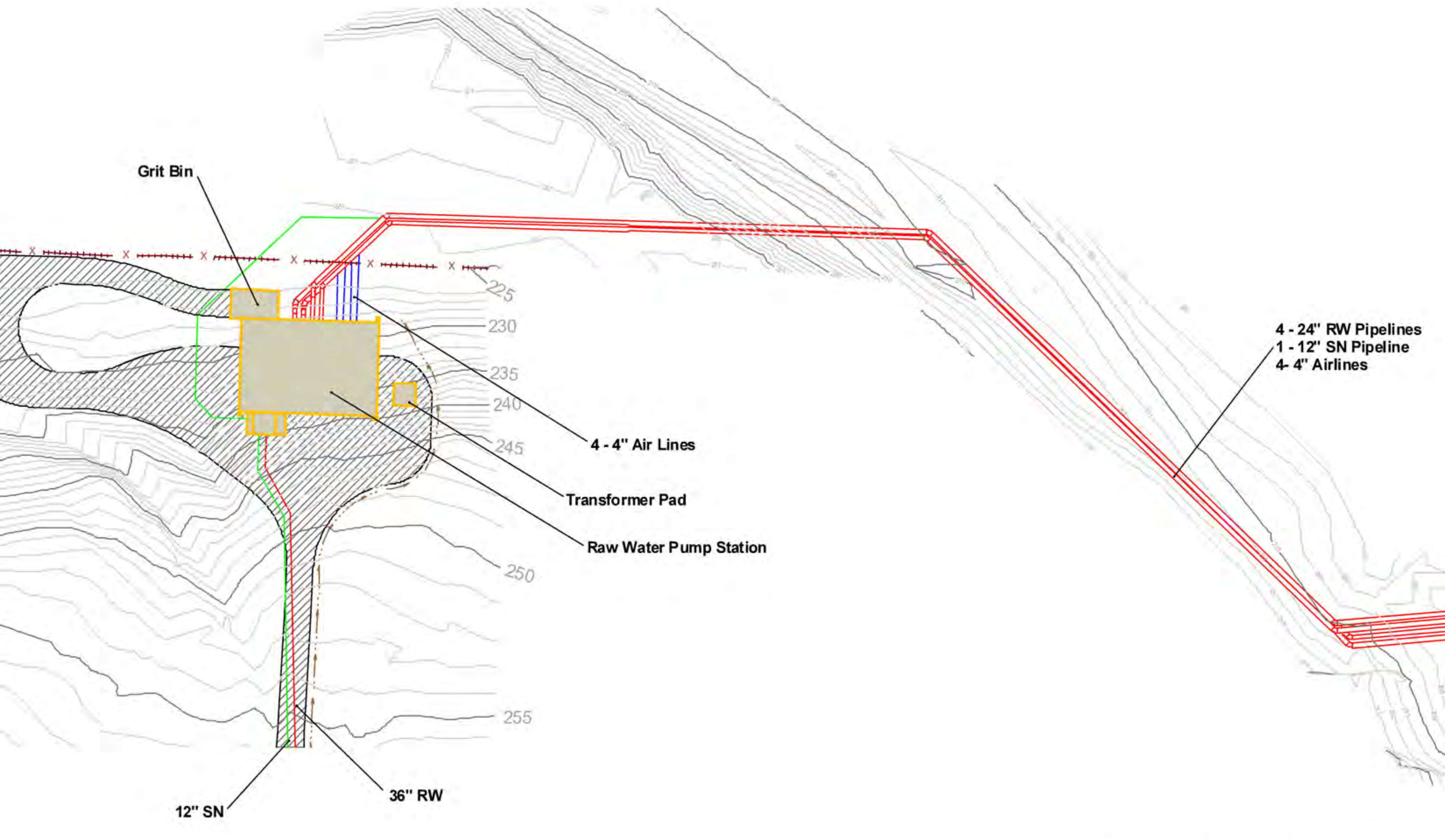
The raw water transmission pipe will extend from the raw water pump station to the Scravel Hill treatment plant. The alignment of the raw water transmission line is shown in ***Figure 9-2***. As shown on this figure, the raw water transmission line is a 36-inch diameter pressure pipeline extending approximately 8,600 feet, from the raw water pump station to the Scravel Hill WTP.

SCRAVEL HILL WATER TREATMENT PLANT

The Scravel Hill Water Treatment Plant will be constructed east of Millersburg on a 7.4-acre parcel north of Teddy Avenue, as shown on ***Figure 9-5*** and ***9-6***. Preliminary planning and conceptual design of the Scravel Hill WTP are based on use of a membrane filtration system. Membrane filtration is a treatment process that relies on an extremely "fine mesh" fabric that acts as a selective barrier, allowing treated water to pass while blocking contaminants. A vacuum is typically applied to draw raw water across the membrane. Advantages of membrane treatment systems are summarized below:

- Reliable treatment, relatively independent of raw water quality,
- Well positioned for future regulations,
- Readily expandable,
- Small land area requirements,
- Operational simplicity, and
- Potential automation.

It is important to note that some of these advantages require reliable computer and instrumentation systems to achieve the simplicity and automation expected. Current and forthcoming drinking water regulations recognize membrane technologies as the state of the art filtration technology for surface water treatment. Membrane filtration provides a positive barrier to pathogens such as *Giardia* and *Cryptosporidium*. The rapid adoption of membrane filtration for municipal water treatment has led to continued reductions in equipment costs, especially for larger capacity plants. The Scravel Hill WTP will utilize membrane treatment units manufactured by US Filter Corporation. US Filter was selected as the membrane provider through a competitive process based on performance, cost and ease of expansion. The membranes are expected to achieve a four-log removal of *Cryptosporidium*.



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Water

RAW WA

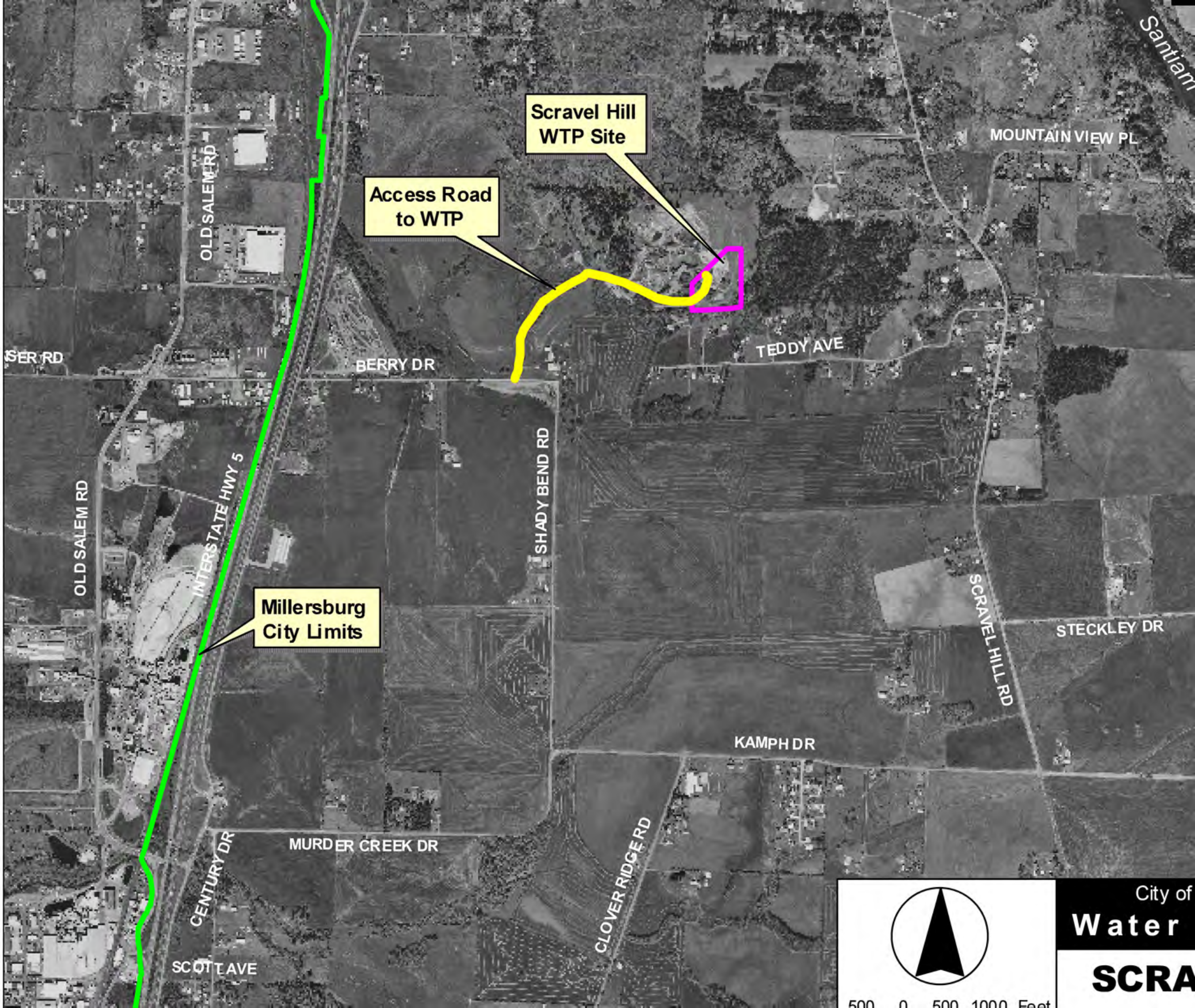

Scravel Hill
WTP Site
Access Road
to WTP
Millersburg
City Limits
Santiam
MOUNTAIN VIEW PL
TEDDY AVE
BERRY DR
SER RD
OLD SALEM RD
OLD SALEM RD
INTERSTATE HWY 5
SHADY BEND RD
STECKLEY DR
SCRAVEL HILL RD
KAMPH DR
CLOVER RIDGE RD
MURDER CREEK DR
CENTURY DR
SCOTT AVE
500 0 500 1000 Feet
City of
Water
SCRA

Figure 9-6: Footprint of Scravel Hill Treatment Plant

SOURCE WATER QUALITY

Water temperature and turbidity are fundamental criteria used to determine the feasibility and capacity of a membrane treatment system. Capacity of a membrane treatment system decreases as water temperatures decreases and as turbidity increases.

CH2M-Hill conducted a pilot study at the time the Millersburg WTP project was being developed. The pilot test used a membrane treatment process located at the City of Lebanon's WTP. The City of Lebanon is supplied by water from the South Santiam River via the Santiam-Albany Canal. The pilot test involved use of membrane treatment followed by bench scale testing and computer modeling to estimate finished water quality. The pilot study concluded that membrane treatment was a viable option and would meet state and federal drinking water requirements.

Historic water quality data is not available for the intake site. The most complete record of water quality data on the Santiam River system is available from studies on the North Santiam River performed by and for the City of Salem at Mehema, approximately 27 miles upstream of the JWP intake site. Data from this monitoring station at Mehama was used as the basis for assessing water quality in the Santiam River.

The most recent two full years of provisional data (June 26, 2000 - June 25, 2002), consisting of readings taken every half-hour, was used to establish design parameters. The lowest 5th percentile of the average daily temperatures and the highest 95th percentile of the maximum daily turbidity values for each month were used as a basis to size membrane treatment units.

Because the source water will typically be warmer and less turbid than these assumptions, the treatment plant is expected to have additional capacity above the 12 MGD initial requirement. Warmer source water temperatures should coincide with warm weather and high demand periods. Membrane treatment units are sized based on cooler water temperatures, and the warmer water will result in greater membrane treatment capacity at a time when it is most needed. The capacity of the Scravel Hill WTP will increase during summer months with warmer raw water. The summertime capacity is estimated to increase to 16.5 MGD due to this greater treatment efficiency.

Pilot testing and water quality monitoring for raw water turbidity, conductivity and temperature are now underway at the intake site. This work will confirm the treatment approach and identify any additional treatment needs required. Should pretreatment become necessary, space has been reserved at the Scravel Hill WTP site for this purpose.

FINISHED WATER QUALITY

The water produced from a conventional coagulation/clarification/filtration plant may be different from that produced from a membrane plant without pretreatment. Conventional plant processes typically remove 20 to 50 percent of the natural organic matter in the raw water. By contrast, a microfiltration membrane plant may only remove 10 to 20 percent of organics. As a result, the membrane filtrate may have a greater disinfection by-product (DBP) formation potential than the conventional coagulation/clarification/filtration plant. The Vine Street WTP is a conventional coagulation/clarification/filtration plant that receives its raw water from the Santiam-Albany Canal (Canal). The limited raw water data from the Canal at the diversion dam, along the Canal and at the Vine Street WTP, indicates that TOC level in the Canal can increase by 50 to 60 percent from the diversion point to the Vine Street WTP. The Scravel Hill WTP is a direct river intake and should experience lower TOC concentrations apparent in the Santiam River. Consequently DBPs are not anticipated to be a concern at the Scravel Hill WTP.

FINISHED WATER STORAGE RESERVOIR

Finished water storage at the Scravel Hill WTP is needed to meet treatment plant operational requirements, Millersburg storage needs, and to supplement Albany's emergency water storage.

The treatment plant requires finished water storage for disinfection (chlorine contact time) and plant operations. State drinking water regulations require that the plant provide sufficient contact time for the chlorine to kill pathogens that could be in the water. The amount of disinfection storage is directly dependent on the capacity of the plant. The treatment plant also has operating needs for finished water storage such as back flushing of the membranes, process water, wash down, and drinking water.

In addition to operational storage, finished water storage is required to meet short-term demands (equalization storage), to provide water to fight fires (fire storage), and to provide an emergency reserve (emergency storage) if the water supply is interrupted. The finished water reservoir at the treatment plant site will be used to meet operational storage requirements and finished water storage needs of both communities. Millersburg plans to use the treatment plant site to meet their equalization, fire and emergency storage needs. Albany plans to use storage at the Scravel Hill WTP to supplement existing reservoir storage in their distribution system. ***Tables 9-2*** and ***9-3*** summarize storage needs at the treatment plant for Millersburg and Albany. Although Millersburg's share of the plant capacity is less than Albany's, their total storage need is approximately equal to Albany's need at this location. This is because the Scravel Hill WTP will provide storage for all of Millersburg's distribution needs and only a portion of Albany's needs.

Table 9-2: Initial Storage Needs for Millersburg and Albany

Storage Component	***Millersburg (MG)***	***Albany (MG)***	***Total (MG)***
Disinfection	0.1	0.8	0.9
Distribution	2.8	2.0	4.8
Total	***2.9***	***2.8***	***5.7***

Table 9-3: Buildout Storage Needs for Millersburg and Albany

Storage Component	***Millersburg (MG)***	***Albany (MG)***	***Total (MG)***
Disinfection	0.5	1.8	2.3
Distribution	5.1	4.0	9.1
Total	***5.6***	***5.8***	***11.4***

FINISHED WATER PIPELINE

Finished water will need to be transported from the water treatment plant to each community's distribution system. A single 42-inch diameter water line will extend approximately 6,100 feet from the Scravel Hill WTP along Berry Drive to Century Drive. At Century Drive flow will be split to the south with a 42-inch water line serving Albany and to the west with a 24-inch water line serving Millersburg. Flow meters will be installed at each community's connection to the finished water line. Use of a control valve will be evaluated during design of the finished water pipeline. A pressure and/or flow control valve is one option that may be used to balance Albany's Zone 1 pressures (overflow elevation 385 feet) with the higher overflow elevation of 415 feet at the Scravel Hill Reservoir. Decisions concerning the control valve may influence the recommended line size for distribution project P36 and valving recommendations related to the Knox Butte Reservoir. This pipeline improvement project and valving recommendations are discussed in *Chapter 10 – Distribution System Evaluation*. A schematic showing the approximate alignment for the finished water lines and their relationship to both communities' distribution systems is shown as ***Figure 9-2***.

The transmission line improvements will include conversion of Millersburg's existing connection with Albany's distribution system to an emergency intertie between Albany's and Millersburg's distribution systems (see ***Figure 9-2***). This intertie will normally remain closed, but will be available as one of two possible interconnection points between the two distribution systems (the intersection of Century Drive and Berry Drive being the other) that could be used to meet emergency needs of either community.

SUMMARY AND CONCLUSIONS

The initial phase of construction for the Scravel Hill WTP will result in a shared treatment facility with a total capacity of 12 MGD; 2 MGD allocated to Millersburg and 10 MGD allocated to Albany. The scope and cost of the initial phase of improvements required for the Joint Water Project are summarized in ***Table 9-4***.

A preliminary planning level cost estimate has been developed for the cost of expanding the Scravel Hill WTP to provide a firm capacity of 26 MGD (JWP2), 20 MGD allocated to Albany and 6 MGD allocated to Millersburg. The scope of work required to increase the Scravel Hill WTP to reach full capacity involves addition of a fifth raw water pump, installation of a fifth membrane filtration unit and construction of a second 5.7 MG finished water reservoir. The total cost for this expansion is estimated to be $3.9 million.

Table 9-4: Estimated Project Costs, Joint Water Project, (JWP1)

Improvement/Funding	Cost Allocation		Total
	Albany	Millersburg	Cost
Raw water Intake/Transmission			
Intake facility	$ 732,000	$ 218,000	$ 950,000
Raw water pump station	$ 1,325,000	$ 345,000	$ 1,670,000
Raw water transmission (to JWP WTP)	$ 1,372,000	$ 410,000	$ 1,782,000
Sub-total	$ 3,429,000	$ 973,000	$ 4,402,000
Scravel Hill WTP			
Treatment Plant	$ 10,927,000	$ 2,238,000	$ 13,165,000
Instrumentation and Control	$ 1,851,000	$ 379,000	$ 2,230,000
Backwash ponds	$ 77,000	$ 23,000	$ 100,000
Sub-total	$ 12,855,000	$ 2,640,000	$ 15,495,000
Finished water Storage	$ 860,000	$ 844,000	$ 1,704,000
Sub-total	$ 860,000	$ 844,000	$ 1,704,000
Finished water transmission lines			
Shared transmission line	$ 1,129,000	$ 337,000	$ 1,466,000
Albany tranmission line	$ 2,931,000	$ -	$ 2,931,000
Sub-total	$ 4,060,000	$ 337,000	$ 4,397,000
Credits			
Intake site purchase	$ (300,000)	$ 300,000	$ -
Initial engineering	$ 1,200,000	$ (1,200,000)	$ -
Sub-total	$ 900,000	$ (900,000)	$ -
Other			
General Conditions	$ 1,533,000	$ 407,000	$ 1,940,000
Contingency	$ 3,410,000	$ 780,000	$ 4,190,000
Engineering	$ 5,218,000	$ 1,208,000	$ 6,426,000
Sub-total	$ 10,161,000	$ 2,395,000	$ 12,556,000
Total[1]	**$ 32,300,000**	**$ 6,300,000**	**$ 38,600,000**

[1] *Total costs are rounded to the nearest $100,000.*

CHAPTER 10 – DISTRIBUTION SYSTEM EVALUATION

INTRODUCTION

This chapter discusses the evaluation of Albany's distribution system and the resulting recommended improvements. The evaluation was completed through the development of a hydraulic model, a comparison of pipelines, pump stations and reservoirs to the planning criteria, field inspections of pump stations and reservoirs, and meetings and interviews with staff. Recommended improvement projects resulting from this evaluation are summarized at the end of this chapter and presented in *Chapter 12 - Recommended Plan*.

HYDRAULIC MODEL OF THE DISTRIBUTION SYSTEM

Model Methodology

The steady state hydraulic model developed for this water facility plan uses WaterCAD version 5.0 software. This software is currently one of the leading commercially available hydraulic analysis tools for the evaluation of water distribution systems and is supported by Haestad Methods, Inc. an engineering software development company. The WaterCAD model developed for this plan analyzes the system at one point in time with known boundary conditions (e.g. reservoir levels, pump status, demands) – known as a "steady-state model". This type of model is used for planning purposes and allows multiple scenarios to be reviewed and compared to planning criteria. WaterCAD provides an interface between Geographical Information Systems (GIS) and the model database. The user can automatically import/export data files (both graphics and attributes) using the SHAPEFILE (e.g., ArcView), and comma-delimited text (CSV) data.

Model Development

Currently the City maintains GIS coverage of the distribution network with over 9,000 pipeline segments, with multiple fields of attribute data for each segment. Because of the size of the GIS coverage and relative density of the pipelines it was necessary to combine pipelines with similar characteristics (diameter, material, and age) before inserting them into the hydraulic model. This process resulted in GIS coverage of the distribution system represented by over 4,000 pipeline segments. The reduction of pipeline segments from the GIS coverage to the model did not change the hydraulic representation of the pipelines. Utilizing additional geographic boundaries and database coverages developed by the City, topographic, pressure zone, land use, and water use information were associated with the model file. After the base model was established a representation of each pump station, reservoir, and the supply source was developed.

Demand Allocation

Existing demands were allocated to the model using GIS coverage representing water meter locations and Utility Billing records of metered water demands from 1995 through 2000. Meter demands were then aggregated into hydraulic demand nodes. As a result, meter records were associated with 3,500 modeled demand nodes to represent the existing water demand in the distribution system.

Future demand projections are based on population and land use data, as presented in *Chapter 4 - Population and Water Demand Projections*. Water use factors considering population (gallons per capita per day) and land-use (gallons per acre per day) were applied to determine the total projected demand at buildout of the Urban Growth Boundary. A 1,000 foot by 1,000 foot grid was created and overlaid on the projected service area at buildout to establish a water demand per grid. Water demands within each grid were aggregated to project a demand per grid. A total of 709 grids were used in the model.

Model Calibration

The purpose of the model calibration is to provide a check to ensure that the model accurately represents the physical system. Typically a calibration within 10 percent of field collected information is satisfactory for a planning level model[70]. The WaterCAD model developed for this facility plan is within 10 percent of field data for all calibration tests as discussed below.

The calibration of the model was a "steady-state" calibration. That is, the model was calibrated assuming that flows, reservoir levels, pumping rates, and other system conditions occur at one point in time and does not consider time dependent factors. While this assumption is appropriate for planning purposes and determining the overall condition of the system for development of recommended improvement projects, it is not adequate as an ongoing operational tool. The City may want to expand how the model is used to evaluate alternative operating strategies. This will require development of an extended period simulation (EPS) model. An EPS model will better represent the filling and draining of storage reservoirs as well as the performance of pump stations during diurnal demand conditions.

Calibration of the hydraulic model consisted of conducting a series of hydrant flow tests during peak season demands and recording all pertinent system parameters during the testing period. This allowed boundary conditions to be set in the model so that pressure and flow parameters from the hydrant tests could be compared with model results under comparable operating conditions. The water distribution model was calibrated (within 10 percent of field tests) by adjusting pipeline roughness coefficients and minor losses. Pump operating curves, pressure regulating valve settings, and location and settings of isolation valves were also checked and verified during the calibration effort.

Flow testing was conducted at 17 locations throughout Albany's distribution system as shown in ***Figure 10-1***. These tests were performed on August 9, 10 and 14 in 2001 and a series of retests were performed on January 30 in 2002. 22 calibration points were initially considered but tests at sites 11, 13, 18 and 19 were not performed due to inability to isolate the distribution system properly and the results for the test performed at site 6 were not included due to an error in data recording. Retests were required at sites 1, 2, 7 and 22 because a large isolation valve in Pressure Zone 3 was found to be partially closed during initial testing. Site 12 was retested because of suspect test results.

At each location three hydrants were used for model verification. A flow hydrant was established to record flow rates with the hydrant fully open, while static (no flow) and residual (during flow) pressures were recorded at two secondary hydrants. Flow hydrants were isolated to allow unidirectional flow through both secondary hydrants immediately upstream. ***Figure 10-2*** shows a diagram of the typical testing setup. The pipeline characteristics and length between the two pressure hydrants were then used to develop a roughness coefficient to hydraulically represent the pipeline in the model. ***Tables 10-1*** and

[70] Mays, Larry (in association with AWWA), American Water Works Association (AWWA) Water Distribution System Handbook, McGraw-Hill, 2000.

10-2 and ***Figures 10-3*** and *10-4* summarize calibration results for the flow tests. ***Tables 10-1*** and *10-2* and ***Figures 10-3*** and *10-4* show the static and residual pressure condition recorded in the field compared to the model results that represent the field condition for each test site. The figures show that the 10 percent allowable error was never exceeded.

The most common parameter that complicates calibration of a hydraulic model is an error in the setting of isolation valves. Throughout a system network there may be numerous valves (i.e. isolation valves) that are unaccounted for and/or misrepresented. The setting of a valve may be represented in the model as fully opened when actually it is partially or fully closed. A partially closed valve can be very difficult to identify and may result in extreme hydraulic variations in the distribution system that are not accounted for in the model. As the Albany supply and distribution system changes over time and additional calibration efforts are made to the hydraulic model, the model analyst should consider closed or partially closed valves if future model calibration becomes difficult.

After calibration, the model was updated with projected demands and boundary conditions reflecting the planning criteria (i.e. pump stations at firm capacity, reservoirs at ¾ full or within 10 feet of overflow, whichever is higher) developed in *Chapter 5 - Planning Criteria*. Hydraulic deficiencies were then identified for existing and buildout conditions.

Pipeline Analysis

Planning criteria developed in *Chapter 5 – Planning Criteria*, were used to assess pipeline hydraulic capacities for current and projected water demands. Pipelines function as part of a network and are best evaluated through modeling for various water demand scenarios. Three demand scenarios were investigated for the current and buildout demand conditions:

- peak hour demands for distribution pipelines (velocity and head loss criteria),
- maximum day demands for transmission pipelines (velocity and head loss criteria), and
- maximum day demands plus fire flows for distribution and transmission pipelines (pressure criteria).

Using these scenarios, planning criteria presented in *Chapter 5 – Planning Criteria* were used to identify deficiencies in pipelines. These included:

Peak Hour Demand Scenario

Distribution pipelines (pipelines less than 16-inch diameter) with velocities greater than 10 fps and/or head losses greater than 10 ft per 1,000 ft.

Maximum Day Demand Scenario

Transmission pipelines (pipelines 16-inch diameter and larger) with velocities greater than 5 fps and/or head losses greater than 3 ft per 1,000 ft.

Maximum Day Demand Plus Fire Flow Scenario

Pressures less than 20 psi.

The total system maximum day demand (MDD) used in the existing system analysis was 16.0 MGD as defined in *Chapter 4 - Population and Water Demands*. The total system peak hour demand (PHD), which corresponds to a flow of twice the maximum day demand, was 32.0 MGD. The projected buildout water demand in year 2074 is a MDD of 40.0 MGD.

The hydraulic model was used to analyze the fire flow capacity of the transmission and distribution system. Fire flows were applied to every demand node in the hydraulic model based on unit fire flow requirements shown in ***Table 5-2***. The model was run using maximum day demands plus the applied fire flow at each demand node. The results were analyzed and nodes with a system pressure below 20 psig were identified for further analysis. These nodes were typically located in industrial, commercial, and institutional land use areas with 3,500 to 5,000 gpm fire flow requirements.

Typically, industrial, commercial, and institutional land uses rely on several fire hydrants to meet fire flow demands. The next step in the fire flow analysis accounted for this use of multiple fire hydrants to meet greater fire flows. Nodes not meeting fire flow requirements in the first model run were grouped together based on area to represent areas not meeting fire requirements. Albany's Engineering and Fire Departments worked together to select 30 locations from these groupings for further evaluation. Each location grouped 2 or 3 nodes and divided the fire flow demand equally to each node. In general, for fire flows of 3,500 gpm and greater 3 nodes were used and for fire flows less than 3,500 gpm 2 nodes were used. Although these 30 sites did not include every failed demand node, the selected locations were representative of the areas that failed to meet fire flow requirements in the first model run.

Once re-allocation of fire flow demands was completed the model was re-run and locations that still did not meet the 20 psig planning criteria were identified. Out of the 30 selected locations, 15 were identified as not meeting the minimum pressure criteria. Improvements required to correct the deficiency were identified at these locations and included in the list of recommended improvement projects.

EVALUATION RESULTS

Pipelines

The buildout system evaluation was the basis for the recommended pipeline improvement projects summarized at the end of this chapter and presented in *Chapter 12 - Recommended Plan*. Improvements identified through the existing system evaluation were compared to improvements identified though the buildout system evaluation in order to determine which projects are growth-related and which are related to existing system deficiencies. Alignments associated with recommended improvements are approximate and therefore project lengths may be slightly different when evaluated during the design process.

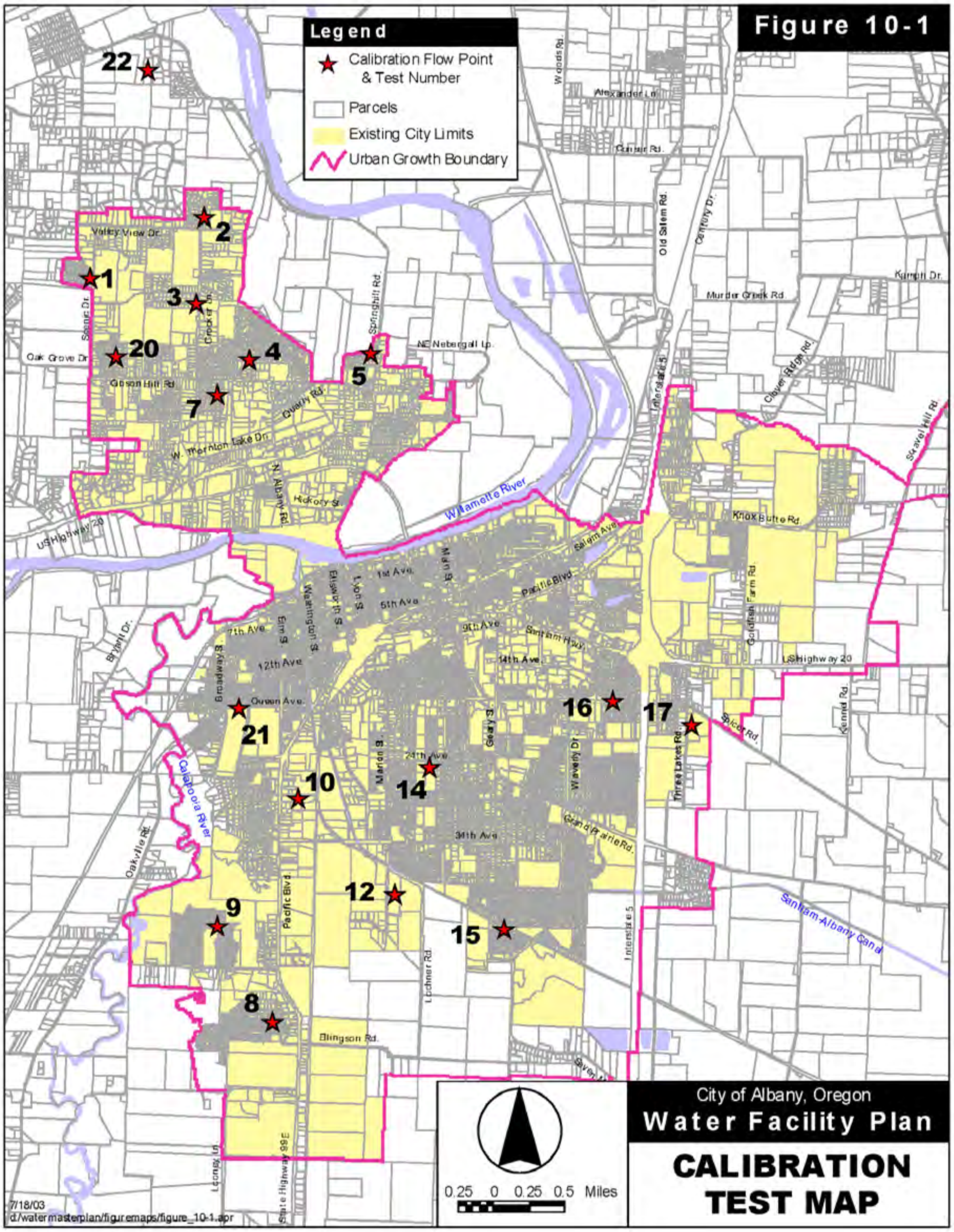


Figure 10-1
Legend
Calibration Flow Point & Test Number
Parcels
Existing City Limits
Urban Growth Boundary
City of Albany, Oregon
Water Facility Plan
CALIBRATION TEST MAP
0.25 0 0.25 0.5 Miles
7/18/03
d:/watermasterplan/figuremaps/figure_10-1.apr
Willamette River
Calapooia River
Santiam-Albany Canal
Knox Butte Rd.
US Highway 20
Pacific Blvd.
Queen Ave.
Ellingson Rd.
Lochner Rd.
Interstate 5
Three Lakes Rd.
Waverly Dr.
34th Ave.
Grand Prairie Rd.
Murder Creek Rd.
Old Salem Rd.
NE Nebergall Lp.
Springhill Rd.
Valley View Dr.
Oak Grove Dr.
Gibson Hill Rd.
W. Thornton Lake Dr.
Hickory St.
N. Albany Rd.
Salem Ave.
Santiam Hwy.
14th Ave.
12th Ave.
7th Ave.
5th Ave.
1st Ave.
9th Ave.
24th Ave.
Marion St.
Geary St.
Broadway St.
Elm St.
Washington St.
Ellsworth St.
Lyon St.
Main St.
Goldfish Farm Rd.
Oakville Rd.
Bryant Dr.
Spicer Rd.
Kennel Rd.
Century Dr.
Clover Ridge Rd.
Scravel Hill Rd.
Kamph Dr.
Alexander Ln.
Woods Rd.
Looney Ln.
State Highway 99E
1 2 3 4 5 7 8 9 10 12 14 15 16 17 20 21 22

Figure 10-2: Field Calibration Schematic

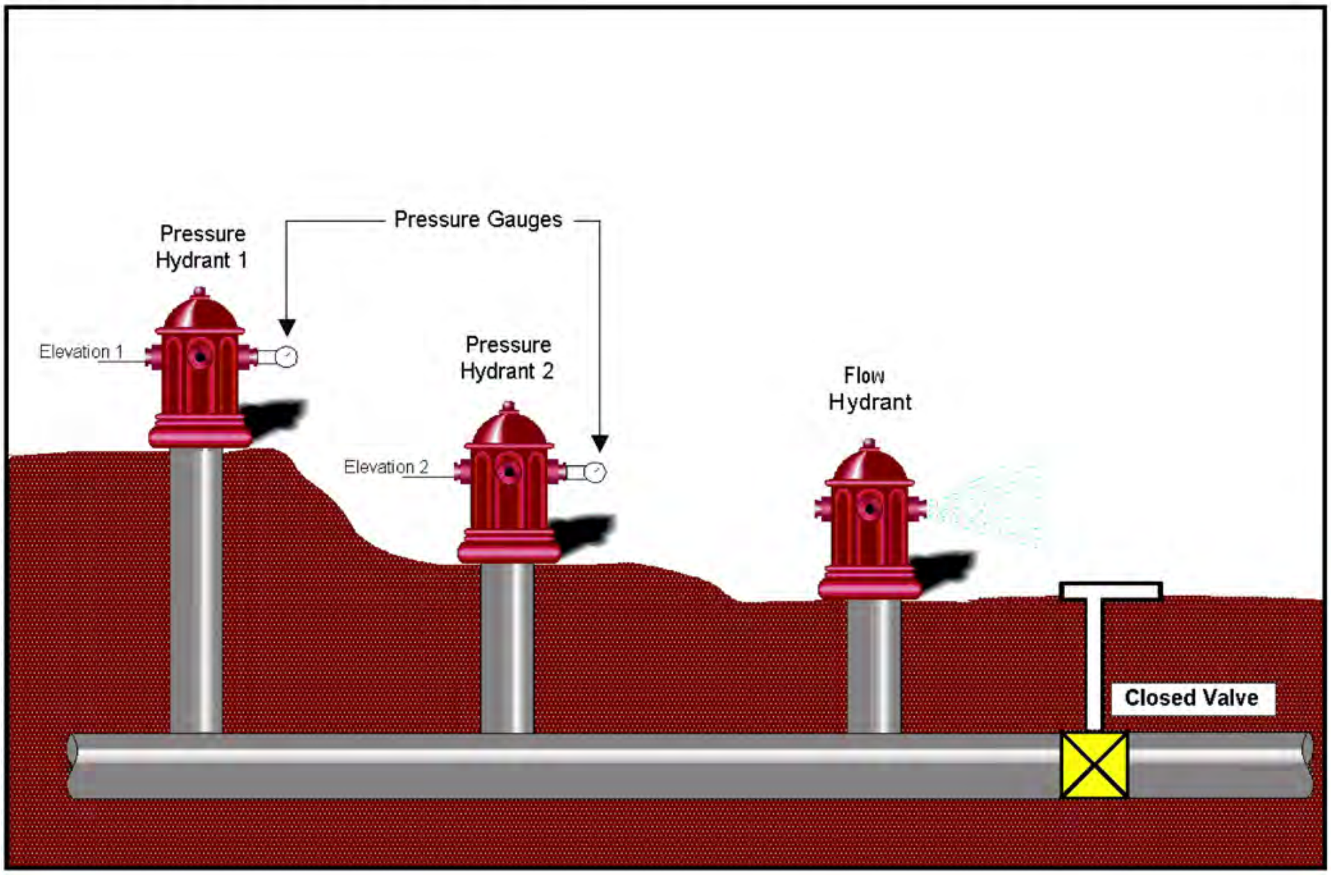


Table 10-1: Hydrant 1 Calibration Results

						Static Pressure (psig)			Residual Pressure (psig)		
Test No.	*Test Date*	*Fireflow (gpm)*	*Pressure Zone*	*Hydrant Location*	*Model Junction Node*	*Field*	*Model*	*Percent Difference*	*Field*	*Model*	*Percent Difference*
No. 1	January 30, 2002	1075	3	2575 Oak Grove Lp	J292	97.0	96.8	0.2	92.0	92.0	0.0
No. 2	January 30, 2002	770	3	3150 Crocker Lane	J3489	71.0	68.7	3.2	43.0	44.5	-3.5
No. 3	August 14, 2001	890	2	2290 Robin Hood Lane NW	J491	82.0	79.9	2.5	43.0	44.9	-4.3
No. 4	August 10, 2001	1125	2	1228 19th Ave NW	J442	66.0	66.9	-1.4	58.0	63.8	-10.0
No. 5	August 10, 2001	890	1	117 Country Club Lane	J87	80.0	84.4	-5.5	62.0	65.7	-6.0
No. 7	January 30, 2002	750	3	1925 Sarah Ave NW	J514	96.0	92.8	3.4	69.0	65.4	5.3
No. 8	August 10, 2001	890	1	1038 Morse Lane SW	J3405	65.6	68.7	-4.7	60.0	60.7	-1.1
No. 9	August 9, 2001	965	1	1670 Cougar Ave SW	J3323	76.0	71.1	6.4	55.0	58.3	-6.0
No. 10	August 9, 2001	920	1	29th Ave SW, Oregon Freeze Dry	J3184	77.0	79.1	-2.8	73.0	76.3	-4.5
No. 12	January 30,2002	840	1	3815 Marion St SE	J3484	66.0	65.5	0.8	62.0	57.3	7.6
No. 14	August 9, 2001	805	1	830 24th Ave SE	J2393	62.0	65.8	-6.2	33.0	32.8	0.7
No. 15	August 10, 2001	710	1	4662 Columbus St SE	J2685	56.0	55.1	1.6	50.0	45.3	9.5
No. 16	August 9, 2001	1040	1	3137 18th Ave SE	J3494	68.0	65.9	3.1	64.0	58.2	9.0
No. 17	August 9, 2001	950	1	2151 3 Lakes Road SE	J2012	64.0	63.7	0.5	57.0	52.5	7.9
No. 20	August 10, 2001	890	2	Ravenwood Dr NW	J365	72.0	71.1	1.3	64.0	65.8	-2.8
No. 21	August 9, 2001	1160	1	1450 Queen Ave SW	J3053	75.0	78.4	-4.5	72.0	71.3	1.0
No. 22	January 30, 2002	605	3	5140 Winn Drive	J3492	148.0	146.7	0.9	59.0	61.2	-3.8

Note: Tests 11, 13, 18 and 19 not performed. Results for Test 6 not used due to error in data recording.

Table 10-2: Hydrant 2 Calibration Results

						Static Pressure (psig)			Residual Pressure (psig)		
Test No.	Test Date	Fireflow (gpm)	Pressure Zone	Hydrant Location	Model Junction Node	Field	Model	Percent Difference	Field	Model	Percent Difference
No. 1	January 30, 2002	1075	3	2717 Quince St NW	J295	82.0	81.0	1.2	76.0	74.6	1.8
No. 2	January 30, 2002	770	3	2310 Woodcrest Ave	J234	82.0	82.5	-0.6	51.0	55.7	-9.2
No. 3	August 14, 2001	890	2	2299 Squire St NW	J239	74.0	69.6	5.9	34.0	33.1	2.7
No. 4	August 10, 2001	1125	2	1880 Gibson Way NW	J444	78.0	78.2	-0.2	70.0	73.8	-5.4
No. 5	August 10, 2001	890	1	156 Country Club Lane	J3485	78.0	84.0	-7.6	58.0	60.0	-3.5
No. 7	January 30, 2002	750	3	1501 N. Ranch Dr. NW	J3488	96.0	98.4	-2.5	68.0	70.7	-4.0
No. 8	August 10, 2001	890	1	1290 Morse Lane SW	J3401	66.0	70.2	-6.3	58.0	55.7	4.0
No. 9	August 9, 2001	965	1	1846 Cougar Ave NW	J3325	73.0	71.3	2.3	49.0	51.5	-5.1
No. 10	August 9, 2001	920	1	770 29th Ave SW	J3188	77.0	75.5	1.9	69.0	70.8	-2.6
No. 12	January 30,2002	840	1	3815 Marion St SE	J3483	67.0	65.0	3.0	62.0	56.3	9.2
No. 14	August 9, 2001	805	1	705 24th Ave SE	J2403	63.0	67.1	-6.5	34.0	32.3	5.1
No. 15	August 10, 2001	710	1	1968 47th Ave SE	J2684	57.0	55.5	2.7	42.0	43.4	-3.4
No. 16	August 9, 2001	1040	1	3237 18th Ave SE	J3495	66.0	65.9	0.2	61.0	54.9	9.9
No. 17	August 9, 2001	950	1	2151 3 Lakes Road SE	J2013	60.0	63.9	-6.6	49.0	50.6	-3.3
No. 20	August 10, 2001	890	2	1835 Ravenwood Dr NW	J3487	72.0	71.5	0.7	59.0	64.7	-9.6
No. 21	August 9, 2001	1160	1	1450 Queen Ave SW	J2975	71.0	75.5	-6.3	68.0	67.4	1.0
No. 22	January 30, 2002	605	3	5430 Winn Dr	J3491	150.0	148.5	1.0	51.0	55.0	-7.8

Note: Tests 11, 13, 18 and 19 not performed. Results for Test 6 not used due to error in data recording.

Figure 10-3: Recording Hydrant No. 1 Calibration Results

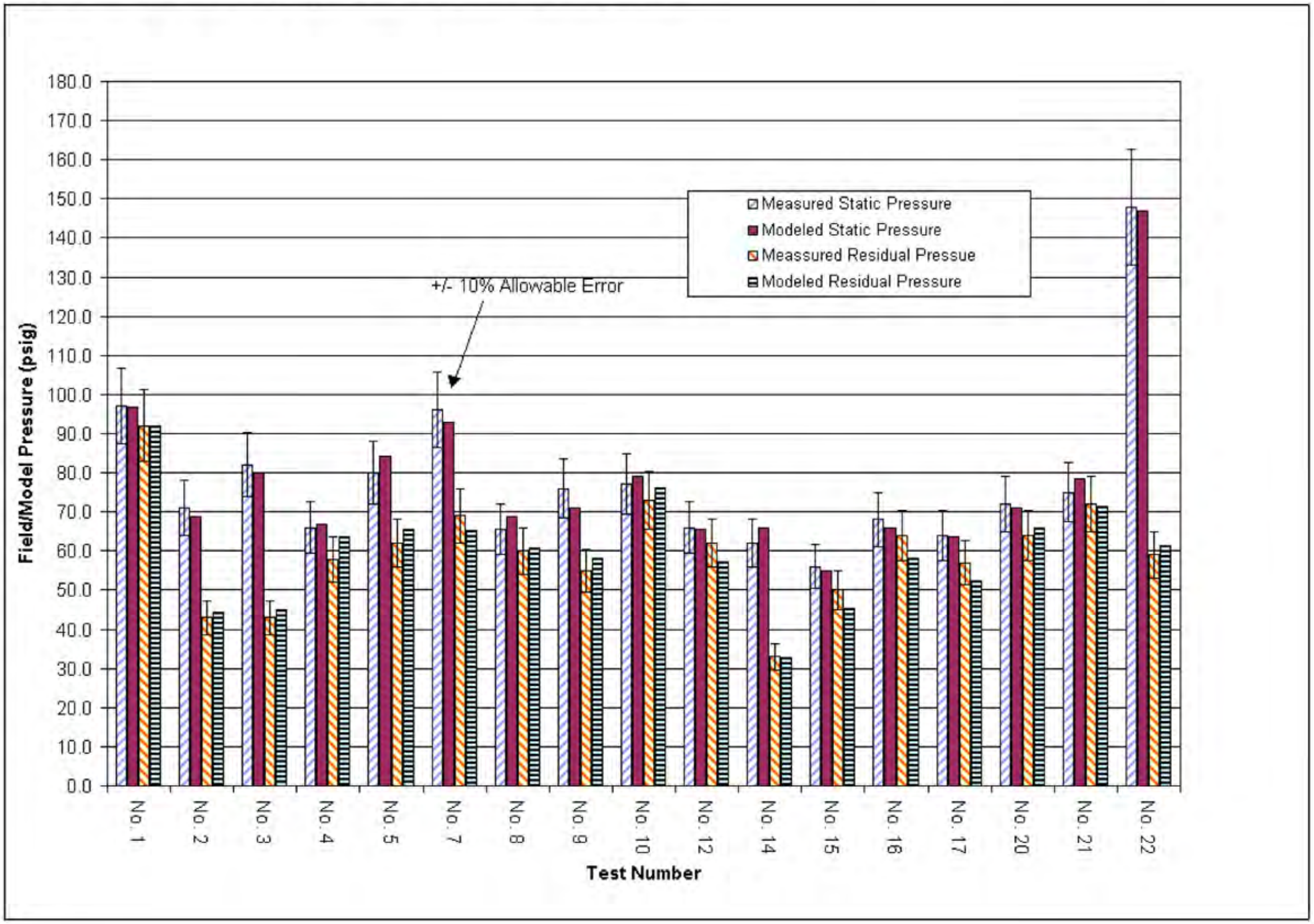
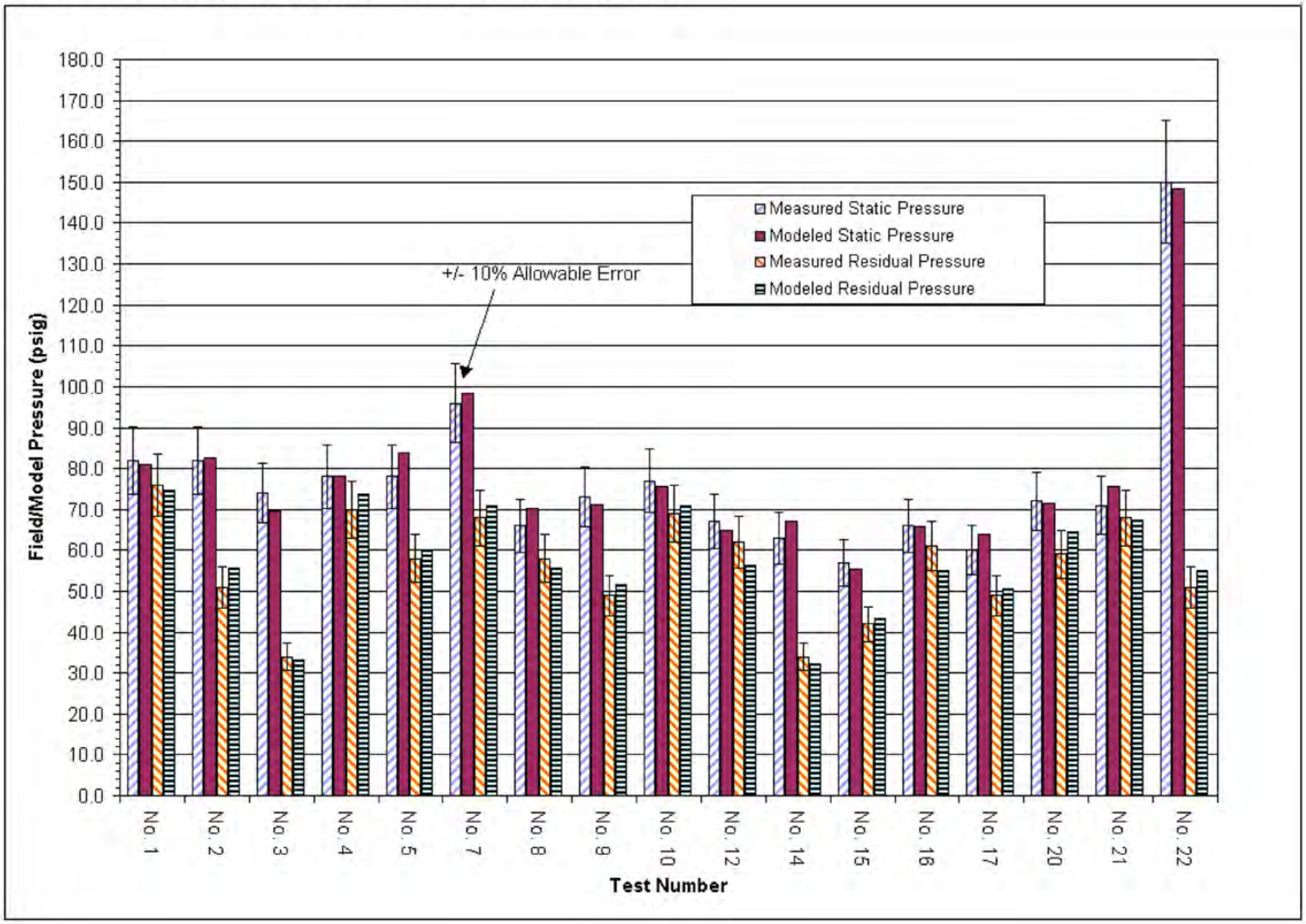


Figure 10-4: Recording Hydrant No. 2 Calibration Results



Existing System

The existing pipeline network was evaluated based on current water demands. The evaluation was done using a multiple-step process, shown in ***Figure 10-5.*** The process began with the hydraulic model of Albany's existing water system. The first step was to evaluate the system under existing maximum day demand conditions. Improvement projects were identified for all pipes violating planning criteria and the model was updated to reflect these improvements. The next step involved analyzing the updated model for the existing peak hour demands. Again, improvement projects were identified and the model was updated to reflect these improvements. The final step was to identify deficiencies based on maximum day demand plus fire flow conditions. Deficiencies and required improvements to meet only existing needs are included in ***Appendix I.*** This evaluation did not consider growth-related needs and was done to identify existing system deficiencies and possible remedies to assist in cost allocations used in the water financial plan.

Buildout System

As shown in ***Figure 10-5***, evaluation of Albany's water system at buildout began with a model of the existing distribution system. As previously mentioned, improvements identified through the existing system model were used to identify existing system deficiencies but were not used in evaluating the buildout system. The Scravel Hill WTP, discussed in *Chapter 9 – Joint Water Project*, and approximate locations of future piping and reservoirs necessary to serve future development, were added to the model to represent the buildout water system. Pipelines were positioned in existing road right-of-ways or in future right-of-way identified in Albany's *Transportation System Plan*.[71] Once the configuration of the buildout water system was determined, the model was run to evaluate the system under buildout maximum day demand conditions. Improvements were identified for all pipes violating planning criteria and the model was updated to reflect these improvements. The next step involved analyzing the updated model for peak hour demands at buildout. Again, improvements were identified and the model was updated to reflect these improvements. The final step was to apply the fire flow analysis for maximum day demand plus fire flow conditions and identify necessary improvements.

Resulting distribution system improvement projects with like characteristics were grouped together and named for ease in identification. ***Figure 10-6*** shows project groupings and the following project descriptions discuss why these projects are required. Finished water transmission piping that will be constructed as part of the Joint Water Project is discussed in *Chapter 9 – Joint Water Project*.

East End Transmission Projects (Project Numbers P1, P2, P3, P4, P5, P6, & P7)

The East End Transmission Project is required to fully utilize the initial 10 MGD capacity generated for Albany by the Scravel Hill Water Treatment Plant (*Chapter 9 - Joint Water Project*). These transmission lines include approximately 10,300 feet of 24, 20, 12, and 8-inch water lines. These water lines are also needed to raise service pressures in the southeast Albany area.

[71] Kimley-Horn and Associates, Inc., *City of Albany, Oregon Transportation System Plan*, 1997

South Albany Transmission Project (Project Number P8)

The South Albany Transmission Project consists of a 16-inch transmission line that begins at the intersection of Pacific Boulevard and 34th Avenue and ends at Cougar Avenue. This transmission line is needed to improve fire flows and service pressures, and to provide a redundant supply line to the southwest Albany area.

North Albany Distribution Projects (Project Numbers P9, P10, P11, P12, P13, P14, P15, P16, PS1, PS2, & PS11)

The North Albany Distribution Projects include upsizing approximately 14,300 feet (750 feet of steel) of undersized water lines to 8 and 12-inch water lines in order to meet fire flow requirements. This project group also includes a pump station and piping necessary to create a fourth pressure zone that is discussed later in this chapter. In addition, this project group includes a Pressure Regulating (Reducing) Valve (PRV) in Zone 3. The valve location is along the pipeline serving NW Winn Drive in North Albany outside the UGB. The valve should be set to maintain a maximum pressure of 80 psig within the regulated area and be able to pass MDD + Fire Flow while maintaining a minimum pressure of 20 psi throughout the regulated area.

Zone 1 Distribution Projects (Project Numbers P17, P18, P19, P20, P21, P22, & P23)

Zone 1 Distribution Projects include approximately 2,700 feet of water lines necessary to meet fire flow requirements and approximately 1,500 feet of water lines required to meet future peak hour and maximum day demands. These projects are dispersed throughout pressure Zone 1.

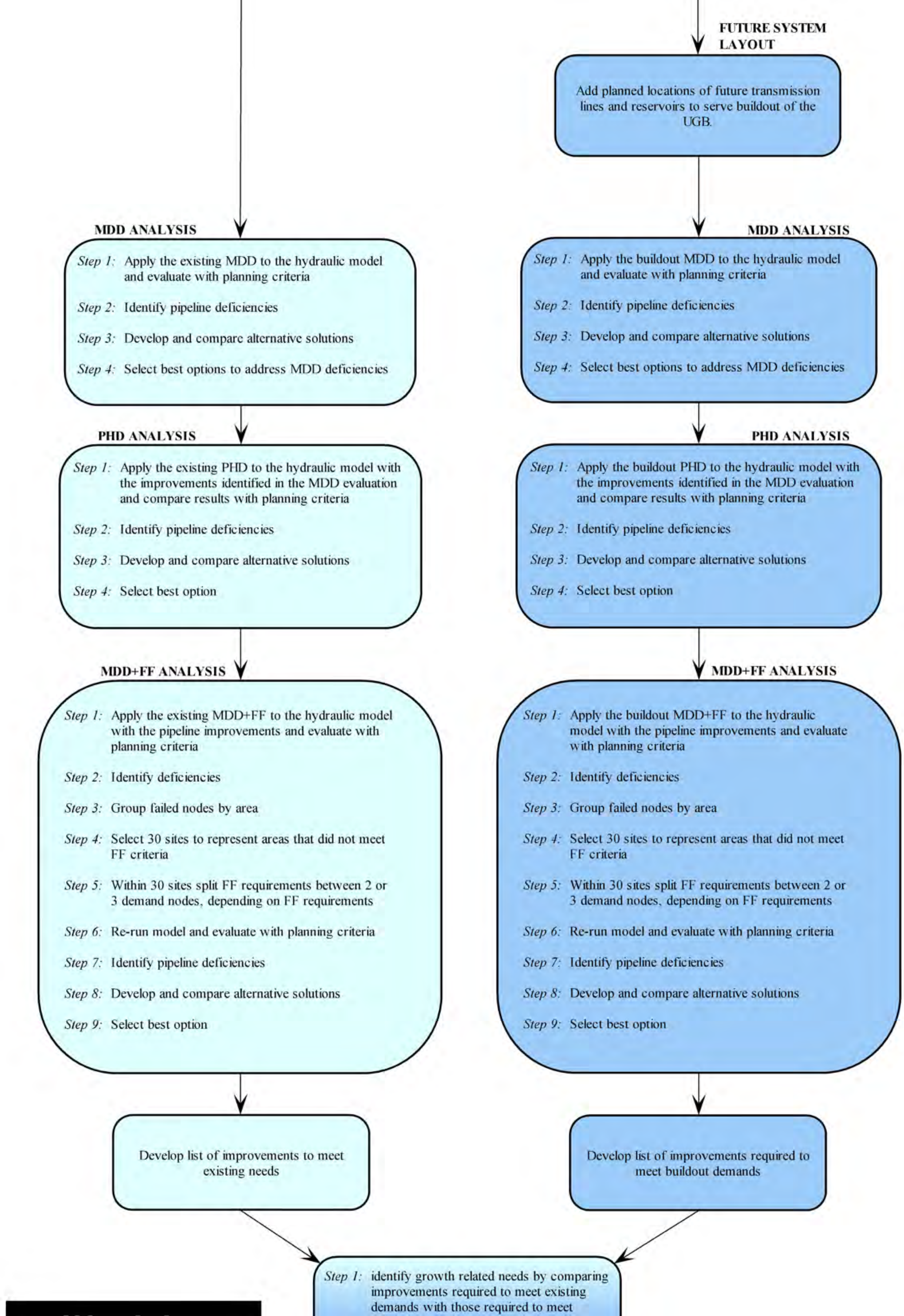
Ellingson Road Reservoir Project (Project Numbers P24 & S6)

The Ellingson Road Reservoir Project includes a 2 MG elevated reservoir, discussed later in this chapter, located on Ellingson Road and approximately 2,100 feet of 16-inch pipeline necessary to connect the reservoir to the existing water line on Pacific Boulevard. This project is needed to meet future storage requirements in Zone 1 and will provide local fire protection storage and enhanced service pressures in the Southwest Albany area.

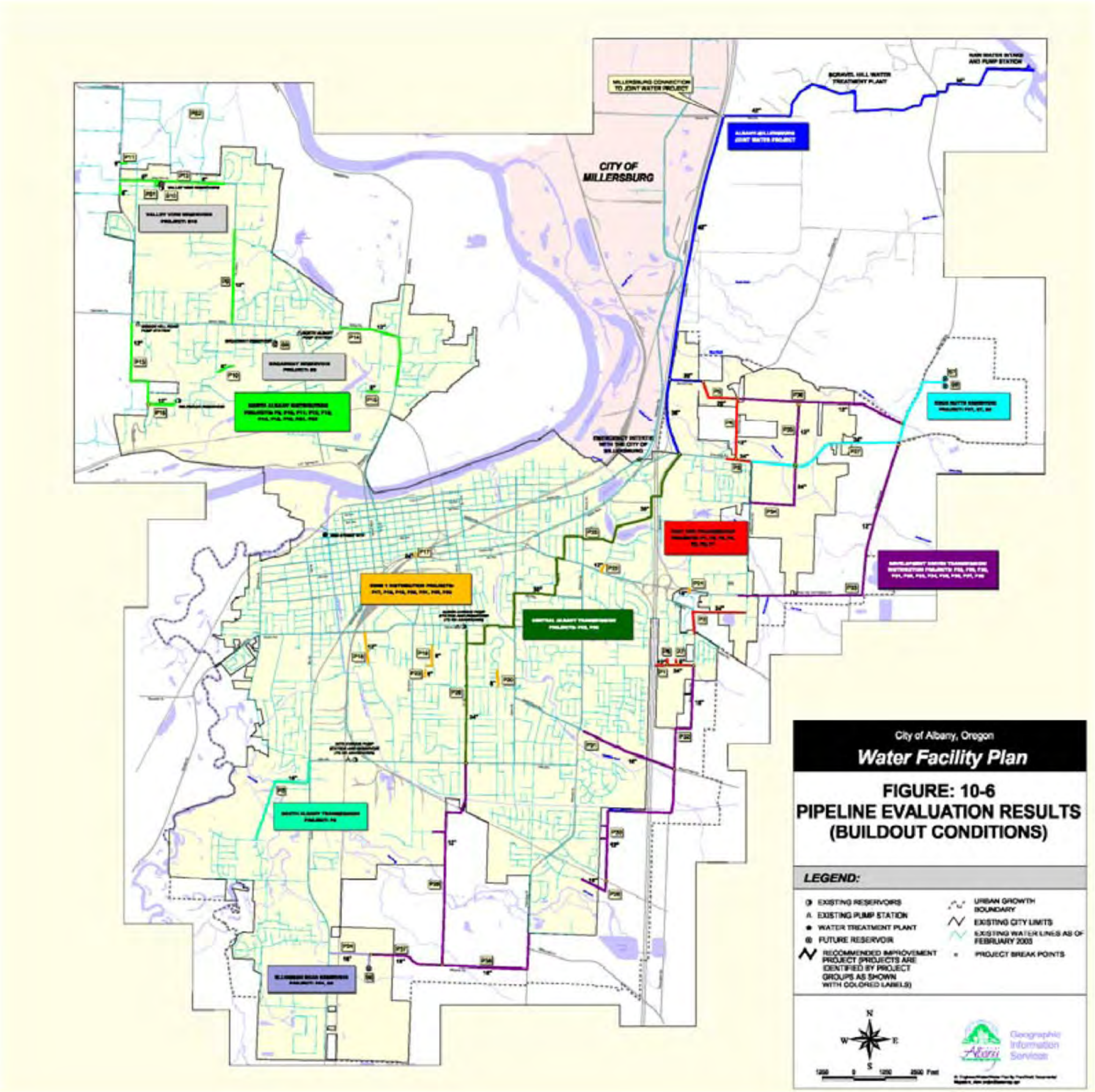
If the City elects to allow at-grade pumped storage in the future, this site should be considered for a larger at-grade storage reservoir because of its proximity to high demand customers. A larger reservoir at this site would provide a more even distribution of finished water storage throughout pressure Zone 1.

Central Albany Transmission Project (Project Numbers P25 & P26)

The Central Albany Transmission Project is required to meet future maximum day demand conditions and is also required to realize the benefit of future Scravel Hill WTP expansions. The project consists of approximately 14,300 feet of 30-inch water line from Knox Butte Road to Main Street and approximately 6,700 feet of 24-inch water line from Queen Avenue to 34th Avenue. This project incorporates the replacement of approximately 1.25 miles of deteriorated steel water lines. Alternate alignments for the 30-inch water line could be investigated if the City decides to incorporate this transmission project with steel pipeline replacement along Pacific Boulevard.



FUTURE SYSTEM LAYOUT
Add planned locations of future transmission lines and reservoirs to serve buildout of the UGB.
MDD ANALYSIS
Step 1: Apply the existing MDD to the hydraulic model and evaluate with planning criteria
Step 2: Identify pipeline deficiencies
Step 3: Develop and compare alternative solutions
Step 4: Select best options to address MDD deficiencies
MDD ANALYSIS
Step 1: Apply the buildout MDD to the hydraulic model and evaluate with planning criteria
Step 2: Identify pipeline deficiencies
Step 3: Develop and compare alternative solutions
Step 4: Select best options to address MDD deficiencies
PHD ANALYSIS
Step 1: Apply the existing PHD to the hydraulic model with the improvements identified in the MDD evaluation and compare results with planning criteria
Step 2: Identify pipeline deficiencies
Step 3: Develop and compare alternative solutions
Step 4: Select best option
PHD ANALYSIS
Step 1: Apply the buildout PHD to the hydraulic model with the improvements identified in the MDD evaluation and compare results with planning criteria
Step 2: Identify pipeline deficiencies
Step 3: Develop and compare alternative solutions
Step 4: Select best option
MDD+FF ANALYSIS
Step 1: Apply the existing MDD+FF to the hydraulic model with the pipeline improvements and evaluate with planning criteria
Step 2: Identify deficiencies
Step 3: Group failed nodes by area
Step 4: Select 30 sites to represent areas that did not meet FF criteria
Step 5: Within 30 sites split FF requirements between 2 or 3 demand nodes, depending on FF requirements
Step 6: Re-run model and evaluate with planning criteria
Step 7: Identify pipeline deficiencies
Step 8: Develop and compare alternative solutions
Step 9: Select best option
MDD+FF ANALYSIS
Step 1: Apply the buildout MDD+FF to the hydraulic model with the pipeline improvements and evaluate with planning criteria
Step 2: Identify deficiencies
Step 3: Group failed nodes by area
Step 4: Select 30 sites to represent areas that did not meet FF criteria
Step 5: Within 30 sites split FF requirements between 2 or 3 demand nodes, depending on FF requirements
Step 6: Re-run model and evaluate with planning criteria
Step 7: Identify pipeline deficiencies
Step 8: Develop and compare alternative solutions
Step 9: Select best option
Develop list of improvements to meet existing needs
Develop list of improvements required to meet buildout demands
Step 1: identify growth related needs by comparing improvements required to meet existing demands with those required to meet



MILLERSBURG CONNECTION TO JOINT WATER PROJECT
CITY OF MILLERSBURG
City of Albany, Oregon
Water Facility Plan
FIGURE: 10-6
PIPELINE EVALUATION RESULTS
(BUILDOUT CONDITIONS)
LEGEND:
EXISTING RESERVOIRS
EXISTING PUMP STATION
WATER TREATMENT PLANT
FUTURE RESERVOIR
RECOMMENDED IMPROVEMENT PROJECT (PROJECTS ARE IDENTIFIED BY PROJECT GROUPS AS SHOWN WITH COLORED LABELS)
URBAN GROWTH BOUNDARY
EXISTING CITY LIMITS
EXISTING WATER LINES AS OF FEBRUARY 2003
PROJECT BREAK POINTS
N
W
E
S
Feet
Geographic Information Services

Knox Butte Reservoir Project, Phase 1 (Project Numbers P27, S7)

The Knox Butte Reservoir Project, Phase 1, includes a 5-MG Knox Butte concrete storage reservoir, discussed later in this chapter, and approximately 9,700 feet of 24-inch water line necessary to connect it to the distribution system. This reservoir is needed to meet future storage requirements in Zone 1. When the Knox Butte Reservoir is put into service the reservoir will be hydraulically favored over both the Broadway and Ellingson Reservoirs. To balance the system hydraulically after the Knox Butte Reservoir is put into service either two valves will need to be closed or a flow control valve will need to put on the waterline serving Albany from the Scravel Hill WTP. Decisions on flow control from the Scravel Hill WTPwill be made during final design of the JWP. Without knowing the outcome of the final design process this plan is based on closing two valves in order to hydraulically balance the system. The first valve to isolate is along Knox Butte Road west of Timber Street and east of I-5. The second valve is located along the proposed 20-inch diameter pipeline along Bernard Avenue. By closing and isolating these two valves the system will become hydraulically balanced.

Development Driven Transmission/Distribution Projects (Project Numbers P28, P29, P30, P31, P32, P33, P34, P35, P36, P37, & P38)

The Development Driven Transmission/Distribution Projects include approximately 11 miles of 12, 16, and 24-inch pipelines needed to serve future development. Timing for these projects is development dependent.

Replacement Programs

As part of the pipeline evaluation process, three pipeline replacement programs were identified. These programs are the:

- Steel Pipeline Replacement Program
- Undersized Pipelines with Hydrants Replacement Program
- Perpetual Life Pipeline Replacement Program.

Steel Pipeline Replacement Program (Program 1)

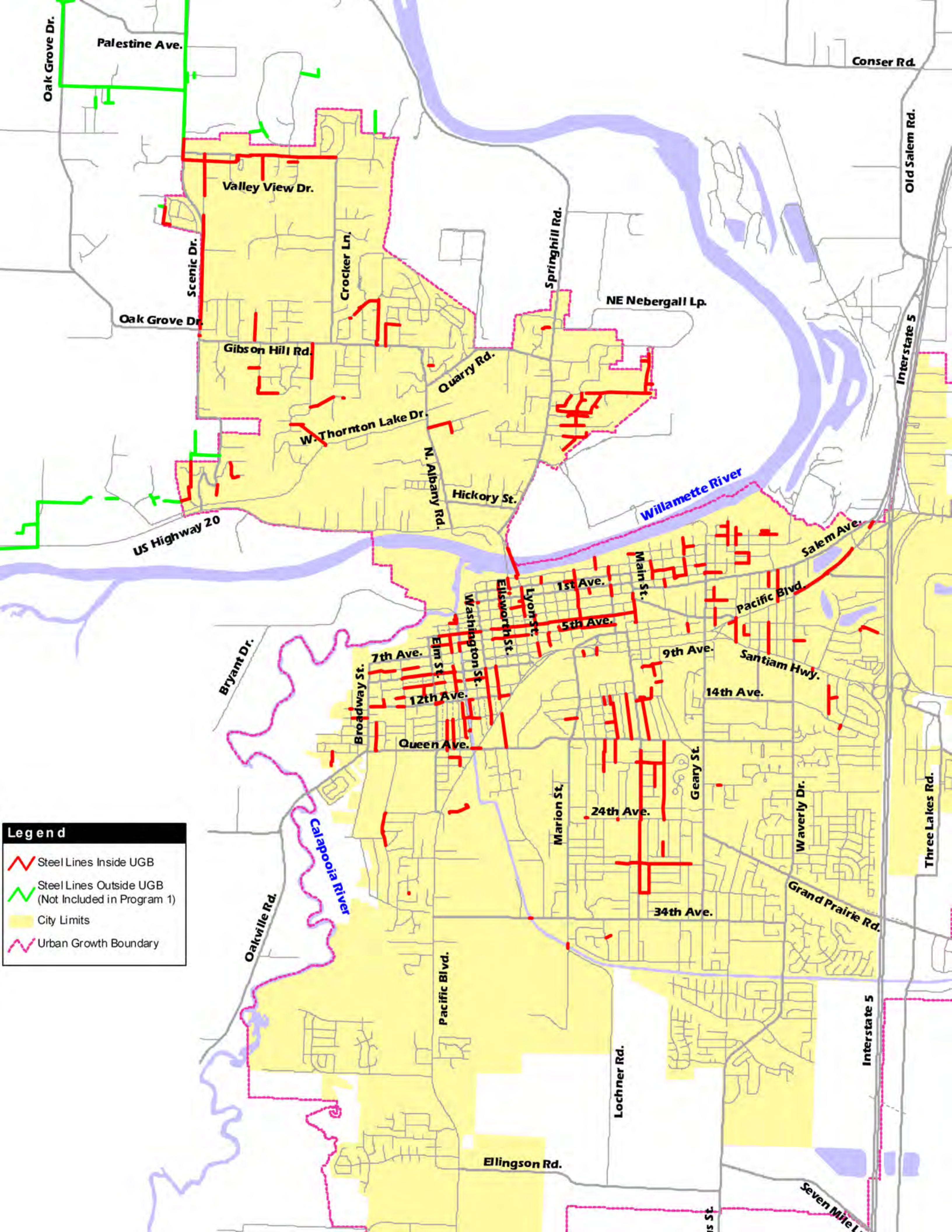
In January 2001 the City completed a steel pipe inventory and developed replacement strategies to effectively manage system maintenance efforts and reduce system water loss due to leakage[72]. Most of the City's water line leaks are thought to occur in steel pipes (steel pipes being classified as wrought iron, galvanized iron, steel, outside diameter dipped and wrapped steel, and unknown pipe types). These pipes, mainly installed prior to 1955, have exceeded their service life and must be replaced. The City has been replacing failing steel water lines as funds are made available but approximately 28.7 miles of steel water lines still exist within Albany's water system; 24.0 miles within Albany's urban growth boundary (UGB), 1.4 miles of which will be replaced though the improvement projects listed above, and 4.7 miles located outside the UGB. These pipes continue to experience failures and maintenance crews often do numerous temporary repairs to a failing steel water line before funds become available for replacement. This plan recommends replacing all steel pipes by 2020. ***Figure 10-7*** depicts the locations of the remaining 28.7 miles of steel water lines.

A replacement diameter of 8-inches was used to determine replacement costs. Required pipelines replacements based on model results are not included in this program and pipelines located outside the UGB have not been included in program costs.

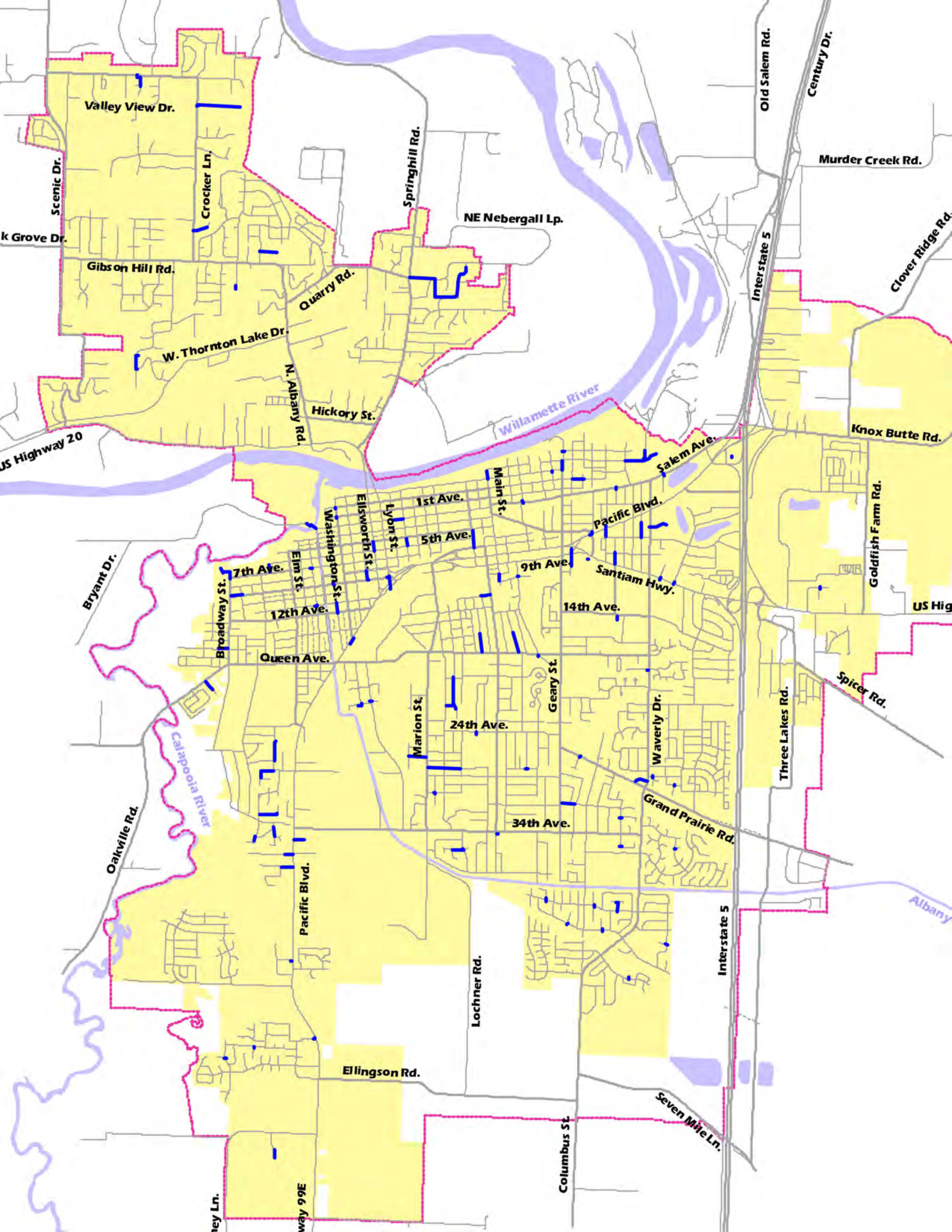
Undersized Pipelines with Hydrants Replacement Program (Program 2)

The Undersized Pipelines with Hydrants Replacement Program is needed to replace small waterlines unable to provide necessary fire flows to existing hydrants. This program includes lines that are 6-inches in diameter, non-steel, non-looped, and support a hydrant, or are less than 6-inches in diameter, non-steel, and support a hydrant. This program will replace approximately 25,600 feet, or 4.8 miles, of these undersized lines by 2025. Only the length required to reach the hydrant from an adequately sized water line is considered for replacement. ***Figure 10-8*** depicts the locations of these undersized water lines. Lines located outside of the UGB and required pipeline replacements based on model results are not included in this program. The replacement diameter used for estimating program costs was 8-inches.

[72] Young, Jim *City of Albany Steel Pipe Replacement Report*, January 2001



Palestine Ave.
Oak Grove Dr.
Conser Rd.
Old Salem Rd.
Valley View Dr.
Scenic Dr.
Crocker Ln.
Springhill Rd.
NE Nebergall Lp.
Oak Grove Dr.
Gibson Hill Rd.
Quarry Rd.
Interstate 5
W. Thornton Lake Dr.
N. Albany Rd.
Hickory St.
Willamette River
US Highway 20
Salem Ave.
Main St.
1st Ave.
Ellsworth St.
Lyon St.
Washington St.
Pacific Blvd.
5th Ave.
Bryant Dr.
7th Ave.
Elm St.
9th Ave.
Santiam Hwy.
Broadway St.
12th Ave.
14th Ave.
Queen Ave.
Geary St.
Marion St.
24th Ave.
Waverly Dr.
Three Lakes Rd.
Calapooia River
Grand Prairie Rd.
34th Ave.
Oakville Rd.
Pacific Blvd.
Lochner Rd.
Interstate 5
Ellingson Rd.
Seven Mile L
Legend
Steel Lines Inside UGB
Steel Lines Outside UGB (Not Included in Program 1)
City Limits
Urban Growth Boundary



Valley View Dr.
Scenic Dr.
Crocker Ln.
k Grove Dr.
Gibson Hill Rd.
Quarry Rd.
W. Thornton Lake Dr.
N. Albany Rd.
Hickory St.
Springhill Rd.
NE Nebergall Lp.
Old Salem Rd.
Century Dr.
Murder Creek Rd.
Interstate 5
Clover Ridge Rd.
Willamette River
US Highway 20
Knox Butte Rd.
Salem Ave.
Main St.
1st Ave.
Ellsworth St.
Lyon St.
Washington St.
5th Ave.
Pacific Blvd.
Goldfish Farm Rd.
Bryant Dr.
7th Ave.
Elm St.
9th Ave.
Santiam Hwy.
Broadway St.
12th Ave.
14th Ave.
US Hig
Queen Ave.
Geary St.
Spicer Rd.
Calapooia River
Marion St.
24th Ave.
Waverly Dr.
Three Lakes Rd.
Oakville Rd.
Grand Prairie Rd.
34th Ave.
Pacific Blvd.
Albany
Interstate 5
Lochner Rd.
Ellingson Rd.
Seven Mile Ln.
Columbus St.
ey Ln.
way 99E

Perpetual Life Pipeline Replacement Program (Program 3)

The Perpetual Life Pipeline Replacement Program was developed to plan for replacement of water lines as they reach the end of their expected service life. This program considers the estimated service life of the existing water lines in Albany's water system and schedules their replacement. Ductile iron water lines were considered to have a 100-year service life and all other pipe types were considered to have a 75-year service life. Pipelines included in the previous two programs and required pipeline replacements based on model results are not included as part of this program. This program identifies approximately 15 miles of pipe that will need to be replaced by 2025 and another 94 miles by 2074 (buildout). An estimated replacement diameter of 8-inches was used for establishing replacement costs of lines less than 16-inches in diameter and an estimated replacement diameter of 20-inches was used for establishing replacement costs of lines 16-inches and larger in diameter.

Pump Stations

Albany's water system has five pump stations located in three different pressure zones as shown in ***Figure 3-2***. Each pumping facility was evaluated for capacity, general condition, safety, and operating efficiencies. This was completed through a capacity evaluation and a field study conducted in June 2001. *Chapter 3 - Existing System Description*, includes a description of each pump station.

Comparisons of existing pump station capacities to pumps station demands at buildout are shown in ***Table 10-3***. The firm capacity indicated in ***Table 10-3*** is the rated capacity with the largest pump out of service. Actual capacity will be different from the rated firm capacity because pumps are operated at different points on their operating curve. Although pump station capacities are adequate for current maximum day demand conditions (see ***Table 3-6***), additional pumping capacity will need to be added at the North Albany and Gibson Hill Road Pump Stations as demands increase.

It should be noted that pumps in Zones 1, 2 and 3 are required to have an additive flow rate inclusive of each upper zone. Therefore, the Zone 2 pump station will be required to meet the maximum day demand of Zone 2 and 3. The Zone 1 pump stations (HSPSs at the Vine Street WTP and Joint Project WTP) will be required to pump the entire system's maximum day demand at firm capacity.

Recommendations for each facility are summarized in the following sections and field inspection forms are included in ***Appendix E***. References to facility condition reflect conditions as noted at the time of the field inspection. The HSPS is considered part of the Vine Street WTP and therefore the detailed evaluation of its components is located in *Chapter 8 – Vine Street WTP*. However, an overall capacity evaluation of the HSPS is included in ***Table 10-3***.

Queen and 34th Avenue pump stations and reservoirs are pumped storage systems; this type of storage is not as reliable as gravity feed systems. Therefore continued reliance upon pumped storage is not considered in this plan. However, the City's existing storage deficit does not allow for their immediate abandonment. Timing for abandonment is dependent on the construction of new non-pumped storage reservoirs and is estimated in the storage evaluation of this chapter. Based on projected demands and timing for additional storage interim improvements are recommended for these facilities.

34^th^ Avenue Pump Station (Project Number PS3)

This pump station works with the 34th Avenue Reservoir to boost service pressures to customers in the vicinity of the reservoir. Currently, this station is not equipped with a backup power supply rendering the reservoir useless during a power outage. This plan recommends installing a backup power outlet at this station. Abandonment of this pump station is discussed in the storage evaluation section of this chapter.

Queen Avenue Pump Station (Project Numbers PS4, PS5, PS6 & PS7)

The Queen Avenue Pump Station is located at the intersection of Hill Street and Queen Avenue. The pump house faces and is in close proximity to Queen Avenue. This plan recommends that a security evaluation be made of this facility to better protect the station as it is located in a highly traveled area. Recommendations made from the Brown and Caldwell study include replacing the pressure regulating control valve with a motorized control valve and replacing the 30 HP pump (Pump No. 21) with a 30 HP pump with an operating curve covering a broader range of flows and operating heads.

Similar to the 34th Avenue Pump Station and Reservoir, this pump station works with the Queen Avenue Reservoir to boost service pressures to customers in the vicinity of the reservoir. Currently, this station is not equipped with a backup power supply rendering the reservoir useless during a power outage. This plan recommends installing a backup power outlet at this station. Abandonment of this pump station is discussed in the storage evaluation section of this chapter.

North Albany (Zone 2) Pump Station (Project Number PS12)

Table 10-3 shows that the existing pumping capacity of the North Albany Pump Station is insufficient to meet buildout demands. This plan recommends increasing the pump impeller size to increase the pump capacity when needed to meet future water demands.

Gibson Hill Road (Zone 3) Pump Station (Project Number PS13)

Similar to the North Albany Pump Station, the existing pumping capacity of the Gibson Hill Road Pump Station is insufficient to meet buildout demands. This plan recommends increasing the pump impeller size to increase the pump capacity when needed to meet future water demands.

Future Zone 4 Pump Station (Project Numbers PS1 & PS11)

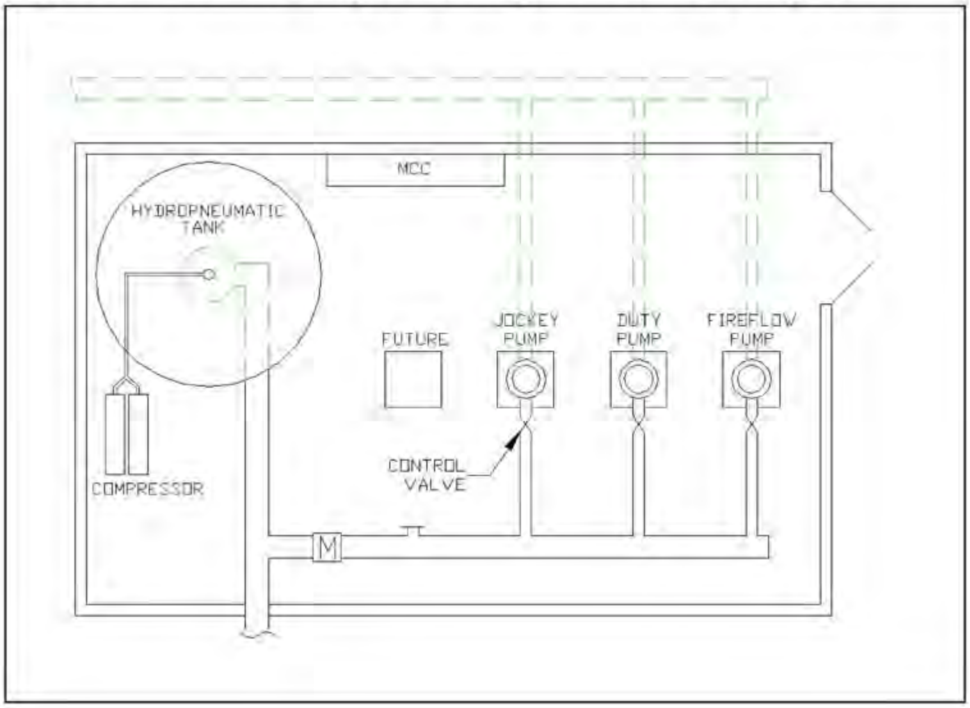
As discussed in *Chapter 3* - *Existing System Description*, a low-pressure area exists at the upper elevations of Zone 3. The surrounding topography is such that if the reservoir levels drop below an elevation of 560 feet some customers will experience lower than desired water pressures. To better serve these customers a 4th pressure zone should be created. Initially, this would require a 7.5 hp booster pump station plus a fire flow pump to be installed at the Valley View Reservoir site. A general pump station layout is shown in ***Figure 10-9.*** This zone separation is discussed in more detail under existing storage requirements.

Table 10-3: Pump Station Capacities and Demands

Pressure Zone	*Facility Name*	*Total Installed HP*	*Source*	*Service Area/Reservoir*	*Firm Capacity Flow rate (gpm)*	*Total Nominal Flow rate (gpm)*	*Buildout Demand (gpm)*	*Surplus or (Deficit) (gpm)*
Zone 1	**High Pressure Pump Station**	1,050	Vine Street WTP	Zone 1 Broadway, Queen and 34th Reservoir	15,950	22,650	13,889	2,061
Zone 1	**Queen Avenue**[1]	105	Vine Street WTP	Zone 1 (local area)	500	1,900	n/a	n/a
Zone 1	**34th Avenue Pump Station**[1]	275	Vine Street WTP	Zone 1 (local area)	2,800	5,800	n/a	n/a
Zone 2	**North Albany Pump Station**	150	Broadway Reservoir	Zone 2 and Wildwood Reservoir	1,400	2,800	2,104	(704)
Zone 3	**Gibson Hill Road Pump Station**	150	Wildwood Reservoir	Zone 3 and Valley View Reservoirs	900	1,800	1,049	(149)

1. Queen and 34th Pump Station and Reservoirs are planned to be taken off line.

Figure 10-9: General Pump Station Layout for Zone 4 Pump Station



Reservoirs

Including the Maple Street Reservoir, Albany currently has eight reservoirs serving three pressure zones as shown in ***Figure 3-2***. The reservoirs range in size from 0.25 MG to 8 MG and are constructed of either concrete or steel. Each storage facility was evaluated for capacity and general conditions, such as seismic restraints, condition of the exterior coating system, structural integrity, underdrain systems, and overflow drain systems. This was completed through a capacity evaluation and a field inspection conducted in June 2001. References to facility condition reflect conditions as noted at the time the fieldwork was performed. This section covers results of the field study and summarizes the storage capacity analysis.

As discussed in the pump station evaluation section of this chapter, continued reliance on pumped storage (Queen and 34th Avenue reservoirs) is not included in this plan. However, the City's existing storage deficit does not allow for their immediate abandonment. Timing for abandonment is estimated in this section but the actual time of abandonment is dependent on future water demands and construction of new non-pumped storage reservoirs. Based on projected demands and timing for additional storage, interim improvements are recommended for these facilities.

Field Inspection Results

The field inspection consisted of visiting reservoir sites and making visual observations, interviewing operators, and reviewing past operational data. Recommended improvements based on this process are identified for each reservoir site. Descriptions of each facility and how it operates within the system are discussed in *Chapter 3 - Existing System Description*, and field inspection reports are included in ***Appendix E***.

Maple Street Reservoir (Project Numbers S1, S2, & S3)

Other than treatment related improvements discussed in *Chapter 8 – Vine Street Water Treatment Plant*, the only improvements recommended for the Maple Street Reservoir are for increased seismic protection and overflow piping.

- The Maple Street Reservoir is vulnerable to damage during a seismic event. There are no seismic restraints to protect the structure and no seismic valves to prevent the reservoir from draining during an earthquake. This plan recommends installing seismic restraints and valves at this facility.
- Based on observations made during the field inspection, the Maple Street Reservoir overflow piping appears to be undersized. The overflow piping should be further evaluated and sized to pass the maximum flow rate into the reservoir.

Queen Avenue Reservoir (Project Numbers S1, S2, S3 & S5)

Several improvements are recommended for the Queen Avenue Reservoir addressing deficiencies with reservoir circulation, overflow piping, and seismic protection.

- Currently, there are no seismic restraints or valves to protect this reservoir during an earthquake. This plan recommends installing seismic restraints and valves at this facility.
- The emergency overflow is undersized and consequently cannot pass the maximum flow rate into the reservoir. The reservoir is filled through a control valve based on system pressure. The valve is programmed to close when the reservoir is full or when the pump station turns on. Under its current configuration, if the shut off valve fails during filling, the tank could pressurize and cause damage to the structure. Overflow piping should be improved and sized to allow the discharge of the maximum flow rate into the reservoir. Past problems with the existing overflow configuration are discussed in *Chapter 3 – Existing System Description*.
- Reservoirs that rely on pumped storage are susceptible to poor circulation and may have difficulty maintaining minimum desired chlorine residuals. This project involves an evaluation of the Queen Avenue reservoir to determine reservoir turnover rates and possible changes in operation or improvements that will maintain desired chlorine residuals over a range of demand conditions.

34th Avenue Reservoir (Project Numbers S1, S2, S3, S4 & S5)

Recommended improvements for the 34th Avenue Reservoir are similar to those for the Queen Avenue Reservoir. There are existing deficiencies with dechlorination, reservoir circulation, overflow piping, and seismic protection.

- When drained for cleaning or repairs, the 34th Avenue Reservoir drains to a manhole that is connected to the stormwater system. The current configuration requires that City staff be on site to add a dechlorination agent. This plan recommends installing a vault over the existing manhole that will allow the automatic injection of a chlorine reducing agent (i.e. sodium bisulfite, calcium thiosulfate, ascorbic acid) to dechlorinate the drain water as required by the Oregon Department of Environmental Quality (DEQ).
- Currently, this structure has no seismic protection. This plan recommends installing seismic restraints to protect its structural integrity and seismic valves to prevent the tank from draining during an earthquake.

- The emergency overflow is undersized and consequently cannot pass the maximum flow rate into the reservoir. The reservoir is filled through a control valve based on system pressure and is programmed to close when the reservoir is full or when the pump station turns on. Under its current configuration, if the shut off valve were to fail during filling, the tank could pressurize and cause structural damage. Overflow piping should be improved and sized to allow the discharge of the maximum flow rate into the reservoir. Past problems with the existing overflow configuration are discussed in *Chapter 3 – Existing System Description.*

- Reservoirs that rely on pumped storage are susceptible to poor circulation and may have difficulty maintaining minimum desired chlorine residuals. This project involves an evaluation of the 34th Avenue reservoir to determine reservoir turnover rates and possible changes in operation or improvements that will maintain desired chlorine residuals over a range of demand conditions.

Broadway Reservoir (Project Numbers S2 & S4)

The Broadway Reservoir serves pressure Zone 1 and is filled by the Vine Street HSPS. The field inspection identified recommended improvements relating to seismic protection and dechlorination facilities.

- This plan recommends installing seismic valves to prevent reservoir drainage during an earthquake.

- Broadway Reservoir drains to the stormwater system when drained for cleaning or repairs. The current configuration requires that City staff be on site to add a dechlorination agent. This plan recommends installing a vault over the existing manhole that will allow the automatic injection of a chlorine reducing agent (ie sodium bisulfite, calcium thiosulfate, ascorbic acid) to dechlorinate the drain water as required by DEQ.

Wildwood Reservoir (Project Numbers S2 & S4)

Constructed in 1999, Wildwood Reservoir is the newest reservoir in Albany's water system. Recommendations for increased seismic protection and improved dechlorination facilities are the only recommended improvements that resulted from the field inspection.

- This plan recommends installing seismic valves to prevent reservoir drainage during an earthquake.

- When drained for cleaning or repairs, the Wildwood Reservoir drains to a stormwater manhole that outlets to an open ditch. The current configuration requires that City staff be on site to add a dechlorination agent. This plan recommends installing a vault over the existing manhole that will allow the automatic injection of a chlorine reducing agent (ie sodium bisulfite, calcium thiosulfate, ascorbic acid) to dechlorinate the drain water as required by DEQ.

Valley View Reservoirs (Project Numbers S1, S2, S3& S4)

The Valley View Reservoirs serve pressure Zone 3 and are the highest reservoirs in the City's distribution system. The field inspection resulted in recommended improvements to dechlorination facilities, seismic protection, and overflow piping.

- When drained for cleaning or repairs, the Valley View Reservoirs drain to an existing storm drain manhole that routes drain water to an open ditch. The current configuration requires that City staff be on site to add a dechlorination agent. This plan recommends installing a vault over the existing manhole through which the reservoir's primary drain pipeline is routed. This vault will allow the automatic injection of a chlorine reducing agent (ie sodium bisulfite, calcium thiosulfate, ascorbic acid) to dechlorinate the drain water as required by DEQ.
- Currently, these reservoirs do not have adequate seismic protection. This plan recommends installing seismic restraints and valves.
- The emergency overflows are incapable of passing the maximum flow rates into these reservoirs. Overflow piping should be improved and sized to allow the discharge of the maximum flow rate into the reservoirs. Also, there is not adequate capacity in the existing storm drainage system to accept water from the reservoirs. Downstream improvements may be needed to accommodate for reservoir overflow and draining.

Water Storage Capacity Evaluation

Based on the planning criteria presented in *Chapter 5 – Planning Criteria*, with two sources of supply, the required storage in each pressure zone consists of 25% of the projected maximum day demand for equalization storage, plus the fire flow storage, plus one average day demand for emergency storage. With one source of supply the required emergency storage is two times the average day demand.

Existing Storage Requirements

Table 10-4 summarizes Albany's current storage needs with respect to the criteria described above and compares them to the available capacity. Under existing demand conditions an overall deficit of about 9.71 million gallons (MG) exists within the system.

Table 10-4: Existing Storage Requirements

Pressure Zone	*ADD (mgd)*	*MDD (mgd)*	*Fire Flow (gpm - hrs)*	*Equalization Storage (MG)*	*Emergency Storage (MG)*	*Fire Flow Volume (MG)*	*Total Storage Required (MG)*	*Existing Storage[1] (MG)*	*Surplus or (Deficit) (MG)*
Zone 1	7.49	14.99	5000 - 4	3.75	14.98	1.20	19.93	11.00	(8.93)
Zone 2	0.19	0.37	1500 - 2	0.09	0.38	0.18	0.65	1.15	0.50
Zone 3 & 4	0.32	0.64	1500 - 2	0.16	0.64	0.18	0.98	0.20	(0.78)
Total	***8.00***	***16.00***	-	***4.00***	***16.00***	***1.56***	***21.56***	***12.35***	***(9.71)[2]***

1) Effective storage

2) Does not consider surplus in Zone 2

Existing storage deficits in Zone 1, 8.93 MG, and Zone 3, 0.78 MG, combine to make the total existing deficit of 9.71 MG. However, the storage deficit in pressure Zone 1 will be temporarily eliminated when the Scravel Hill WTP comes online in 2006. Albany will add a second source of supply and 2.0 MG of available storage in Zone 1 with the completion of this project. In addition, the overall system demands will decrease with the separation of Millersburg.

The storage deficit in pressure Zone 3 is created by the inability to use all the storage in the Valley View Reservoirs. These reservoirs have a combined capacity of 1.35 MG but only 0.2 MG is available as effective storage. The surrounding topography is such that if the reservoir levels drop below 560 feet some customers will experience water pressures approaching the state minimum of 20 psi. This plan recommends creating a 4th level pressure zone to eliminate this problem.[73] Once a 4th pressure zone is created the total Valley View storage capacity can be realized and customers in the upper end of Zone 3 (future Zone 4) will have water pressures in the recommended normal range.

In order to complete this separation, a 4th Level booster pump station is required at the Valley View Reservoir site and approximately 5,200 feet of pipe will need to be installed. The Zone 4 boundary and required piping are shown in ***Figure 10-10***. An elevation of 430 feet has been selected as the lower limit of pressure Zone 4 to allow the City flexibility in reservoir configurations for future replacements. The existing Valley View Reservoirs are tall, smaller diameter reservoirs; in the future, the City may want to consider an equivalent volume reservoir with a larger diameter and shorter height to reduce pressure variations within pressure Zone 3.

Figure 10-10 also shows several pipe reaches to be abandoned. These pipes run along property owners back lot lines and are not located in the public right-of-way. This situation makes them harder to maintain, as access can be cumbersome. Before these pipes are abandoned, detailed analysis should be performed to verify that future development will have access to water service. Also, as part of the zone separation, some water services will have to be relocated in order to connect to new piping.

Future Storage Requirements (Project Numbers S6, S7, S8, S9, & S10)

As Albany's population and water needs increase, more reservoir storage will be required to meet the recommended planning criteria. ***Table 10-5*** shows future storage requirements at buildout based on population and water demand forecasts presented in *Chapter 4 – Population and Water Demands*. ***Figure 10-11*** shows the estimated time of construction of new reservoirs in order to meet future requirements and the estimated times to take Queen and 34th Reservoirs off-line. In all, seven new reservoirs are planned for construction as shown in ***Table 10-6***.

[73] *North Albany Water Facility Plan*, July, 1996

In order to meet existing and future storage deficiencies, construction of new storage facilities is required in pressure Zones 1 and 3. New reservoir siting in Albany is complicated by flat terrain with little opportunity for siting of gravity feed reservoirs. Albany's *1988 Water Facility Plan* identified Knox Butte as a potential storage site. Since the development of the 1988 Facility Plan the City has purchased a Knox Butte site for a storage facility and its location is shown on ***Figure 10-12***. As shown in the reservoir storage analysis, this plan recommends construction of two 5 MG reservoirs at this location to meet future demands. At the time the first reservoir is required, a comparison should be made between the cost of constructing the second reservoir required at the Scravel Hill WTP versus piping and reservoir capacity necessary for the Knox Butte site. Both reservoirs will be required at buildout but depending on the level of development in the Knox Butte area some cost savings may result from constructing the second Scravel Hill reservoir before the first Knox Butte reservoir.

The storage analysis also identified the need for a 2 MG, Zone 1, elevated storage reservoir in 2010. This plan recommends that this storage be located in southwest Albany on Ellingson Road. This area is currently one of the most hydraulically remote areas relative to storage and subsequently experiences low residual pressures during fire flow events. The topography in southwest Albany is such that elevating storage is necessary to avoid at-grade pumped storage.

If the City elected to allow at-grade pumped storage, the second 8 MG reservoir proposed for the Broadway reservoir site could be relocated to provide a more even distribution of finished water storage. One option would be to leave the 34th Avenue reservoir and pump station in service and to locate an at-grade 8 MG reservoir and pump station at the proposed Ellingson Road site. At the time additional storage is needed, Albany will need to evaluate reservoir storage options in detail. If the City elects to use pumped storage reservoirs in the future, a dedicated on-site generator should be installed to address reliability concerns.

Table 10-5: Buildout Storage Requirements

Pressure Zone	*ADD (mgd)*	*MDD (mgd)*	*Fire Flow (gpm - hrs)*	*Equalization Storage (MG)*	*Emergency Storage (MG)*	*Fire Flow Volume (MG)*	*Total Storage Required (MG)*
Zone 1	20.81	36.97	5000 - 4	9.24	20.81	1.20	31.25
Zone 2	0.79	1.52	1500 - 2	0.38	0.79	0.18	1.35
Zone 3 & 4	0.79	1.51	1500 - 2	0.38	0.79	0.18	1.35
Total	***22.4***	***40.0***	-	***10.0***	***22.4***	***1.6***	***34.0***

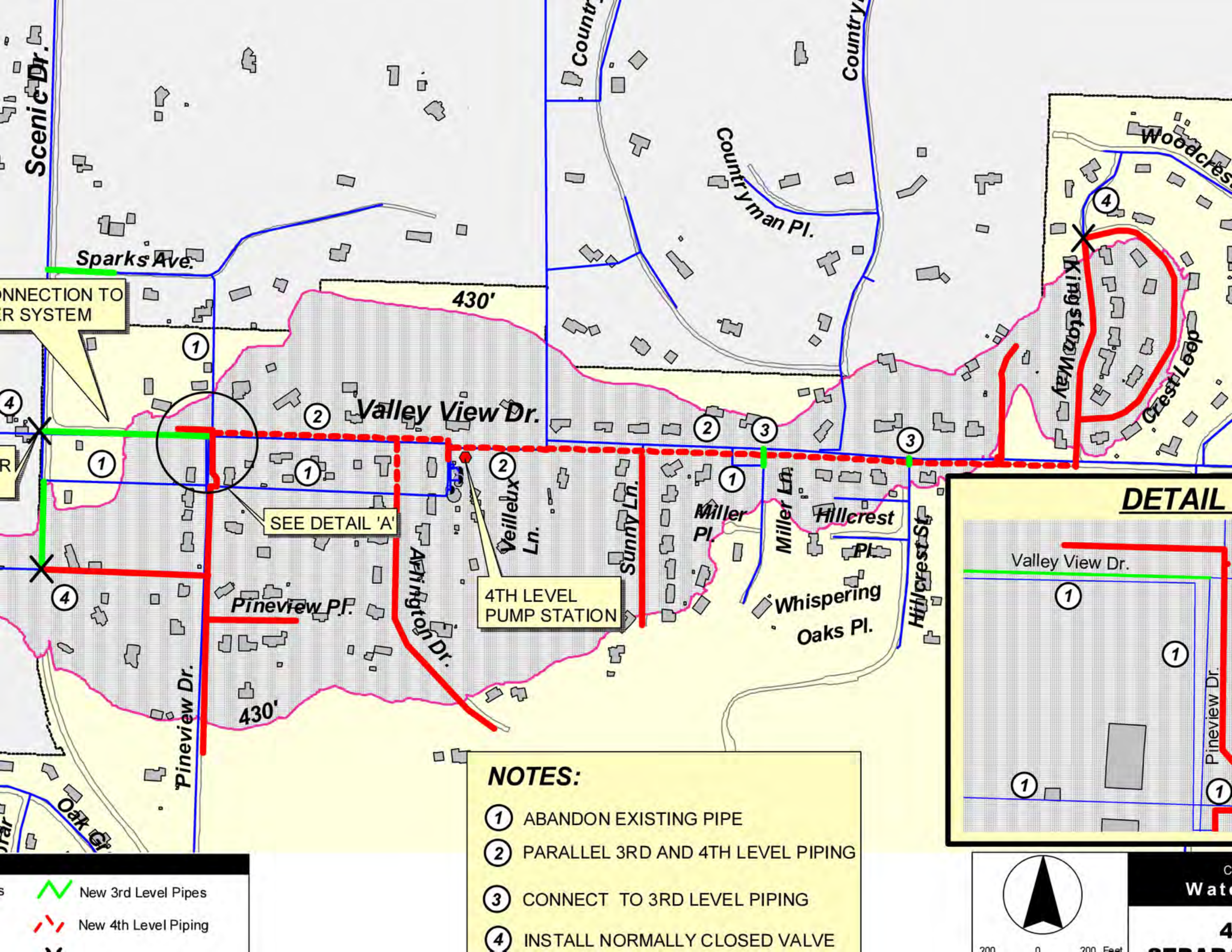
Table 10-6: Future Reservoirs

Projected Year	*Reservoir*	*Storage Site*	*Pressure Zone*	*Material*	*Effective Storage Volume (MG)*
2006	Scravel Hill #1	Scravel Hill WTP	Zone 1	Steel	2.00
2010	Ellingson Road (Elevated)	South West Albany, on Ellingson Road	Zone 1	Steel	2.00
2019	Knox Butte #1	Knox Butte	Zone 1	Concrete	5.00
2027	Knox Butte #2	Knox Butte	Zone 1	Concrete	5.00
2046	Scravel Hill #2	Scravel Hill WTP	Zone 1	Steel	2.00
2053	Broadway #2	Existing Broadway Reservoir site	Zone 1	Concrete	8.00
2060	Valley View Reservoir Replacement*	Existing Valley View Reservoir Site	Zone 3	Steel	0.85
				Total	***24.85***

* This will replace two 0.25 MG reservoirs with one 0.85 MG reservoir.

Recommended Distribution System Improvements

The recommended distribution system improvements are shown in ***Table 10-7***. Project costs shown in ***Table 10-7*** are based on unit costs developed in *Chapter 11 - Basis for Cost Estimates*. Project staging is presented in *Chapter 12 - Recommended Plan*.



Scenic Dr.
Sparks Ave.
NNECTION TO
R SYSTEM
430'
Valley View Dr.
Countryman Pl.
Woodcrest
Kingston Way
Crest Loop
SEE DETAIL 'A'
4TH LEVEL
PUMP STATION
Veilleux Ln.
Arlington Dr.
Sunny Ln.
Miller Pl.
Miller Ln.
Hillcrest Pl.
Hillcrest St.
Whispering Oaks Pl.
Pineview Pl.
Pineview Dr.
Oak Gr
DETAIL
Valley View Dr.
Pineview Dr.
NOTES:
1 ABANDON EXISTING PIPE
2 PARALLEL 3RD AND 4TH LEVEL PIPING
3 CONNECT TO 3RD LEVEL PIPING
4 INSTALL NORMALLY CLOSED VALVE
New 3rd Level Pipes
New 4th Level Piping
200 0 200 Feet
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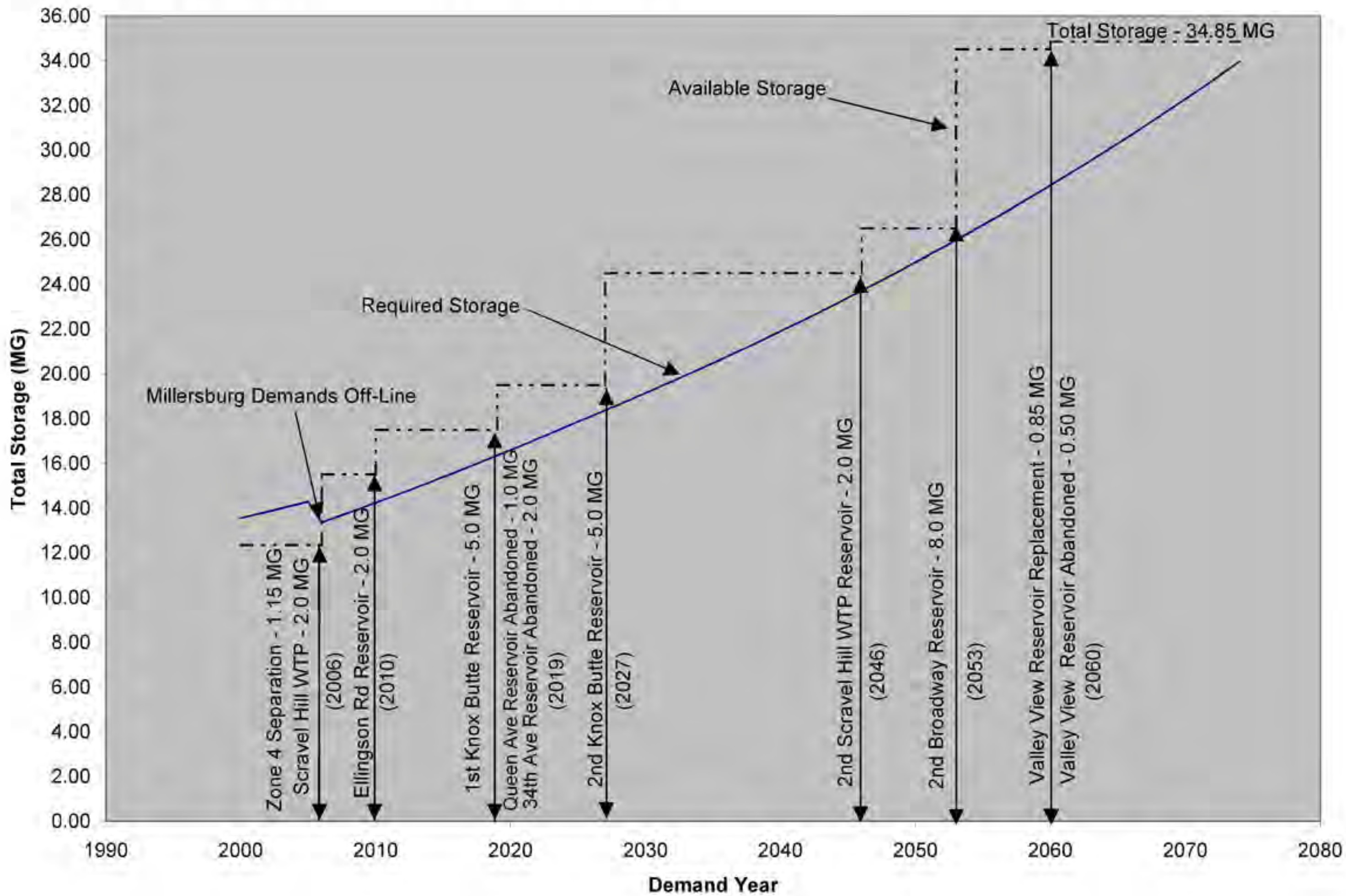
Figure 10-11: Reservoir Staging Through Buildout



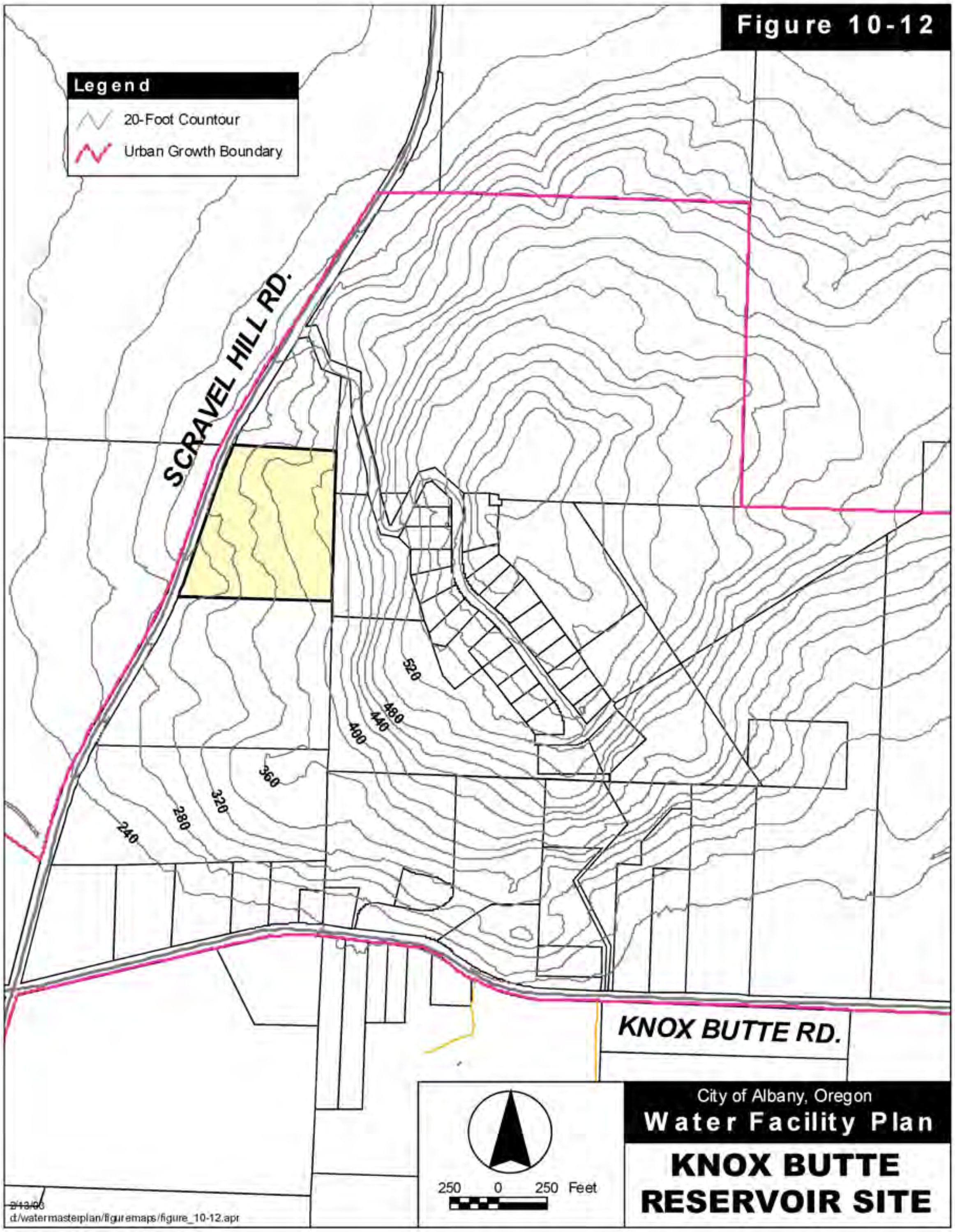


Figure 10-12
Legend
20-Foot Countour
Urban Growth Boundary
SCRAVEL HILL RD.
520
480
440
400
360
320
280
240
KNOX BUTTE RD.
City of Albany, Oregon
Water Facility Plan
KNOX BUTTE
RESERVOIR SITE
250 0 250 Feet
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7: Summary of Recommended Distribution Projects

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF
Transmission Project	P1	Pipeline along 21st Avenue from east of I-5 to Three Lakes Road, coordinate project with P6 & P7	MDD	N/A	24	1,530	$260.40
	P2	Pipeline along Spicer Road from 18th Avenue to 24-inch pipeline along Goldfish Farm Road alignment south of Hwy 20	MDD	N/A	24	2,992	$260.40
	P3	Pipeline along Knox Butte Road from Clover Ridge Road to Gold Fish Farm Road	MDD	N/A	24	970	$260.40
	P4	Pipeline along Clover Ridge Road from Santa Maria Avenue to Knox Butte Road	MDD	N/A	12	2,459	$176.40
	P5	Pipeline from the east end of Bernard Avenue to Santa Maria Avenue	MDD	N/A	20	1,807	$241.20
	P6	Pipeline from the south end of Fescue Street, connect pipeline to Project P1 along 21st Avenue	MDD + FF	N/A	12	255	$176.40
	P7	Pipeline from the south end of Rye Street, connect pipeline to Project P1 along 21st Avenue	MDD + FF	N/A	8	248	$135.60
							Project Total
pany Transmission Project	P8	Pipeline from 34th Avenue and Hwy 99E along 99E to 36th Avenue to Elk Run Drive along Elk Run Drive to Cougar Avenue	MDD	N/A	16	4,633	222.00
							Project Total
any Distribution Projects	P9	Pipeline north along Crocker Lane from Gibson Hill Road	MDD + FF	6	12	3,766	$176.40
	P10	Pipeline along Maier Lane from Skyline Drive to Chad Avenue	MDD + FF	2	8	746	$135.60
	P11	Zone 4 separation - New 3rd level piping along Sparks Avenue, Scenic Drive, and Valley View Drive	MDD + FF	N/A	8	1,367	$135.60
	P12	Zone 4 separation- New 4th level piping parallel to 3rd level pipeline along Valley View Drive	MDD + FF	N/A	8	3,790	$135.60
	P13	Pipeline along Scenic Drive from Gibson Hill Road to Wildwood Drive	PHD	10	12	3,856	$176.40
	P14	Pipeline along Quarry Road from Christmas Tree Lane to Springhill Road, along Springhill Road to Cherry Lane	MDD + FF	6-8	12	4,276	$176.40
	P15	Pipeline along Green Acres Lane from Shady Lane to Green Acres Loop	MDD + FF	6	8	522	$135.60
	P16	Pipeline along Wildwood Drive from Scenic Drive to Wildwood Reservoir site	MDD + FF	8	12	1,175	$176.40
	PS1	Zone 4 separation- Pump station with emergency backup generator (Two 7.5 HP and one 100 HP pumps)	Service Level				
	PS2	Install NA PRVs	Service Level				
	PS11	Zone 4 separation - booster pump station (One 7.5 HP pump)	-				
							Project Total

7: Summary of Recommended Distribution Projects

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF
stribution Projects	P17	Pipeline along Jackson Street north from Highway 99	MDD	18	24	203	$260.40
	P18	Pipeline along Ferry Street from Queen Avenue to 22nd Avenue	PHD	8	12	1,274	$176.40
	P19	Pipeline along Jefferson Street from 20th Avenue to 22nd Avenue to Jackson Street	MDD + FF	6	8	1,147	$135.60
	P20	Pipeline along Oak Street north from 24th Avenue	MDD + FF	6	8	675	$135.60
	P21	Pipeline along Price Road north from Highway 20	MDD + FF	8	16	217	$222.00
	P22	Pipeline along Bain Street north from Highway 20	MDD + FF	6-N/A	12	363	$176.40
	P23	Pipeline along Jackson Street north from 23rd Avenue	MDD + FF	N/A	8	289	$135.60
							Project Total
Road Reservoir Project	P24	Pipeline along Ellingson Road from Pacific Boulevard to elevated storage	MDD	N/A	16	2,091	$222.00
	S6	A 2 MG elevated reservoir	Service Level				
							Project Total
lbany Transmission Project	P25	Cross town transmission pipeline from Knox Butte Road to Main Street	MDD	8/12/20	30	14,303	$320.00
	P26	Cross town transmission pipeline from Queen along Main Street and Hill Street to 34th Avenue	MDD	N/A	24	6,686	$260.40
							Project Total
te Reservoir Project, Phase 1	P27	Pipeline along Scravel Hill Road from Gold Fish Farm Road to proposed Knox Butte Reservoir	MDD	N/A	24	9,679	$260.40
	S7	A 5 MG concrete storage reservoir	Service Level				
							Project Total
te Reservoir Project, Phase 2	S8	A 5 MG concrete storage reservoir	Service Level				
							Project Total
nent Driven Transmission/Distribution	P28	Pipeline from 34th Avenue along Hill Street alignment to Lochner Road, along Lochner Road to Ellingson Road	MDD	N/A	12	9,368	$176.40
	P29	Pipeline from 47th Avenue across railroad right-of-way then southeasterly parallel to railroad	MDD	N/A	16	1,458	$222.00
	P30	Pipeline from P29, parallel with Shortridge Street, to 40th Avenue, east to Three Lakes Road, north to Grand Prairie Road	MDD	N/A	12	7,640	$176.40
	P31	Pipeline along Grand Prairie Road from Three Lakes Road to pipeline stub out east of Waverly Drive	MDD	N/A	16	3,900	$222.00
	P32	Pipeline along Three Lakes Road from Grand Prairie Road to 21st Avenue	MDD	N/A	16	4,719	$222.00
	P33	Pipeline along Hwy. 20 from Gold Fish Farm Road to Scravel Hill Road, along Scravel Hill Road to Knox Butte Road	MDD	N/A	12	10,838	$176.40
	P34	Pipeline from Knox Butte Road south to existing 24-inch pipeline along Gold Fish Farm Road	MDD	N/A	24	3,269	$260.40
	P35	Pipeline from Santa Maria Avenue to Knox Butte Road east of Project P4	MDD	N/A	12	2,565	$176.40
	P36	Pipeline along Santa Maria Avenue from Scravel Hill Road to Clover Ridge Road	MDD	N/A	12	6,762	$176.40
	P37	Pipeline along Ellingson Road from elevated storage to Lochner Road	MDD	N/A	16	2,991	$222.00
	P38	Pipeline along Ellingson Road from Lochner to Columbus	MDD/PHD	N/A	16	4,766	$222.00

7: Summary of Recommended Distribution Projects

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF
Replacement Programs	Program-1	Steel pipeline replacement program	Service Level		8	117,994	$135.60
	Program-2	Undersized pipelines with hydrants replacement program	Fire Flow		8	25,600	$135.60
	Program-3	Perpetual life pipeline replacement program	Service Level		8 / 20	530,340	$135.60 / $241.20
							Project Total
Projects	PS3	34th Avenue backup power outlet	Service Level				
	PS4	Queen PS building security enhancements	Safety				
	PS5	Queen motorized control valve replacement	Capital Maintenance				
	PS6	Replace Queen pump No. 21 (30 HP)	Capital Maintenance				
	PS7	Queen backup power outlet	Service Level				
	S1	Maple Street, 34th, Queen, and Valley View (3), seismic restraints	Seismic				
	S2	Maple Street, Broadway, Wildwood, Valley View, 34th, and Queen seismic valves - 9 valves	Seismic				
	S3	Replace/Repair overflow piping at Maple, Queen, 34th, and Valley View (3) Reservoirs	Service Level				
	S4	Dechlorination facility for 34th, Broadway, Wildwood, and Valley View Reservoirs - 4 vaults	Regulatory				
	S5	Increase reservoir circulation for 34th and Queen reservoirs	Regulatory				
	PS12	Increase level 2 pump station capacity	Service Level				
	PS13	Increase level 3 pump station capacity	Service Level				
							Project Total
Reservoir Project	S9	An 8 MG concrete reservoir	Service Level				
							Project Total
ew Reservoir Project	S10	Replace two 0.25 MG Valley View reservoirs with a single 0.85 MG steel reservoir	Service Level				
							Project Total
					Total Cost for Distribution System Improvements =		

d project costs are based on March 2002 Seattle ENR CCI= 7560
based on individual projects being rounded to the nearest $1 K,
projects to the nearest $1 K, and total distribution system
ents to the nearest $100 K.
own for development driven projects reflect anticipated costs for
g.

Abbreviations:
MDD = MAXIMUM DAY DEMAND
PHD = PEAK HOUR DEMAND
MDD + FF = MAX DAY DEMAND PLUS FIRE FLOW

CHAPTER 11 - BASIS OF COST ESTIMATES

INTRODUCTION

This chapter reviews the basis of cost estimates used in development of project costs for recommended water facility plan improvements.

Estimated project costs for water line, pump station and reservoir improvements were developed on the basis of "unit costs". Individual project cost estimates were developed for improvements to the Canal and treatment plants. Both unit cost estimates and individual project cost estimates are planning level or "order of magnitude" estimates. Planning level estimates are based on a combination of published cost data, MWH's experience with projects of similar scope and a review of Albany's recent bid tab data for water system improvements.

Planning level estimates for facility plans are not designed to predict the low bid of any particular construction project. Rather, they are geared toward the average of all the bids that may be received on any specific project. Order of magnitude cost estimates are used to forecast project costs in facility plans and typically range from 30 percent below to 50 percent above actual project costs[74]. Actual project costs depend on the final project scope, labor and material costs, market conditions, construction schedule, and other variables at the time a project is built.

Unit cost estimates can be developed with the greatest confidence when they are based on actual, local construction bid data. In the case of water lines, communities typically have a local bid history that can be relied upon as a gauge of unit water line costs for smaller diameter pipelines. Regional bid data is often required for larger diameter pipelines, reservoirs and pump stations.

Since construction costs change over time, cost estimates presented in this plan are indexed for ease in updating the costs to reflect future market conditions. The Engineering News-Record (ENR) Construction Cost Index (CCI) is commonly used for this purpose. Costs presented in this plan are based on a Seattle ENR CCI of 7560 for March 2002. The Seattle area construction market was used because it is the nearest market index available.

Total capital costs for each project are comprised of three components; direct construction costs, an allowance for contingencies and an allowance for engineering, legal and administrative costs. The allowance for contingencies covers items such as refinements in project scope during design and unforeseen site conditions encountered during construction. The contingency allowance does not include major project scope additions or additional costs resulting from permit mitigation requirements such as wetlands enhancement. The engineering, legal and administrative allowance provides for design, construction management, legal and administrative costs.

The general basis of the cost estimates for new construction for each type of facility is given below. Any modifications to this general basis, where appropriate, are discussed in projects sheets in *Chapter 12* - *Recommended Plan*.

[74] Douglas D. Gransberg, Editor, *Construction Cost Estimating*, AACE International, 2000

PIPELINES

Unit costs for smaller diameter (less than and equal to 24-inch diameter) pipelines are based on a review of Albany's water line construction bid tab data between 1999 and 2001. The majority of these projects involved water line replacements. Replacement projects often require maintaining service to existing customers, matching fittings and valves to existing pipelines with dissimilar materials, resolving utility conflicts and require more extensive pavement restoration than new construction. Approximately 80 percent of pipeline construction projects recommended in this facility plan involve construction of replacement pipelines with 8-inch diameter water lines. Consequently, Albany's average unit costs for replacement projects have been used as the estimate of water line costs for pipelines less than and equal to 24-inch.

The City of Albany does not have recent bid tab data for larger diameter pipelines. Therefore, unit costs for 30-inch and larger diameter pipelines are based on MWH's regional experience adjusted to a Seattle ENR CCI of 7560.

Average unit construction costs were increased by 20 percent for contingencies and an additional 15 percent for engineering, legal and administrative costs. The unit costs per foot of installed pipe used to estimate water line construction costs are shown below in ***Table 11-1***.

Table 11-1: Pipeline Unit Costs

Diameter (inches)	*Constructed Cost $/ft*	*Contingency (20%) $/ft*	*Engineering, Legal & Administrative (15%) $/ft*	*TOTAL $/ft*
8"	$100.44	$20.10	$15.10	$135.60
12"	$130.67	$26.10	$19.60	$176.40
16"	$164.44	$32.90	$24.70	$222.00
20"	$178.67	$35.70	$26.80	$241.20
24"	$192.89	$38.60	$28.90	$260.40
30"	$237.04	$47.40	$35.60	$320.00
36"	$253.33	$50.70	$38.00	$342.00

Unless otherwise noted, these unit costs are based on the following set of assumptions:

- Streets have an average asphalt paving depth of 4 inches;
- Asphalt trench patching is assumed to be required for the full project length as opposed to a full street overlay;
- There are no significant utility relocations required for pipe installation;
- Trenching is in soil, with no rock encountered;
- Trench width is equal to the nominal pipe diameter plus 2 feet and trench depth assumes cover to top of pipe equal to 3 ½ feet;
- No trench dewatering is required;
- Unless specifically noted, joints are unrestrained;
- Pipe material is ductile iron, Class 52, cement lined and asphalt coated;

- Hydrant spacing is 400 feet;
- Valve spacing varies by pipe size as follows, two valves per:
  - 250 feet for 8-inch and 12-inch pipe,
  - 350 feet for 20-inch pipe, and
  - 500 feet for 24-inch and larger.
- Valves are gate valves for 8-inch pipelines and butterfly valves for 12-inch and larger;
- Projects are in the range of 100 feet to 5,000 feet in length, and
- There are no costs for property or easement acquisition.

When pipelines are designed, consideration should be given to the use of restrained joints on a case-by-case basis. Restrained joints should be used when pipelines cross unstable land, railroad tracks, freeways, or other locations that could result in unusual ground movements. Use of restrained joints should also be considered if failure of a water line could result in significant property damage or present life, health and safety risks. Use of restrained joints could add 10 percent to the construction costs. If pipeline excavation encounters rock rather than soil, project costs could increase a minimum of 10-20 percent.

STORAGE RESERVOIRS

Costs for storage reservoirs assume construction without any special site constraints or other requirements unless specifically noted. Unit costs for reservoirs are based on MWH's regional experience between 1990 and 2000, and were adjusted to a Seattle ENR CCI of 7560.

These unit cost estimates are based, unless otherwise noted, on the following assumptions:

- Concrete reservoirs are constructed of poured-in-place concrete;
- Steel reservoirs are installed at existing grade;
- Steel reservoirs are assumed not to exceed 2.0 MG;
- No rock is encountered for reservoir foundation excavation;
- Landscaping around the reservoir is limited to grass;
- Seismic reinforcement is to Zone 3 (state Structural Specialty Code, Albany area);
- Piping to bring water to and from the reservoir is located at the site;
- There are no costs for land acquisition or site demolition;
- There are no site or permit constraints that limit use of the most economical height to diameter ratio for the desired reservoir volume, and
- There are no special site, environmental or community mitigation costs associated with the reservoir construction.

Unit costs for concrete and at-grade (not elevated) steel reservoirs are shown below in ***Tables 11-2*** and ***11-3*** respectively.

Table 11-2: Concrete Reservoir Unit Costs

Size (Million Gallon)	*Construction ($/gallon)*	*Contingency ($/gallon)*	*Engineering, Legal & Administrative (15%) ($/gallon)*	*Total Cost ($/gallon)*
1.0	$0.92	$0.18	$0.14	$1.24
1.5	$0.82	$0.16	$0.12	$1.10
2.0	$0.72	$0.14	$0.11	$0.97
3.0	$0.62	$0.12	$0.09	$0.83
3.5	$0.60	$0.12	$0.09	$0.81
4.0	$0.55	$0.11	$0.08	$0.74
5.0	$0.52	$0.10	$0.08	$0.70

Table 11-3: At Grade Steel Reservoir Unit Costs

Size (Million Gallon)	*Construction ($/gallon)*	*Contingency ($/gallon)*	*Engineering, Legal & Administrative (15%) ($/gallon)*	*Total Cost ($/gallon)*
0.25	$0.70	$0.14	$0.11	$0.95
0.50	$0.57	$0.11	$0.09	$0.77
0.75	$0.48	$0.10	$0.07	$0.65
1.00	$0.45	$0.09	$0.07	$0.61
1.50	$0.39	$0.08	$0.06	$0.53
2.00	$0.35	$0.07	$0.05	$0.47
3.00	$0.30	$0.06	$0.05	$0.41

One elevated reservoir is proposed in the recommended improvements discussed in *Chapter 10 - Distribution System Evaluation*. A unit cost of $2/gallon was used for this reservoir. The unit cost was based upon limited project cost data for similar reservoirs in Oregon (such as the City of West Linn's 400,000 gallon elevated tank constructed in the 1990's, and from a vendor cost estimate[75]).

Seismic requirements for facilities in the Pacific Northwest have changed substantially over the last several years due to increased understanding of seismic risk in the region. Rather than utilize the seismic zones established as part of the state Structural Specialty Code, many communities have adopted the seismic standards of the International Building Code[76] (IBC). New facilities that are considered "lifelines", such as reservoirs, are required to have a site-specific seismic analysis under this Code. If the City of Albany were to use this approach to reservoir design, the site-specific seismic assessments may result in either higher, or lower, project costs, depending on the magnitude of the site-specific seismic concerns compared to the Zone 3 requirements.

Special screening or landscape requirements that are specific to a site could add up to 30 percent to the costs of a reservoir.

[75] 2002, Planning level estimate of cost for 2.0 MG elevated steel tank from Caldwell Tank, Inc.
[76] International Building Code, Section 1617—*Earthquake Loads-Minimum Design Lateral Force and Related Effects*

PUMP STATIONS

Unit costs for pump stations ranging in size from 50 to 1,000 pump motor horsepower are shown in ***Table 11-4***. These unit costs are based on the horsepower requirements for pump stations with the largest motor out of service (firm capacity). Costs for pump stations assume construction without any special site constraints or other requirements unless otherwise noted. Assumptions used to develop these unit cost estimates include:

- No rock is encountered during excavation;
- Landscaping around the site is limited to grass;
- Seismic reinforcement is to Zone 3 (state Structural Building Code, Albany area);
- There are no costs for land acquisition or site demolition;
- Piping is limited to supply and discharge pipes within the pump station yard;
- There are no special site, environmental or community mitigation costs associated with the pump station construction;
- Buildings are of concrete masonry construction, and
- Standby generator costs are not included, but project costs do include the cost of an emergency connection to a portable standby generator

As noted in *Chapter 10 – Distribution System Evaluation*, pump station improvements have been sized assuming firm capacity. Similar to reservoirs, unit costs for pump stations are based on MWH's regional experience between 1990 and 2000 adjusted to a Seattle ENR CCI of 7560. Pump station upgrades that involve replacement or addition of pumps or installation of larger impellers have been estimated individually.

Table 11-4: Pump Station Unit Costs

Size Installed Firm HP	Construction ($)	Contingency (20%) ($)	Engineering, Legal & Administrative (15%) ($)	Total Cost ($)
50	$138,479	$27,696	$20,772	$186,900
75	$207,718	$41,544	$31,158	$280,400
100	$276,958	$55,392	$41,544	$373,900
200	$553,916	$110,783	$83,087	$747,800
300	$830,873	$166,175	$124,631	$1,121,700
500	$1,384,789	$276,958	$207,718	$1,869,500
1000	$2,769,578	$553,916	$415,437	$3,738,900

CANAL

Costs of each of the recommended improvement projects for the Canal shown in *Chapter 7 – Canal Evaluation* and *Chapter 12 - Recommended Plan*, were developed as independent estimates. These estimates include a 20 percent allowance for contingencies and a 15 percent allowance for engineering, legal and administrative costs. Costs for some projects were prepared as an allocation of reasonable anticipated costs, as opposed to estimates, because the scope of the effort is unknown at this time.

VINE STREET WATER TREATMENT PLANT

Costs for recommended improvements for the Vine Street WTP are heavily influenced by equipment costs. Equipment costs were developed from MWH's project experience in the Willamette Valley and quotes from equipment manufacturers and product line representatives in the Pacific Northwest[77]. The recommended improvements for the Vine Street WTP require the replacement and/or upgrade of equipment, piping and unit process facilities. Costs of each of the improvement projects are shown in *Chapter 8 – Vine Street Water Treatment Plant* and *Chapter 12 - Recommended Plan*, and were developed as independent project costs. Cost estimates for the Vine Street WTP projects are planning level estimates and include a 20 percent contingency allowance and a 20 percent allowance for engineering, legal and administrative expenses. A greater allowance for engineering, legal and administrative fees has been assumed for the Vine Street WTP due to the inherent complexities associated with plant design and retrofitting and the historic designation of the structure.

JOINT WATER PROJECT

As discussed in *Chapter 9 - Joint Water Project*, predesign estimates for this project have been prepared under a separate contract between the City of Albany and the firm of CH2M-Hill. Predesign estimates for the Joint Water Project are included with *Chapter 9 - Joint Water Project* and *Chapter 12 - Recommended Plan*. The basis for predesign estimates is also discussed in detailed technical memorandums prepared by CH2M-Hill. As noted earlier, these memorandums are included as ***Appendix G***.

[77] Mike Neal, C.W. Neal Company, Maple Street Reservoir Baffle Replacement (Interior Inspection 4/24/02)

CHAPTER 12 – RECOMMENDED PLAN

INTRODUCTION

This chapter summarizes recommended improvements identified in Chapters 7 through 10. The Santiam-Albany Canal, the Vine Street WTP, the Joint Water Project, and the distribution system (pipes, pump stations, and reservoirs) were evaluated individually and in aggregate. Each facility was evaluated for existing system deficiencies, the remaining life of key components, regulatory requirements, and the improvements required over time to meet increasing water demands. Improvements are based on fixing existing system deficiencies, meeting future needs and recognizing inter-relationships. Therefore, projects required to meet existing system demands have been sized to meet future needs as well. Based on the evaluation of these facilities, projects are scheduled in four planning stages as defined in *Chapter 5 - Planning Criteria*:

- Stage 1 from Year 2005 to 2009,
- Stage 2 from Year 2010 to 2014,
- Stage 3 from Year 2015 to 2024 and
- Stage 4 from Year 2025 to buildout (2074).

These improvements are recommended as a long-term guide for the development of the City's water system. While projects are listed as being scheduled for construction in a specific stage, staging is intended only to provide a general guideline of priorities, relationships between projects, ties to levels of growth, and understanding of maintenance priorities. Actual timing for construction of each project should be based on a regular review of system needs and actual water demands. For example, it is impossible to predict when development driven projects shown in Stage 4 will actually be needed. Timing for those projects is entirely development dependent and it is important that the plan have flexibility to respond to development as it occurs.

SUMMARY OF IMPROVEMENT PROJECTS

Tables 12-1 and 12-2 summarize and prioritize into stages recommended improvements necessary to fix existing system deficiencies, replace key components as they exceed their service life, meet regulatory requirements, and meet future water demands. Improvements required to meet future water demands, or capacity increasing improvements, are eligible to be funded through the improvement portion of System Development Charges (SDC_is).

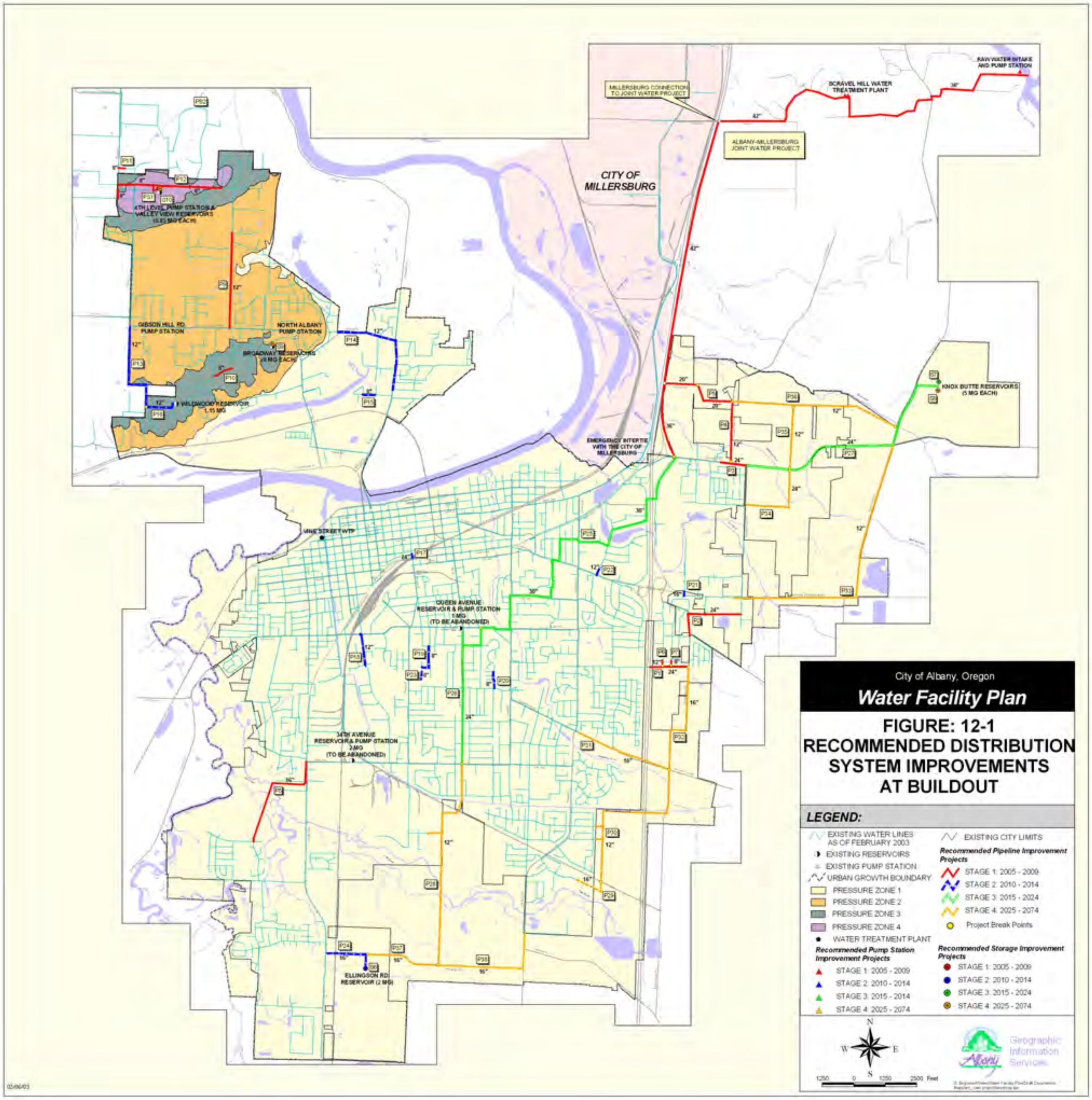
Projects that are sized to correct existing deficiencies and to meet future water demands are "shared projects" and are eligible to be partially funded through SDC_is. ***Table 12-2*** distinguishes between improvement costs that are SDC_i eligible and those that are not. The methodology used to allocate project costs for shared projects is included in ***Appendix A*** in the minutes for the November 5, 2002 Water Task Force meeting.

Figure 12-1 shows distribution improvements by location and stage. Project worksheets have been included at the end of this chapter to summarize, in detail, recommended improvement projects.

Table 12-1: Recommended Improvements by Category

Project Category	Stage 1	Stage 2	Stage 3	Stage 4	Total[1]
Canal	$3,460,000	$2,830,000	$2,110,000	$0	***$8,400,000***
Vine Street WTP	$2,535,000	$3,077,000	$1,997,000	$0	***$7,600,000***
Distribution System	$14,350,000	$15,479,000	$25,793,000	$79,397,000	***$135,000,000***
JWP	$32,300,000	$0	$0	$3,900,000	***$36,200,000***
TOTAL[1]	***$52,600,000***	***$21,400,000***	***$29,900,000***	***$83,300,000***	***$187,200,000***

1) Rounded to the nearest $100 K.


City of Albany, Oregon
Water Facility Plan
FIGURE: 12-1
RECOMMENDED DISTRIBUTION SYSTEM IMPROVEMENTS AT BUILDOUT
LEGEND:
EXISTING WATER LINES AS OF FEBRUARY 2003
EXISTING RESERVOIRS
EXISTING PUMP STATION
URBAN GROWTH BOUNDARY
PRESSURE ZONE 1
PRESSURE ZONE 2
PRESSURE ZONE 3
PRESSURE ZONE 4
WATER TREATMENT PLANT
Recommended Pump Station Improvement Projects
STAGE 1: 2005 - 2009
STAGE 2: 2010 - 2014
STAGE 3: 2015 - 2014
STAGE 4: 2025 - 2074
EXISTING CITY LIMITS
Recommended Pipeline Improvement Projects
STAGE 1: 2005 - 2009
STAGE 2: 2010 - 2014
STAGE 3: 2015 - 2024
STAGE 4: 2025 - 2074
Project Break Points
Recommended Storage Improvement Projects
STAGE 1: 2005 - 2009
STAGE 2: 2010 - 2014
STAGE 3: 2015 - 2024
STAGE 4: 2025 - 2074
Geographic Information Services
CITY OF MILLERSBURG
MILLERSBURG CONNECTION TO JOINT WATER PROJECT
ALBANY-MILLERSBURG JOINT WATER PROJECT
GIBSON HILL PUMP STATION
NORTH ALBANY PUMP STATION
BROADWAY RESERVOIRS (5 MG EACH)
KNOX BUTTE RESERVOIRS (5 MG EACH)
VINE STREET WTP
QUEEN AVENUE RESERVOIR & PUMP STATION 1 MG (TO BE ABANDONED)
34TH AVENUE RESERVOIR & PUMP STATION 2 MG (TO BE ABANDONED)
ELLINGSON RD RESERVOIR (2 MG)
EMERGENCY INTERTIE WITH THE CITY OF MILLERSBURG

COMMENDED PLAN, 2005 - 2074

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF	PROJECT COST[1,2]	COST NON-
nt Water Project, Phase 1	JWP1	Intake, raw water transmission line, WTP, reservoir, finished water transmission line	Service Level					$32,300,000	$16,500,0
							Project Total	$32,300,000	$16,500,0
st End Transmission Project	P1	Pipeline along 21st Avenue from east of I-5 to Three Lakes Road, coordinate project with P6 & P7	MDD	N/A	24	1,530	$260.40	$398,000	$149,00
	P2	Pipeline along Spicer Road from 18th Avenue to 24-inch pipeline along Goldfish Farm Road alignment south of Hwy 20	MDD	N/A	24	2,992	$260.40	$779,000	$292,00
	P3	Pipeline along Knox Butte Road from Clover Ridge Road to Gold Fish Farm Road	MDD	N/A	24	970	$260.40	$253,000	$95,000
	P4	Pipeline along Clover Ridge Road from Santa Maria Avenue to Knox Butte Road	MDD	N/A	12	2,459	$176.40	$434,000	$163,00
	P5	Pipeline from the east end of Bernard Avenue to Santa Maria Avenue	MDD	N/A	20	1,807	$241.20	$436,000	$163,00
	P6	Pipeline from the south end of Fescue Street, connect pipeline to Project P1 along 21st Avenue	MDD + FF	N/A	12	255	$176.40	$45,000	$45,000
	P7	Pipeline from the south end of Rye Street, connect pipeline to Project P1 along 21st Avenue	MDD + FF	N/A	8	248	$135.60	$34,000	$34,000
							Project Total	$2,379,000	$941,00
uth Albany Transmission Projects	P8	Pipeline from 34th Avenue and Hwy 99E along 99E to 36th Avenue to Elk Run Drive along Elk Run Drive to Cougar Avenue	MDD	N/A	16	4,633	$222.00	$1,029,000	$1,029,00
							Project Total	$1,029,000	$1,029,00
rth Albany Distribution Projects, Phase 1	P9	Pipeline north along Crocker Lane from Gibson Hill Road	MDD + FF	6	12	3,766	$176.40	$664,000	$664,00
	P10	Pipeline along Maier Lane from Skyline Drive to Chad Avenue	MDD + FF	2	8	746	$135.60	$101,000	$101,00
	P11	Zone 4 separation - New 3rd level piping along Sparks Avenue, Scenic Drive, and Valley View Drive	MDD + FF	N/A	8	1,367	$135.60	$185,000	$185,00
	P12	Zone 4 separation- New 4th level piping parallel to 3rd level pipeline along Valley View Drive	MDD + FF	N/A	8	3,790	$135.60	$514,000	$514,00
	PS1	Zone 4 separation- Pump station with emergency backup generator (Two 7.5 HP and one 100 HP pumps)	Service Level					$186,000	$186,00
	PS2	Install NA PRVs	Service Level					$15,000	$15,000
							Project Total	$1,665,000	$1,665,0
servoir Projects, Phase 1	PS3	34th Avenue backup power outlet	Service Level					$30,000	$30,000
	PS4	Queen PS building security enhancements	Safety					$15,000	$15,000
	PS5	Queen motorized control valve replacement	Capital Maintenance					$25,000	$25,000
	PS6	Replace Queen pump No. 21 (30 HP)	Capital Maintenance					$35,000	$35,000
	PS7	Queen backup power outlet	Service Level					$30,000	$30,000
	S1	Maple Street, 34th, Queen, and Valley View (3), seismic restraints	Seismic					$500,000	$500,00
	S2	Maple Street, Broadway, Wildwood, Valley View, 34th, and Queen seismic valves - 9 valves	Seismic					$240,000	$240,00
	S3	Replace/Repair overflow piping at Maple, Queen, 34th, and Valley View (3) Reservoirs	Service Level					$234,000	$234,00
	S4	Dechlorination facilities for 34th, Broadway, Wildwood, and Valley View Reservoirs - 4 vaults	Regulatory					$90,000	$90,000
	S5	Increase reservoir circulation for 34th and Queen reservoirs	Regulatory					$45,000	$45,000
							Project Total	$1,244,000	$1,244,0

COMMENDED PLAN, 2005 - 2074

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF	PROJECT COST[1,2]	COST NON
nal Project, Phase 1	C1	Update control structures	Service Level					$760,000	$760,00
	C2	Ensure Canal capacity	Service Level					$1,450,000	$1,450,0
	C3	Channel restoration	Service Level					$1,000,000	$1,000,0
	C4	Improve Canal access	Service Level					$250,000	$250,00
							Project Total	*$3,460,000*	*$3,460,0*
e Street WTP Projects, Phase 1	Planning-1	System wide security assessment	Safety					$150,000	$150,00
	PS8	Replace HSPS pump No. 14 (200 HP)	Capital Maintenance					$75,000	$75,000
	PS9	HSPS backup power outlet	Service Level					$30,000	$30,000
	PS10	Analysis of operating conditions including VFDs at the HSPS	Service Level					$55,000	$55,000
	WTP01	Water quality monitoring upgrades	Regulatory					$84,000	$84,000
	WTP02	Backwash/Surface wash piping system improvements	Capital Maintenance					$329,000	$329,00
	WTP03	Replace accelator #2 settling tubes	Capital Maintenance					$213,000	$213,00
	WTP04	Plant pipeline inspection and cleaning	Capital Maintenance					$112,000	$112,00
	WTP05	Repair Maple Street Reservoir baffle and improve disinfection performance	Regulatory					$115,000	$115,00
	WTP06	Chlorine system safety improvements	Safety					$140,000	$140,00
	WTP07	Replace/Repair control room building HVAC system	Capital Maintenance					$70,000	$70,000
	WTP08	VFD harmonics evaluation	Capital Maintenance					$20,000	$20,000
	WTP09	ADA/OSHA compliance upgrade	Safety					$50,000	$50,000
	WTP10	WTP automation upgrade--plant work	Service Level					$535,000	$535,00
	WTP11	WTP automation upgrade--distribution work	Service Level					$127,000	$127,00
	WTP12	WTP security upgrade	Safety					$150,000	$150,00
	WTP13	WTP filter gallery maintenance	Capital Maintenance					$280,000	$280,00
							Project Total	*$2,535,000*	*$2,535,0*
eline Replacement Programs, Phase 1	Program-1	Steel pipeline replacement program	Service Level		8	44,248	$135.60	$6,000,000	$6,000,0
	Program-2	Undersized pipelines with hydrants replacement program	Fire Flow		8	1,845	$135.60	$250,000	$250,00
	Program-3	Perpetual life pipeline replacement program	Service Level		8	13,150	$135.60	$1,783,000	$1,783,0
							Project Total	*$8,033,000*	*$8,033,0*
		STAGE 1 TOTAL						**$52,600,000**	**$35,400,**

...COMMENDED PLAN, 2005 - 2074

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF	PROJECT COST[1,2]	COST ... NON...
...rth Albany Distribution Projects, Phase 2	P13	Pipeline along Scenic Drive from Gibson Hill Road to Wildwood Drive	PHD	10	12	3,856	$176.40	$680,000	$680,00...
	P14	Pipeline along Quarry Road from Christmas Tree Lane to Springhill Road, along Springhill Road to Cherry Lane	MDD + FF	6-8	12	4,276	$176.40	$754,000	$382,00...
	P15	Pipeline along Green Acres Lane from Shady Lane to Green Acres Loop	MDD + FF	6	8	522	$135.60	$71,000	$71,000
	P16	Pipeline along Wildwood Drive from Scenic Drive to Wildwood Reservoir site	MDD + FF	8	12	1,175	$176.40	$207,000	$207,00...
	PS11	Zone 4 separation - booster pump station (One 7.5 HP pump)	-					$20,000	$0
							Project Total	$1,732,000	$1,340,0...
...ne 1 Distribution Projects	P17	Pipeline along Jackson Street north from Highway 99	MDD	18	24	203	$260.40	$53,000	$0
	P18	Pipeline along Ferry Street from Queen Avenue to 22nd Avenue	PHD	8	12	1,274	$176.40	$225,000	$0
	P19	Pipeline along Jefferson Street from 20th Avenue to 22nd Avenue to Jackson Street	MDD + FF	6	8	1,147	$135.60	$156,000	$156,00...
	P20	Pipeline along Oak Street north from 24th Avenue	MDD + FF	6	8	675	$135.60	$92,000	$92,000
	P21	Pipeline along Price Road north from Highway 20	MDD + FF	8	16	217	$222.00	$48,000	$48,000
	P22	Pipeline along Bain Street north from Highway 20	MDD + FF	6-N/A	12	363	$176.40	$64,000	$64,000
	P23	Pipeline along Jackson Street north from 23rd Avenue	MDD + FF	N/A	8	289	$135.60	$39,000	$39,000
							Project Total	$677,000	$399,00...
...ingson Road Reservoir Project	P24	Pipeline along Ellingson Road from Pacific Boulevard to elevated storage	MDD	N/A	16	2,091	$222.00	$464,000	$0
	S6	A 2 MG elevated reservoir	Service Level					$4,000,000	$0
							Project Total	$4,464,000	$0
...nal Projects, Phase 2	C1	Update control structures	Service Level					$1,130,000	$1,130,0...
	C2	Ensure Canal capacity	Service Level					$1,450,000	$1,450,00...
	C4	Improve Canal access	Service Level					$250,000	$250,00...
							Project Total	$2,830,000	$2,830,0...
...e Street WTP Projects, Phase 2	Planning-2	Facility Plan update	Service Level					$300,000	$135,00...
	WTP13	WTP filter gallery maintenance	Capital Maintenance					$280,000	$280,00...
	WTP14	Clearwell repairs	Capital Maintenance					$70,000	$70,000
	WTP15	Chemical storage improvements	Safety					$28,000	$28,000
	WTP16	Solids handling	Capital Maintenance					$220,000	$220,00...
	WTP17	Seismic upgrades	Seismic					$570,000	$570,00...
	WTP18	Distribution system pressure monitoring improvements	Service Level					$70,000	$70,000
	WTP19	Replace accelator #1 settling tubes	Capital Maintenance					$210,000	$210,00...
	WTP20	Repair/Replace filter media/underdrain system	Capital Maintenance					$682,000	$682,00...
	WTP21	Add granular activated carbon (GAC) to filter media	Service Level					$150,000	$150,00...
	WTP22	Valve maintenance	Capital Maintenance					$497,000	$497,00...
							Project Total	$3,077,000	$2,912,0...
...eline Replacement Programs, Phase 2	Program-1	Steel pipeline replacement program	Service Level		8	44,248	$135.60	$6,000,000	$6,000,0...
	Program-2	Undersized pipelines with hydrants replacement program	Fire Flow		8	7,375	$135.60	$1,000,000	$1,000,0...
	Program-3	Perpetual life pipeline replacement program	Service Level		8	11,844	$135.60	$1,606,000	$1,606,0...
							Project Total	$8,606,000	$8,606,0...
		STAGE 2 TOTAL						$21,400,000	$16,100,...

COMMENDED PLAN, 2005 - 2074

PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF	PROJECT COST[1,2]	COST NON-
ntral Albany Transmission Project	P25	Cross town transmission pipeline from Knox Butte Road to Main Street	MDD	8/12/20	30	14,303	$320.00	$4,577,000	$1,017,00
	P26	Cross town transmission pipeline from Queen along Main Street and Hill Street to 34th Avenue	MDD	N/A	24	6,686	$260.40	$1,741,000	$605,00
							Project Total	***$6,318,000***	***$1,622,0***
servoir Projects, Phase 2	PS12	Increase level 2 pump station capacity	Service Level					$10,000	$0
	PS13	Increase level 3 pump station capacity	Service Level					$10,000	$0
							Project Total	***$20,000***	***$0***
ox Butte Reservoir Project, Phase 1	P27	Pipeline along Scravel Hill Road from Gold Fish Farm Road to proposed Knox Butte Reservoir	MDD	N/A	24	9,679	$260.40	$2,520,000	$0
	S7	A 5 MG concrete storage reservoir	Service Level					$3,500,000	$0
							Project Total	***$6,020,000***	***$0***
nal Projects, Phase 3	C1	Update control structures	Service Level					$2,110,000	$2,110,0
							Project Total	***$2,110,000***	***$2,110,0***
e Street WTP Projects, Phase 3	Planning-2	Facility Plan update	Service Level					$300,000	$158,00
	WTP22	Valve maintenance	Capital Maintenance					$497,000	$497,00
	WTP23	Plant hydraulics	Service Level					$280,000	$0
	WTP24	Instrumentation and control improvements	Capital Maintenance					$840,000	$840,00
	WTP25	Replace Maple Street Reservoir baffle	Capital Maintenance					$80,000	$80,000
							Project Total	***$1,997,000***	***$1,575,0***
eline Replacement Programs, Phase 3	Program-1	Steel pipeline replacement program	Service Level		8	29,499	$135.60	$4,000,000	$4,000,0
	Program-2	Undersized pipelines with hydrants replacement program	Service Level		8	16,380	$135.60	$2,221,000	$2,221,0
	Program-3	Perpetual life pipeline replacement program	Service Level		8	53,200	$135.60	$7,214,000	$7,214,0
							Project Total	***$13,435,000***	***$13,435,0***
		STAGE 3 TOTAL						**$29,900,000**	**$18,700,0**

'COMMENDED PLAN, 2005 - 2074

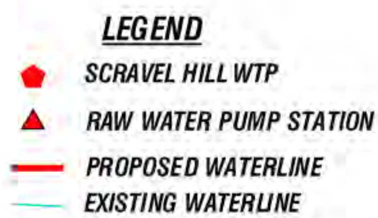
PROJECT NAME	PROJECT ID	PROJECT DESCRIPTION	PROJECT DRIVER	EXISTING DIAMETER (inches)	PROPOSED DIAMETER (inches)	TOTAL LENGTH (LF)	UNIT COST $/LF	PROJECT COST[1,2]	COST / NON-
velopment Driven Transmission/Distribution jects[3]	P28	Pipeline from 34th Avenue along Hill Street alignment to Lochner Road, along Lochner Road to Ellingson Road	MDD	N/A	12	9,368	$176.40	$382,000	$0
	P29	Pipeline from 47th Avenue across railroad right-of-way then southeasterly parallel to railroad	MDD	N/A	16	1,458	$222.00	$126,000	$0
	P30	Pipeline from P29, parallel with Shortridge Street, to 40th Avenue, east to Three Lakes Road, north to Grand Prairie Road	MDD	N/A	12	7,640	$176.40	$312,000	$0
	P31	Pipeline along Grand Prairie Road from Three Lakes Road to pipeline stub out east of Waverly Drive	MDD	N/A	16	3,900	$222.00	$337,000	$0
	P32	Pipeline along Three Lakes Road from Grand Prairie Road to 21st Avenue	MDD	N/A	16	4,719	$222.00	$408,000	$0
	P33	Pipeline along Hwy. 20 from Gold Fish Farm Road to Scravel Hill Road, along Scravel Hill Road to Knox Butte Road	MDD	N/A	12	10,838	$176.40	$442,000	$0
	P34	Pipeline from Knox Butte Road south to existing 24-inch pipeline along Gold Fish Farm Road	MDD	N/A	24	3,269	$260.40	$408,000	$0
	P35	Pipeline from Santa Maria Avenue to Knox Butte Road east of Project P4	MDD	N/A	12	2,565	$176.40	$105,000	$0
	P36	Pipeline along Santa Maria Avenue from Scravel Hill Road to Clover Ridge Road	MDD	N/A	12	6,762	$176.40	$276,000	$0
	P37	Pipeline along Ellingson Road from elevated storage to Lochner Road	MDD	N/A	16	2,991	$222.00	$258,000	$0
	P38	Pipeline along Ellingson Road from Lochner to Columbus Street, Columbus Street to existing 16-inch pipeline	MDD/PHD	N/A	16	4,766	$222.00	$412,000	$0
							Project Total	***$3,466,000***	***$0***
ox Butte Reservoir Project, Phase 2	S8	A 5 MG concrete reservoir	Service Level					$3,500,000	$0
							Project Total	***$3,500,000***	***$0***
adway Reservoir Project	S9	An 8 MG concrete reservoir	Service Level					$5,600,000	$0
							Project Total	***$5,600,000***	***$0***
ley View Reservoir Project	S10	Replace two 0.25 MG Valley View reservoirs with a single 0.85 MG steel reservoir	Service Level					$520,000	$0
							Project Total	***$520,000***	***$0***
nt Water Project, Phase 2	JWP2	Added capacity at WTP, and 2 MG additional reservoir storage	Service Level					$3,900,000	$0
							Project Total	***$3,900,000***	***$0***
eline Replacement Programs, Phase 4	Program-3	Perpetual life pipeline replacement program	Service Level		8 / 20	452,146	$135.60 / $241.20	$66,311,000	$66,311,0
							Project Total	***$66,311,000***	***$66,311,0***
		STAGE 4 TOTAL						**$83,300,000**	**$66,300,0**
		PROGRAM TOTAL (ALL STAGES)						**$187,200,000**	**$136,500,**

PROGRAM SUMMARY[1,2]	
Stage 1	$52,600,000
Stage 2	$21,400,000
Stage 3	$29,900,000
Stage 4	$83,300,000
Total	**$187,200,000**

tnotes:
Estimated project costs are based on a March 2002 Seattle ENR CCI = 7560
osts are based on individual and grouped projects being rounded to
e nearest $1 K and stages and total program cost to the nearest $100 K
osts shown for development driven projects reflect anticipated costs for
versizing.
ee ***Appendix A***, November 5, 2002 Water Task Force Minutes for
osts allocation methodology for "shared projects"

Abbreviations:
MDD = MAXIMUM DAY DEMAND
PHD = PEAK HOUR DEMAND
MDD + FF = MAX DAY DEMAND PLUS FIRE FLOW

Joint Water Project Worksheets



The Joint Water Project, Phase 1

Project ID(s): *JWP1*

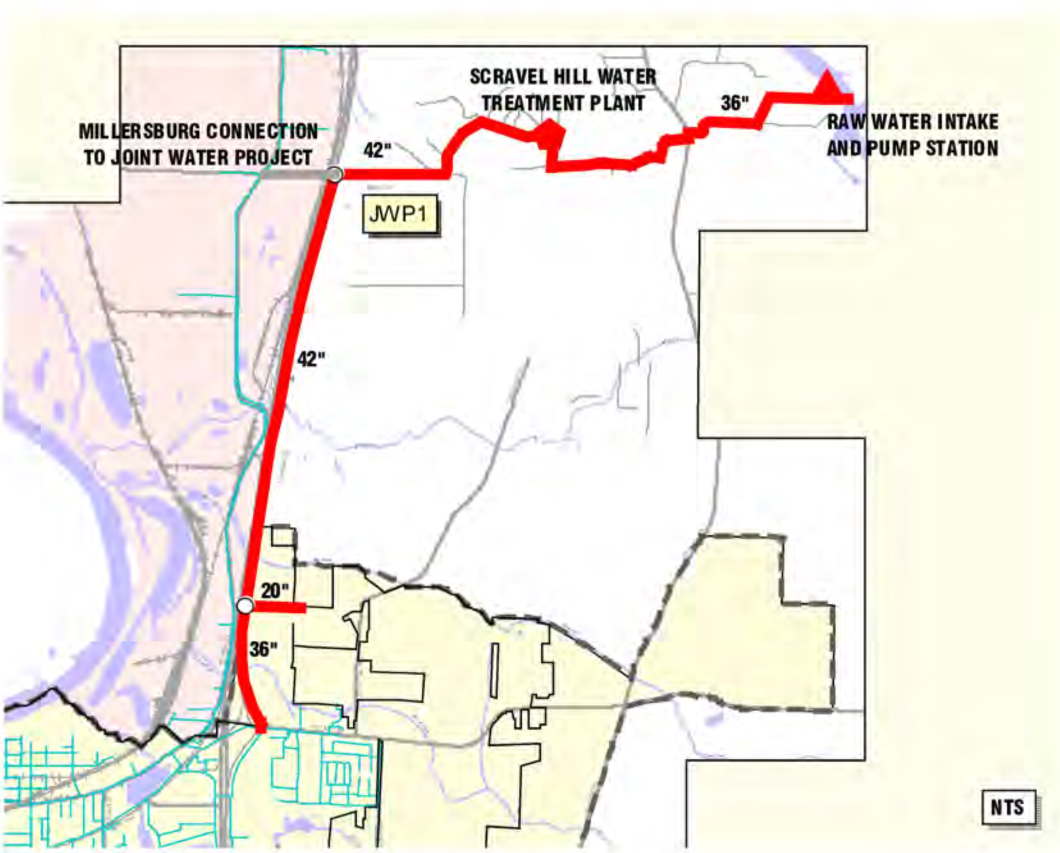
Stage(s): 1

Project Description: The Joint Water Project will be supplied by an intake on the Santiam River located approximately one-quarter mile downstream of the confluence of the North Santiam and South Santiam Rivers. Raw water will be pumped from the intake to a treatment plant located on Scravel Hill. Following treatment, finished water will be conveyed using a shared gravity flow water line from the Scravel Hill Treatment Plant Reservoir to Century Drive and Berry Drive. At this point the water line splits with branches to each community's distribution system. The plant's initial firm capacity will be 12 MGD with 10 MGD allocated to the City of Albany. Project costs listed below are for Albany's portion of the project.

Proposed improvements include:

JWP1 Intake, raw water transmission line, WTP, reservoir, finished water transmission line

Cost: *$32,300,000*



The Joint Water Project, Phase 2

Project ID(s): *JWP2*

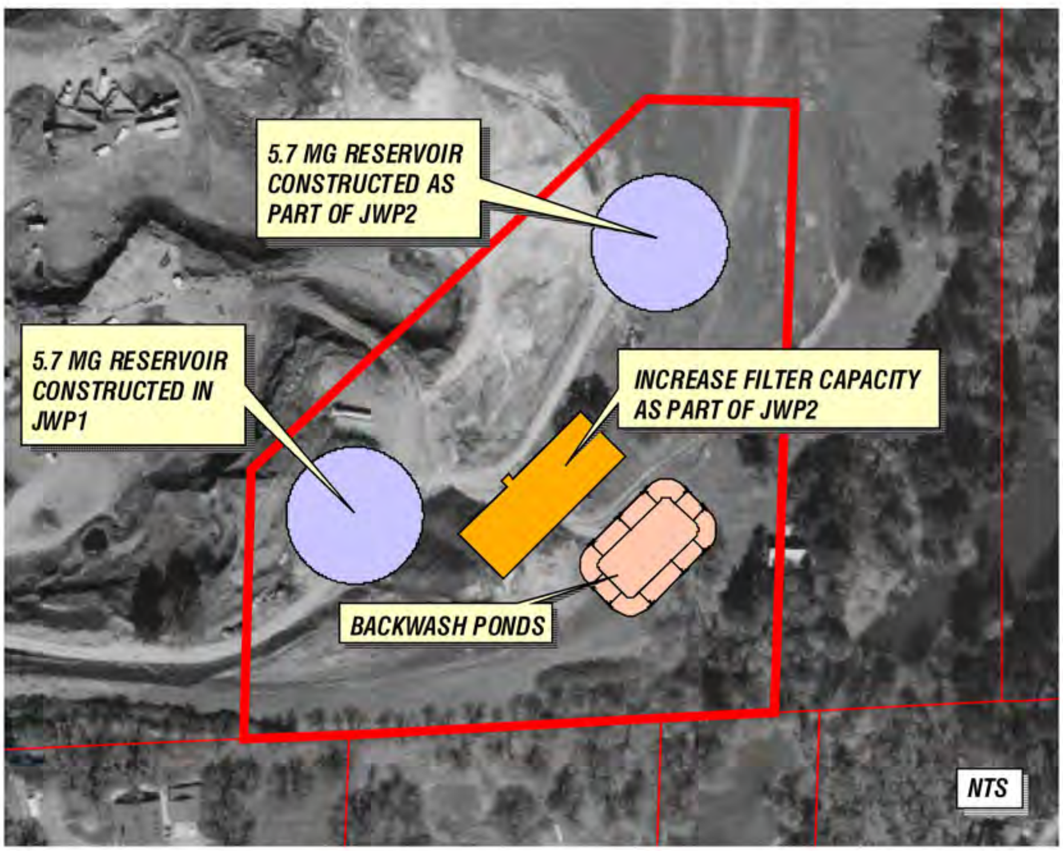
Stage(s): 4

Project Description: The Joint Water Project will be expanded by adding treatment capacity and reservoir storage at the Scravel Hill Water Treatment Plant site. The JWP2 project will satisfy the buildout treatment capacity needed for both Millersburg and Albany. Once this project is completed, the Scravel Hill WTP firm capacity will be increased to 26 MGD with 20 MGD allocated to the City of Albany. Project costs listed below are for Albany's portion of the project.

Proposed improvements include:

JWP2 Added capacity at WTP and additional reservoir storage

Cost: *$3,900,000*



Distribution System Worksheets

(Pipelines, Pump Stations and Reservoirs)

LEGEND

EXISTING FACILITIES

- *WTP*
- *RESERVOIR*
- *PUMP STATION*
- *WATERLINE*

FUTURE WATERLINE

- *STAGE 1*
- *STAGE 2*
- *STAGE 3*
- *STAGE 4*

FUTURE PUMP STATION

- *STAGE 1*
- *STAGE 2*
- *STAGE 3*
- *STAGE 4*

FUTURE RESERVOIR

- *STAGE 1*
- *STAGE 2*
- *STAGE 3*
- *STAGE 4*

East End Transmission Projects

Project ID(s): *P1, P2, P3, P4, P5, P6, & P7*

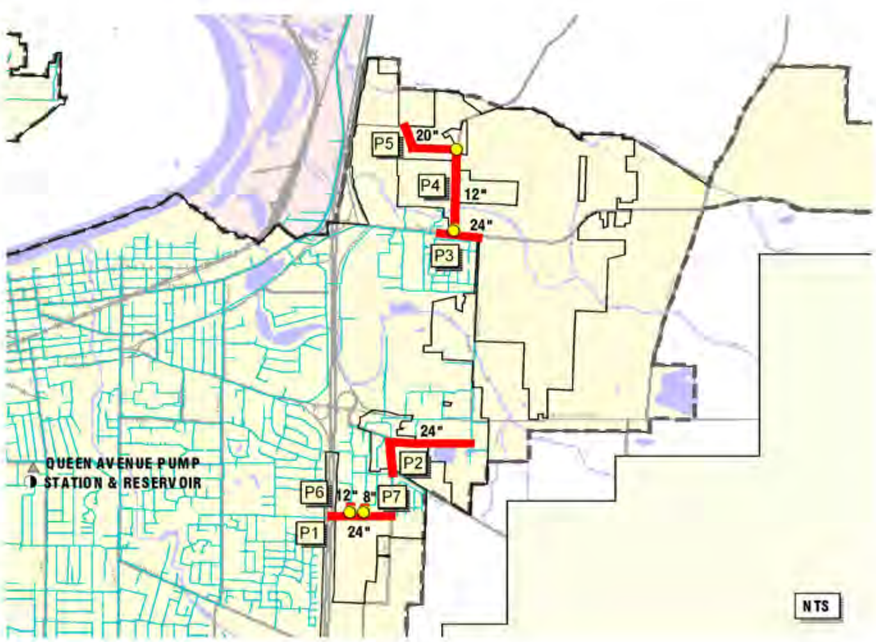
Stage(s): 1

Project Description: The East End Transmission Project is required to fully utilize the initial 10-MGD capacity generated for Albany by the Scravel Hill Water Treatment Plant. These transmission lines include approximately 10,300 feet of 24, 20, 12, and 8-inch water lines. These water lines are also needed to raise service pressures in the southeast Albany area.

Proposed improvements include:

P1 Pipeline along 21st Avenue from east of I-5 to Three Lakes Road, coordinate project with P6 & P7.
P2 Pipeline along Spicer Road from 18th Avenue to 24-inch pipeline along Goldfish Farm Road alignment south of Hwy 20.
P3 Pipeline along Knox Butte Road from Clover Ridge Road to Gold Fish Farm Road.
P4 Pipeline along Clover Ridge Road from Santa Maria Avenue to Knox Butte Road.
P5 Pipeline from the east end of Bernard Avenue to Santa Maria Avenue
P6 Pipeline from the south end of Fescue Street, connect pipeline to Project P1 along 21st Avenue.
P7 Pipeline from the south end of Rye Street, connect pipeline to Project P1 along 21st Avenue.

Cost: $2,379,000



South Albany Transmission Project

Project ID(s): *P8*

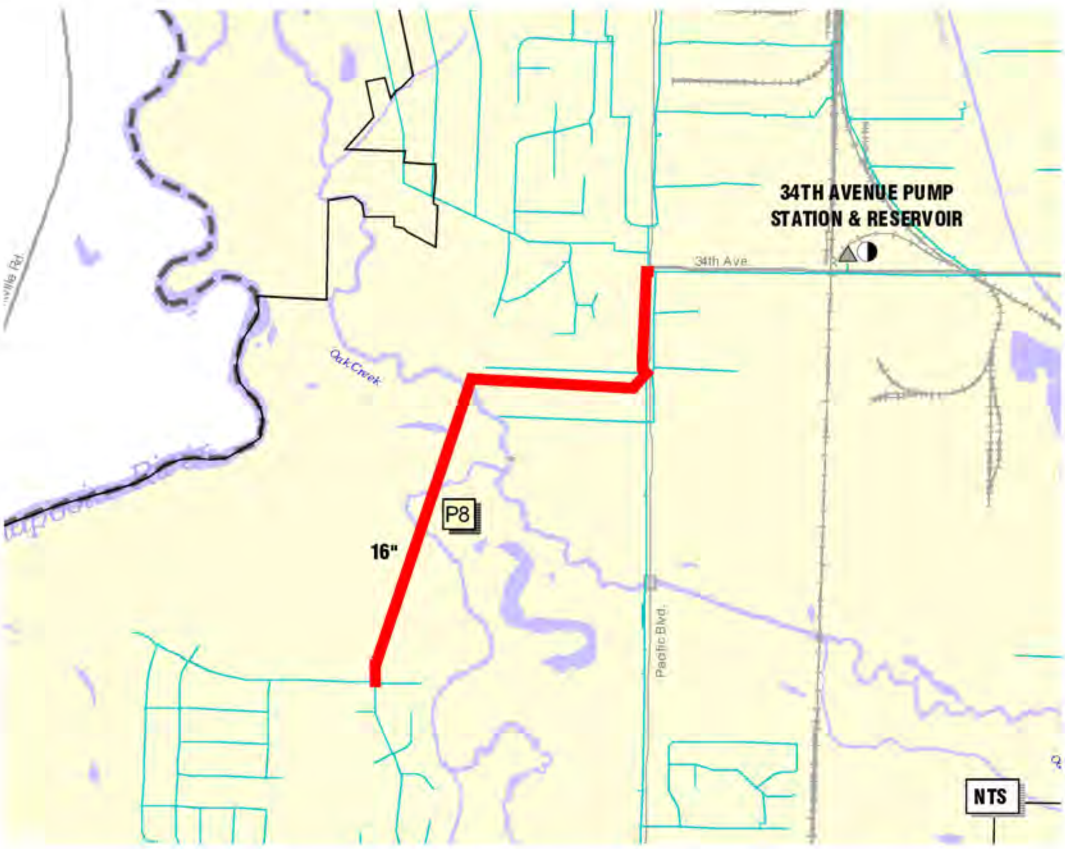
Stage(s): 1

Project Description: The South Albany Transmission Project consists of a 16-inch transmission line that begins at the intersection of Pacific Boulevard and 34th Avenue and ends at Cougar Avenue. This transmission line is needed to improve fire flows and service pressures and to provide a redundant supply line to the southwest Albany area.

Proposed improvements include:

P8 Pipeline from 34th Avenue and Hwy 99E along 99E to 36th Avenue to Elk Run Drive along Elk Run Drive to Cougar Avenue

Cost: $1,029,000



North Albany Distribution Projects, Phase 1

Project ID(s): *P9, P10, P11, P12, PS1, & PS2*

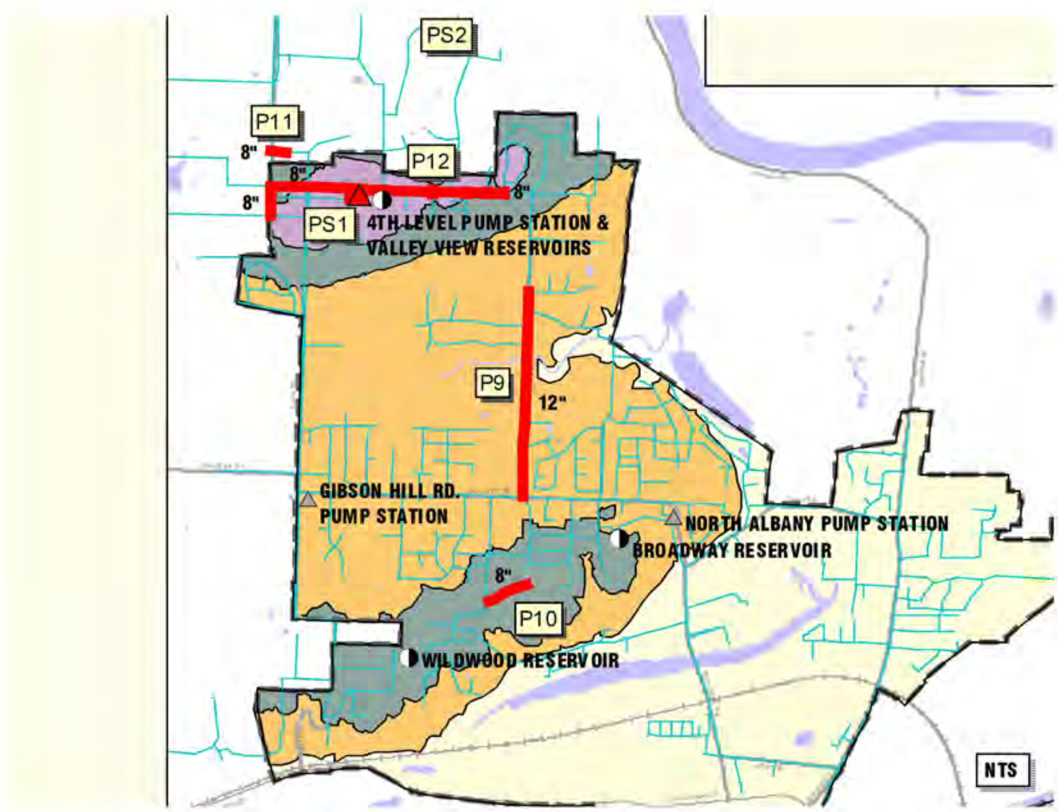
Stage(s): 1

Project Description: The North Albany Distribution, Phase 1 projects include upsizing approximately 4,500 feet (750 feet of steel) of undersized water lines located along Crocker and Maier Lanes to 8 and 12-inch water lines in order to meet fire flow requirements. This project group also includes a pump station and piping necessary to create a fourth pressure zone.

Proposed improvements include:

P9	Pipeline north along Crocker Lane from Gibson Hill Road
P10	Pipeline along Maier Lane from Skyline Drive to Chad Avenue
P11	Zone 4 separation - New 3rd level piping along Sparks Avenue, Scenic Drive, and Valley View Drive
P12	Zone 4 separation- New 4th level piping parallel to 3rd level pipeline along Valley View Drive
PS1	Zone 4 separation- Pump station with emergency backup generator (two 7.5 HP and one 100 HP pumps)
PS2	Install NA PRV on pipeline serving NW Winn Drive

Cost: $1,665,000



Reservoir Projects, Phase 1

Project ID(s): *PS3, PS4, PS5, PS6, PS7, S1, S2, S3, S4, S5*

Stage(s): 1

Project Description: This group of projects is needed to improve existing reservoirs in the distribution system. Projects include improvements to overflow piping and reservoir circulation, installation of seismic protection, dechlorination facilities, back-up power outlets, and replacement of valves and pumps.

Proposed improvements include:

PS3 34th Avenue backup power outlet
PS4 Queen pump station building security enhancement
PS5 Queen pump station motorized control valve replacement
PS6 Replace Queen pump stations pump No. 21 (30 HP)
PS7 Queen pump station backup power outlet
S1 Seismic restraints – Maple Street, 34th Avenue, Queen Avenue, and Valley View (3)
S2 Seismic valves – Maple St., 34th Avenue, Queen Avenue, Broadway, Wildwood, Valley View (3)
S3 Replace/Repair overflow piping – Maple Street, 34th Avenue, Queen Avenue, and Valley View (3)
S4 Dechlorination facilities – 34th Avenue, Broadway, Wildwood, and Valley View
S5 Increase reservoir circulation for 34th and Queen reservoirs

Cost: $1,244,000

Pipeline Replacement Programs, Phase 1, 2, 3, & 4

Project ID(s): *Program-1, Program-2 & Program-3*

Stage(s): 1, 2, 3 & 4

Project Description: These programs are required to replace deteriorating and failing steel water lines, undersized water lines that serve fire hydrants, and other pipes that have surpassed their service life.

Proposed improvements include:

Program 1	Steel pipeline replacement program
Program 2	Undersized pipelines with hydrants replacement program
Program 3	Perpetual life pipeline replacement program

Cost: $96,385,000

North Albany Distribution Projects, Phase 2

Project ID(s): *P13, P14, P15, P16, PS11*

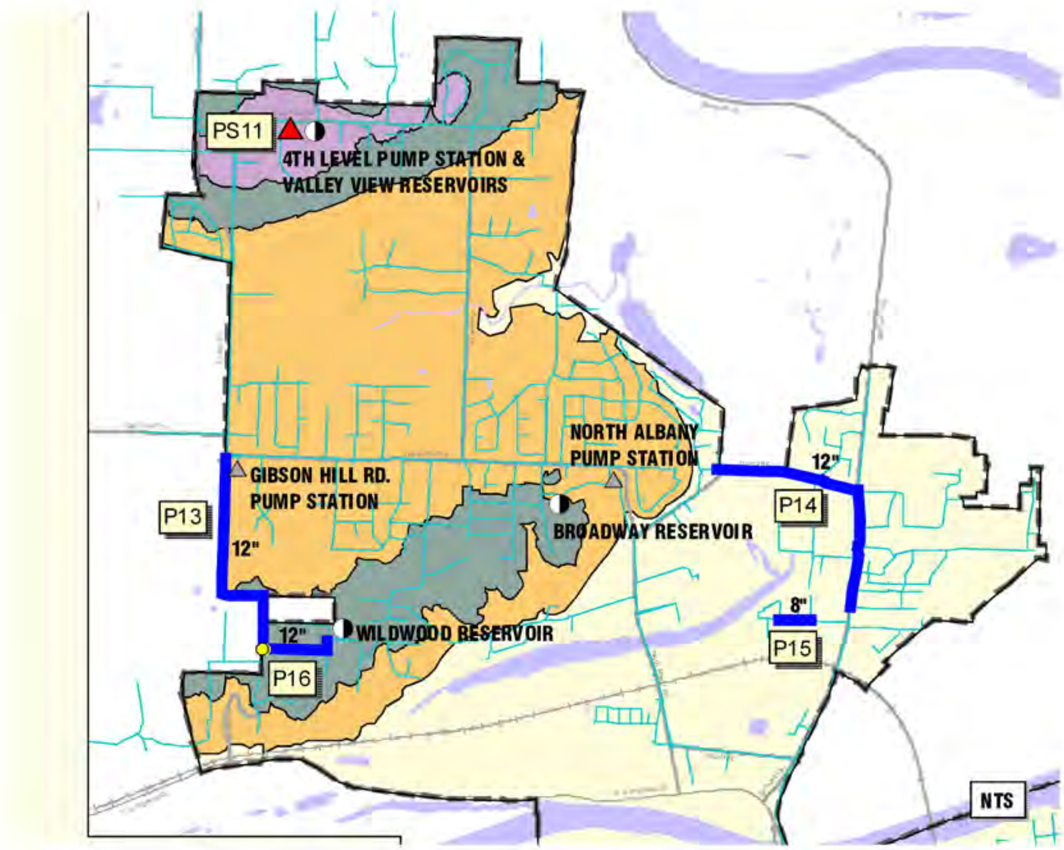
Stage(s): 2

Project Description: The North Albany Distribution Projects, Phase 2, are needed to meet fire flow requirements in the North Albany area and to expand the Zone 4 booster pump station to meet future water demands. These projects will upsize approximately 9,800 feet of water lines to 8 and 12-inch diameter water lines.

Proposed improvements include:

P13 Pipeline along Scenic Drive from Gibson Hill Road to Wildwood Drive
P14 Pipeline along Quarry Road from Christmas Tree Lane to Springhill Road, along Springhill Road to Cherry Lane
P15 Pipeline along Green Acres Lane from Shady Lane to Green Acres Loop
P16 Pipeline along Wildwood Drive from Scenic Drive to Wildwood Reservoir site
PS11 Zone 4 separation - booster pump station (One 7.5 HP pump)

Cost: $1,732,000



Zone 1 Distribution Projects

Project ID(s): *P17, P18, P19, P20, P21, P22, & P23*

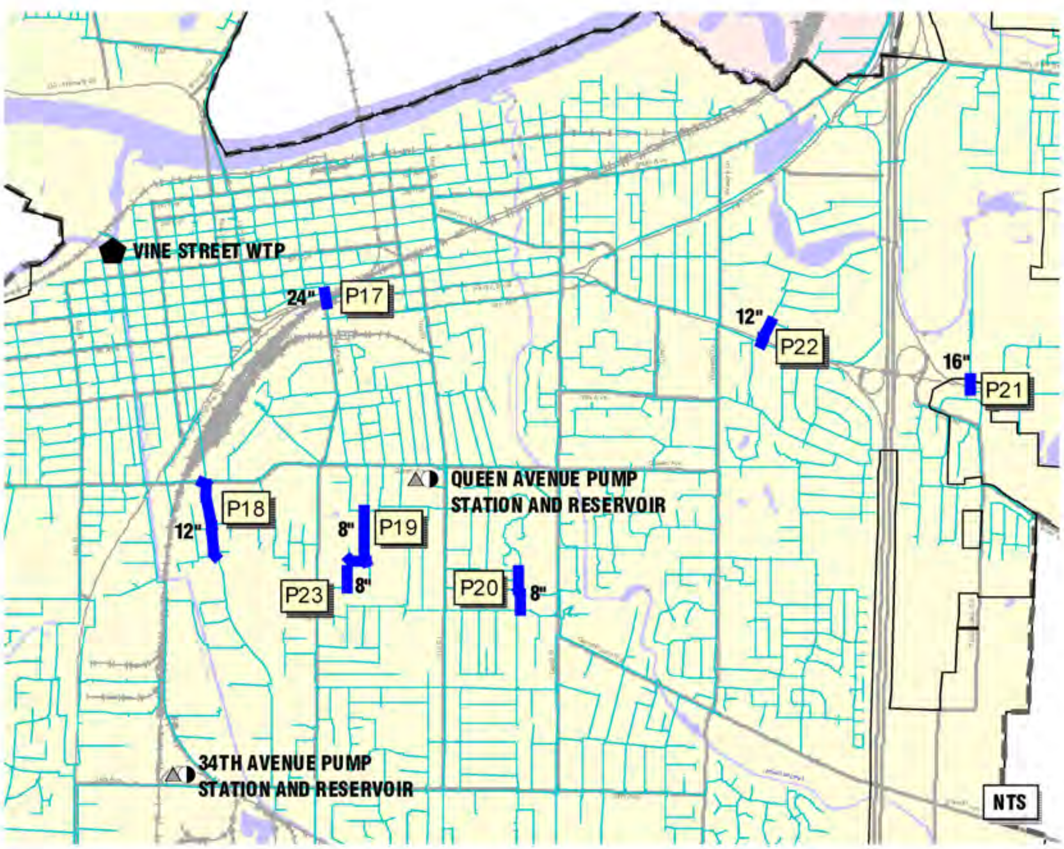
Stage(s): 2

Project Description: Zone 1 Distribution Projects include approximately 2,700 feet of water lines necessary to meet fire flow requirements and approximately 1,500 feet of water lines required to meet future peak and maximum day demands. These projects are dispersed throughout pressure Zone 1.

Proposed improvements include:

P17 Pipeline along Jackson Street north from Highway 99
P18 Pipeline along Ferry Street from Queen Avenue to 22nd Avenue
P19 Pipeline along Jefferson Street from 20th Avenue to 22nd Avenue to Jackson Street
P20 Pipeline along Oak Street north from 24th Avenue
P21 Pipeline along Price Road north from Highway 20
P22 Pipeline along Bain Street north from Highway 20
P23 Pipeline along Jackson Street north from 23rd Avenue

Cost: $677,000



Ellingson Road Reservoir Project

Project ID(s): *P24 & S6*

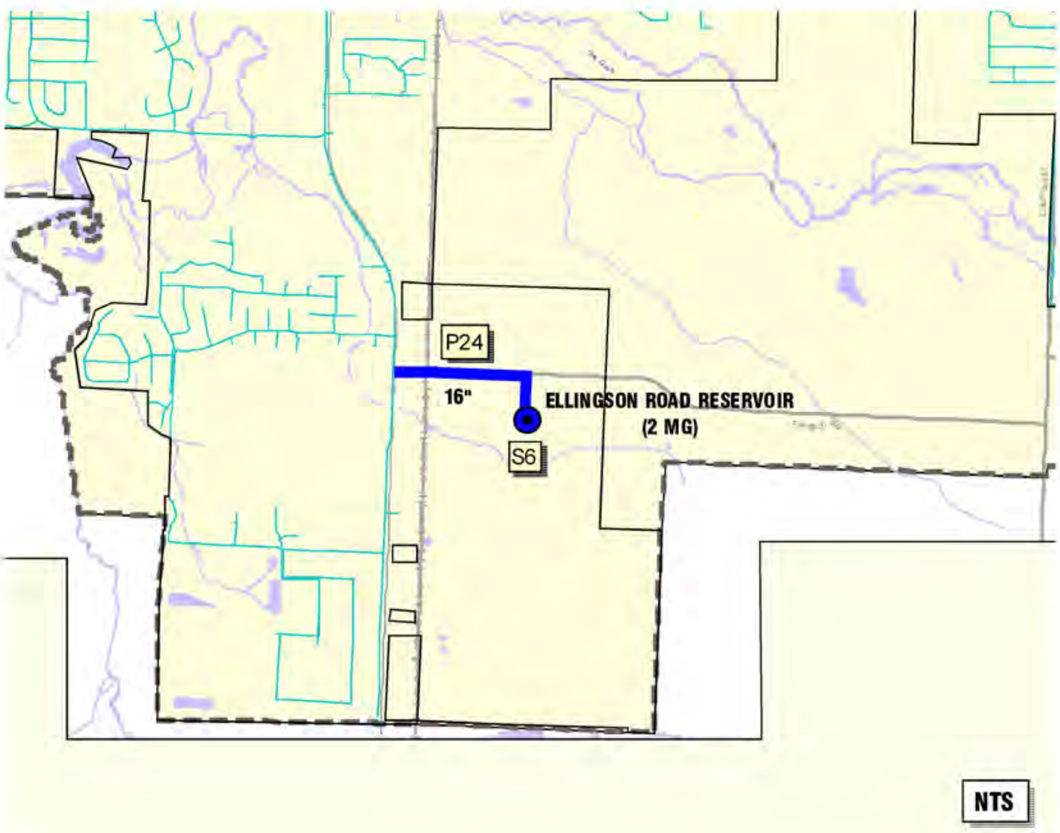
Stage(s): 2

Project Description: The Ellingson Road Reservoir Project includes a 2-MG elevated reservoir located on Ellingson Road, and approximately 2,100 feet of new 16-inch pipeline necessary to connect the reservoir to the existing water line on Pacific Boulevard. This project is needed to meet future storage requirements in Zone 1 and will provide local fire protection storage and enhanced service pressures in the Southwest Albany area.

Proposed improvements include:

P24 Pipeline along Ellingson Road from Pacific Boulevard to elevated storage
S6 A 2 MG elevated reservoir

Cost: $4,464,000



Central Albany Transmission Project

Project ID(s): *P25 & P26*

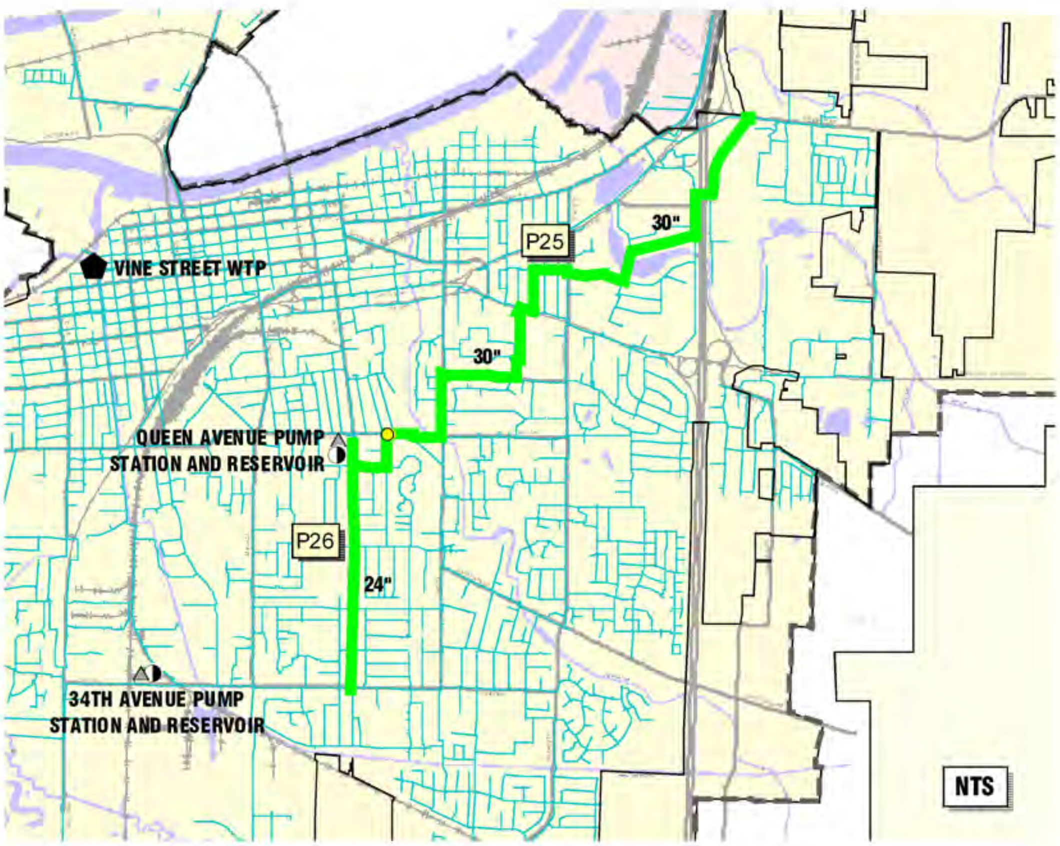
Stage(s): 3

Project Description: The Central Albany Transmission Project is required to meet future maximum day demand conditions and is required to fully realize the benefits of the future Scravel Hill WTP expansion. The project consists of approximately 14,300 feet of 30-inch water line from Knox Butte Road to Main Street and approximately 6,700 feet of 24-inch water line from Queen Avenue to 34th Avenue. This project incorporates the replacement of approximately 1.25 miles of deteriorated steel water lines. Alternate alignments for the 30" water line could be investigated if the City decides to incorporate this transmission project with steel pipeline replacement along Pacific Boulevard.

Proposed improvements include:

P25 Cross town transmission pipeline from Knox Butte Road to Main Street
P26 Cross town transmission pipeline from Queen Avenue along Main Street and Hill Street to 34th Avenue

Cost: $6,318,000



Reservoir Projects, Phase 2

Project ID(s): *PS12, PS13*

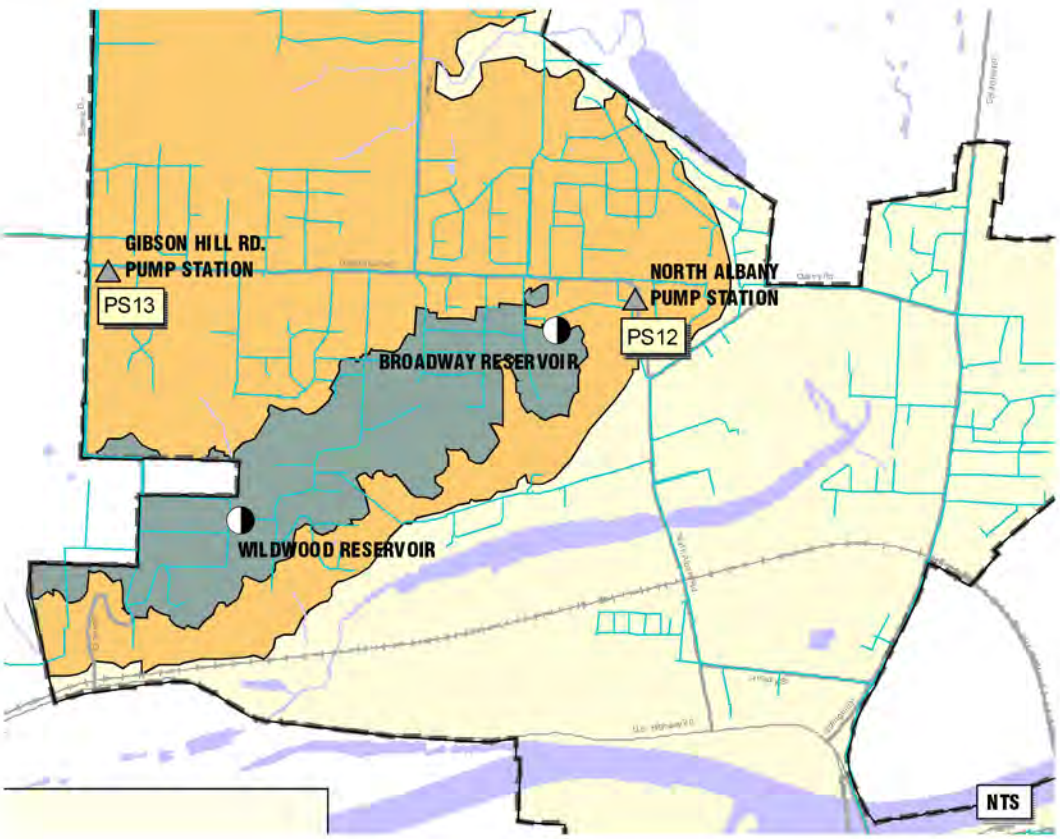
Stage(s): 3

Project Description: These improvements include replacing impellers in the 2nd and 3rd level pump stations to increase pump station capacity as upper service level demands increase.

Proposed improvements include:

PS12 Increase level 2 pump station capacity
PS13 Increase level 3 pump station capacity

Cost: $20,000



Knox Butte Reservoir Project, Phase 1

Project ID(s): *P27 & S7*

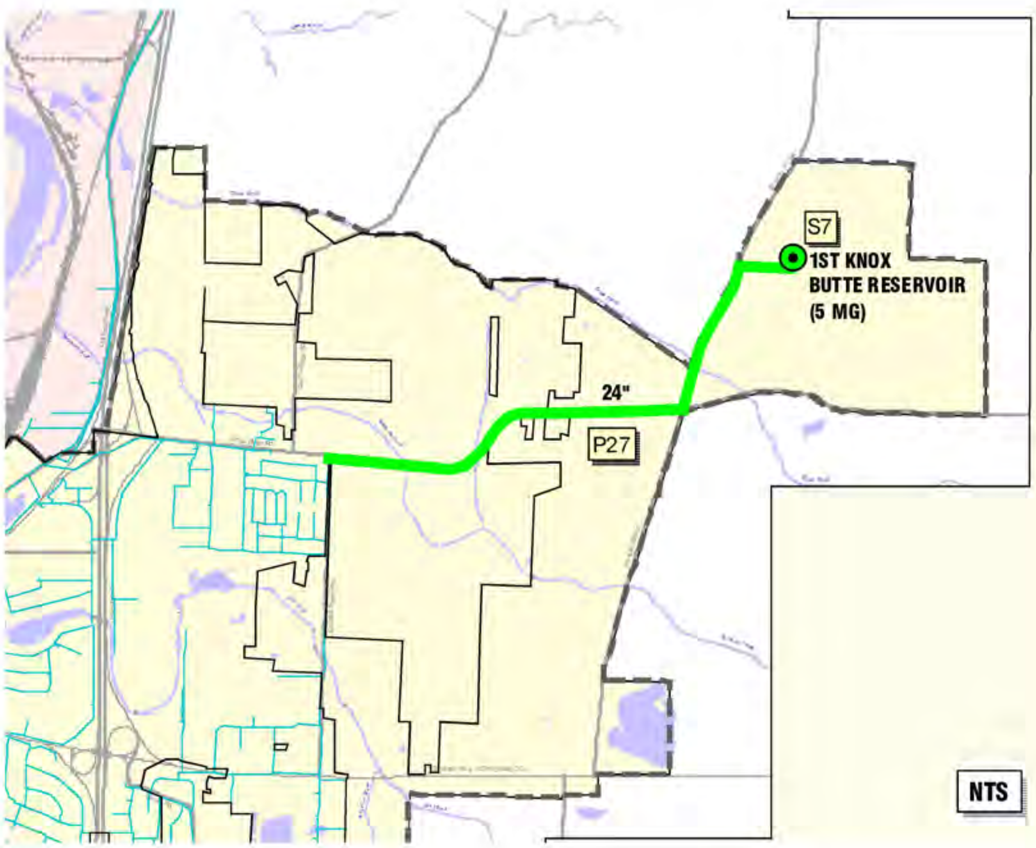
Stage(s): 3

Project Description: The Knox Butte Reservoir Project, Phase 1, includes a 5-MG concrete storage reservoir and approximately 9,700 feet of 24-inch water line necessary to connect it to the distribution system. This reservoir is needed to meet future storage requirements in Zone 1.

Proposed improvements include:

P27 Pipeline along Scravel Hill Road from Gold Fish Farm Road to proposed Knox Butte Reservoir
S7 A 5 MG concrete storage reservoir

Cost: $6,020,000



Development Driven Transmission/Distribution Projects

Project ID(s): *P28, P29, P30, P31, P32, P33, P34, P35, P36, P37, & P38*

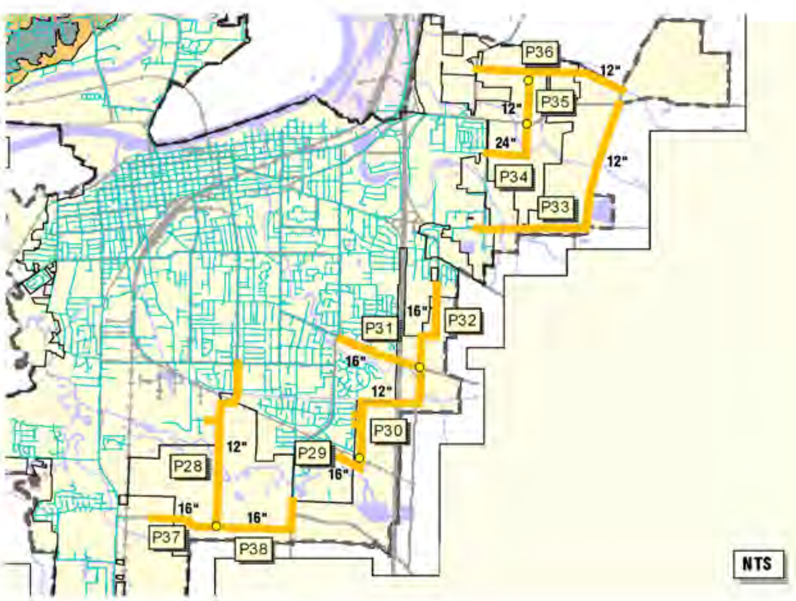
Stage(s): 4

Project Description: The Development Driven Transmission/Distribution Projects include approximately 11 miles of 12, 16, and 24-inch pipelines needed to serve future development. Timing for these projects is development dependent. Project costs presented below only include the anticipated cost to the City for oversizing. Costs for an 8 inch equivalent water line will be funded by development and are not included in the project cost shown below.

Proposed improvements include:

- P28 Pipeline from 34th Avenue along Hill Street alignment to Lochner Road, along Lochner Road to Ellingson Road
- P29 Pipeline from 47th Avenue across railroad right-of-way then southeasterly parallel to the railroad
- P30 Pipeline from P29, parallel with Shortridge Street, to 40th Avenue, east to Three Lakes Road, north to Grand Prairie Road
- P31 Pipeline along Grand Prairie Road from Three Lakes Road to the pipeline stub-out east of Waverly Drive
- P32 Pipeline along Three Lakes Road from Grand Prairie Road to 21st Avenue
- P33 Pipeline along Hwy. 20 from Gold Fish Farm Road to Scravel Hill Road, along Scravel Hill Road to Knox Butte Road
- P34 Pipeline from Knox Butte Road south to the existing 24-inch pipeline along Gold Fish Farm Road
- P35 Pipeline from Santa Maria Avenue to Knox Butte Road east of Project P4
- P36 Pipeline along Santa Maria Avenue from Scravel Hill Road to Clover Ridge Road
- P37 Pipeline along Ellingson Road from the elevated storage reservoir to Lochner Road
- P38 Pipeline along Ellingson Road from Lochner to Columbus Street, Columbus Street to an existing 16-inch pipeline

Cost: $3,466,000



Knox Butte Reservoir Project, Phase 2

Project ID(s): *S8*

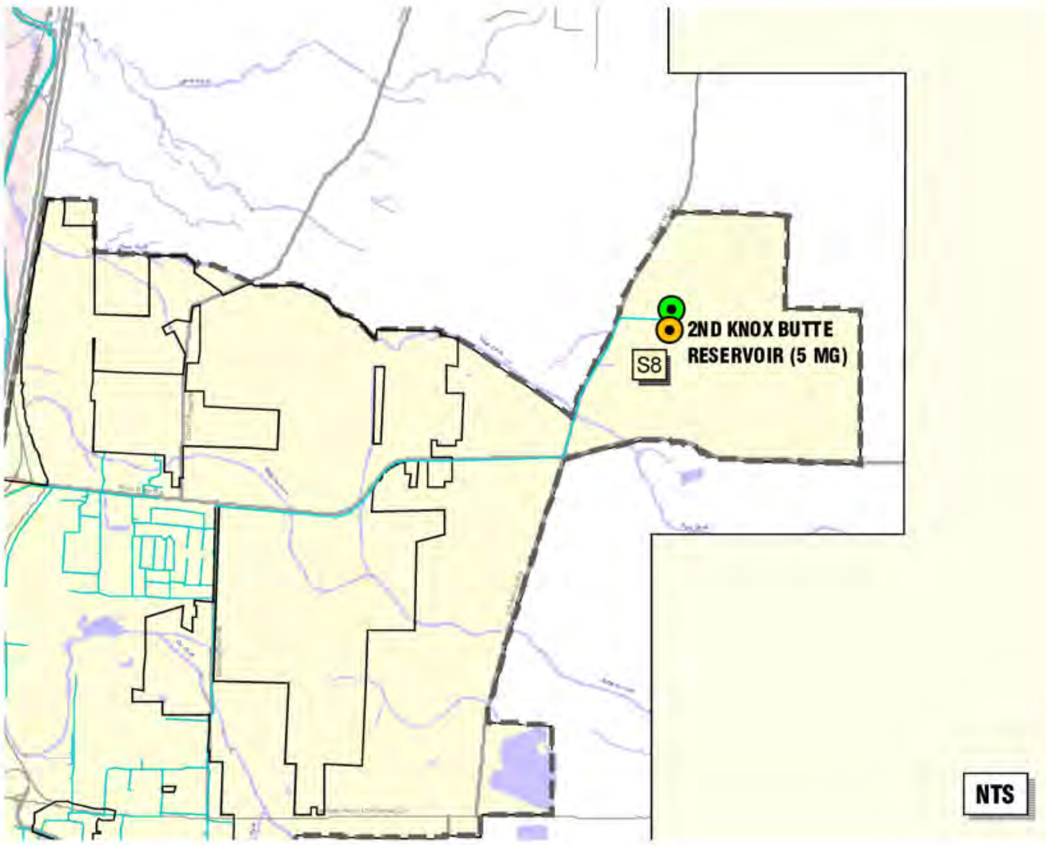
Stage(s): 4

Project Description: The Knox Butte Reservoir Project, Phase 2, includes a 5-MG concrete storage reservoir at the Knox Butte site. This reservoir is needed to meet future storage requirements in Zone 1.

Proposed improvements include:

S8 A 5 MG concrete reservoir

Cost: $3,500,000



Broadway Reservoir Project

Project ID(s): *S9*

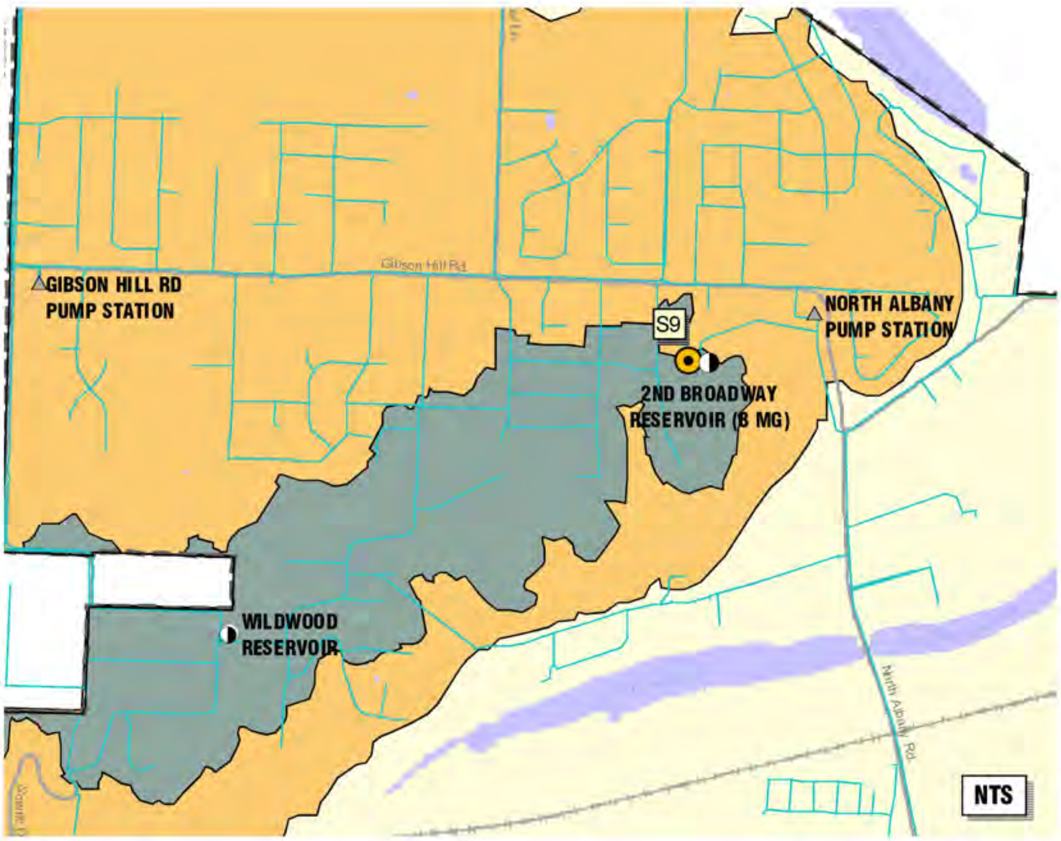
Stage(s): 4

Project Description: The Broadway Reservoir Project will add an additional 8-MG of storage to pressure Zone 1. This storage is required to satisfy equalization, emergency and fire flow storage requirements as water demands increase in pressure Zone 1.

Proposed improvements include:

S9 An 8-MG concrete reservoir

Cost: $5,600,000



Valley View Reservoir Project

Project ID(s): *S10*

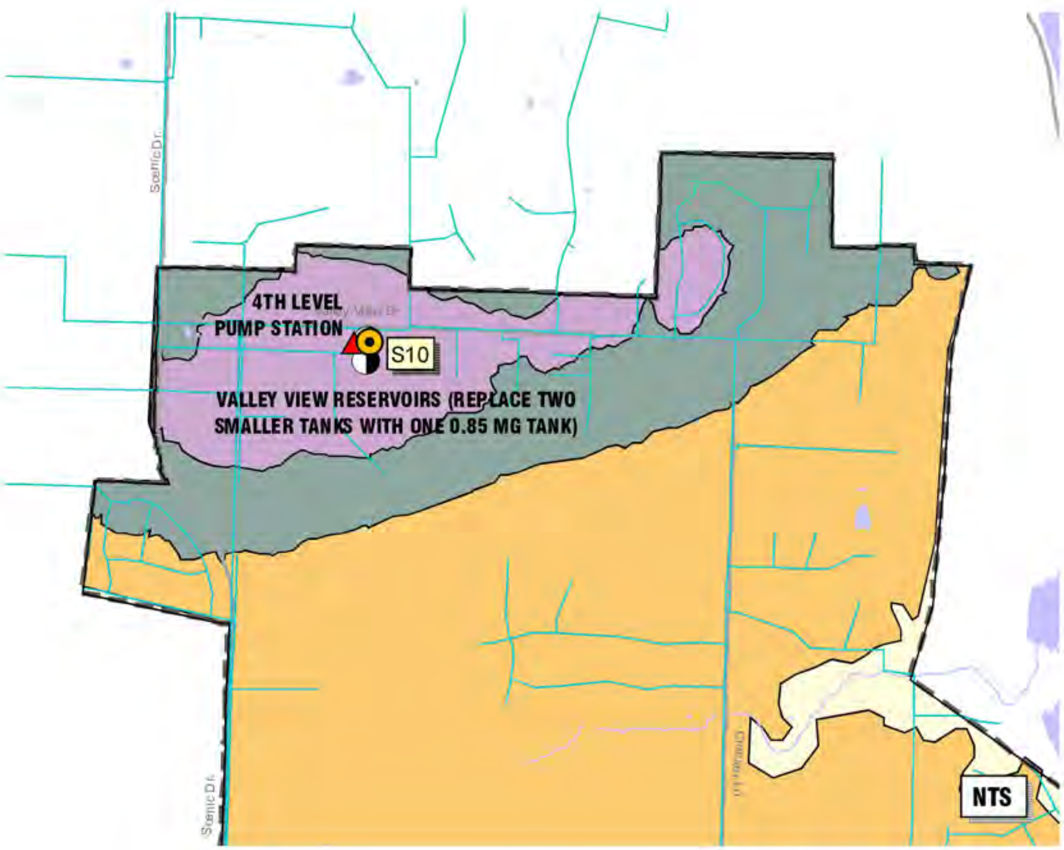
Stage(s): 4

Project Description: The Valley View Reservoir Project will provide 0.3 MG of additional storage for pressure Zones 3 and 4 by replacing two existing 0.25 MG steel reservoirs with a 0.85 MG storage reservoir.

Proposed improvements include:

S10 Replace the two 0.25 MG Valley View reservoirs with one 0.85 MG steel reservoir.

Cost: $520,000



Canal Worksheets

Update Control Structures

Project ID(s): *C1*

Stage(s): 1,2 & 3

Project Description:

Flow control improvements were identified to improve existing control capabilities along the Canal, and to provide instrumentation and control of Canal structures. Coordination of the Canal flow structures is recommended to provide relief during high flow periods. Improvements to existing gate structures, addition of new control gates, and automation and instrumentation of the gates will provide a complementary and flexible system to minimize flooding adjacent to the Canal. The picture below shows the Albany Gates. The Albany gates are one of the six existing control structures with recommended improvements.

Proposed improvements include:

Lebanon WTP and Hydropower Intake (Station 192+00)
Mark's Slough (Station 253+00)
CZ Tailrace (Hospital Slough, Station 280+00)
Albany Gates (Station 287+00)
Cox Creek (Station 538+00)
Rock dam and siphon (Station 688+00)
New control gate (Oak Creek, Station 755+00)
Communication for all structures
Master station
Develop rating curves for remote sites
Hydraulic analysis allowance for receiving drainage channels
Flow augmentation allowance

Cost: $4,000,000

Ensure Canal Capacity

Project ID(s): *C2*

Stage(s): 1 & 2

Project Description:

Based on the analysis of bridge, culvert, and sedimentation issues, the following recommendations are made to ensure the Canal will be able to convey a design flow of 310-cfs. The picture below shows a location of sedimentation build-up in the Canal.

Proposed improvements include:

Private driveway bridge (Station 117+00)
Franklin Street Bridge (Station 137+00)
KGAL Road Culvert (Station 455+00)
Lateral inflow removal
Raise Canal banks
Sediment removal

Cost: $2,900,000

Channel Restoration

Project ID(s): *C3*

Stage(s): 1

Project Description:

Channel restoration work is recommended to rehabilitate the Canal and protect water quality. Items addressed for this category include local sediment removal, debris removal, bank repair, excess vegetation and fallen tree removal to maintain capacity, and water quality issues. The picture below shows a reach of the Canal with excessive bank vegetation.

Proposed improvements include:

Allowance to repair bank damage, remove debris and excess bank vegetation, complete preliminary Cheadle Lake seepage analysis.

Cost: $1,000,000

Improve Canal Access

Project ID(s): *C4*

Stage(s): 1 & 2

Project Description:

Actions recommended under this category include removing excessive bank vegetation and securing legal and physical access along the Canal where practical, such as commissioning a right-of-way survey and removing right-of-way encroachments. The picture below shows an example of an encroachment on Canal right-of-way.

Proposed work includes:

Allowance for ROW survey, removing encroachments, securing ROW, and removing heavy bank vegetation.

Cost: $500,000

Vine Street WTP Worksheets

Replace HSPS Pump No. 14 (200 HP)

Project ID(s): *PS8*

Stage(s): 1

Project Description: The engineering firm of Brown and Caldwell completed an evaluation of the HSPS in 2001. This project is in response to needs identified in the evaluation of Pump No. 14. This improvement will allow pump No. 14 to operate in tandem with the other pumps at the HSPS.

Proposed improvements include:

- 200 HP pump

Cost: $75,000.

HSPS Backup Power Outlet

Project ID(s): *PS9*

Stage(s): 1

Project Description: This project is needed to increase the reliability of the HSPS. A backup power outlet for use with a portable generator is recommended for the HSPS to provide limited power and pumping capacity during power outages. The outlet would be configured to allow a quick connection and transition to power generated on site.

Proposed improvements include:

- Install backup power outlet at the HSPS

Cost: $30,000.

Analysis of Operating Conditions Including VFDs at HSPS

Project ID(s): *PS10*

Stage(s): 1

Project Description: This project involves an analysis of operating conditions at the HSPS and includes the possible use of VFDs to address gaps in the range of available flows for the HSPS.

Proposed improvements include:

- Completion of a study that evaluates the operating conditions at the HSPS including the potential use of VFDs.

Cost: $55,000.

Water Quality Monitoring Upgrades

Project ID(s): *WTP01*

Stage(s): 1

Project Description: Provide additional on-line and bench-top water quality analysis capabilities at the WTP, including instrumentation for settled water turbidity, filtered water particle counters and TOC analysis. Cost estimates include integration with the existing SCADA system.

Proposed instrumentation improvements include:

- 2 new on-line turbidimeters—one on each of the Accelator's effluent pipelines,
- 10 new on-line particle counters—one for each of the existing filters, and
- 1 bench-top UV Spectrophotometer to measure TOC surrogate.

Cost: $84,000.

Backwash/Surface Wash Piping System Improvements

Project ID(s): *WTP02*

Stage(s): 1

Project Description: Provide automatic backwash control capabilities to optimize treatment performance, enhance reliability and reduce risk of filter damage. Replace existing surface wash system and upsize filter-to-waste system pipes and valves to 8-inch in filters 1 through 6. Provide redundant backwash pump for the larger filters by replacing the existing smaller constant speed backwash pump. Improvements include valve and piping modifications where necessary, as well as instrumentation and control facilities to control backwash flow.

Proposed improvements include:

- One new 75 hp, 7,500 gpm vertical turbine backwash pump with VFD,
- Six new surface wash systems for the smaller filters, and
- Six new filter-to-waste pipeline/valves for the smaller filters.
- Incorporation of automatic backwash capabilities, and
- Backwash flow control.

Cost: $329,000

Replace Accelator Settling Tubes

Project ID(s): *WTP03*

Stage(s): 1

Project Description: This project includes a thorough inspection of the mechanical systems and replacement of the tube settlers within Accelator #2.

Proposed work includes:

- Engineering report to evaluate mechanical systems
- Replacement of the existing tube settlers in Accelator #2
- Sand blast/re-coat Accelator's interior

Cost: $213,000

Plant Pipeline Inspection and Cleaning

Project ID(s): *WTP04*

Stage(s): 1

Project Description: Routine, thorough inspections and cleaning of the large diameter pipelines (> 20-inch) at the WTP are required to minimize the accumulation of material in these water lines. This project includes the installation of several new pipeline clean-outs for introduction and retrieval of pipeline cleaning equipment. Once completed, the City will be able to perform video inspections of all larger diameter pipelines to determine degree of corrosion/material accumulation, particularly the 24-inch steel transfer line.

Proposed improvements include:

- Installation of 5 new above-ground pipeline clean-outs,
- Installation of 5 new below-grade pipeline clean-outs, and
- Inspection and report.

Cost: $112,000

Repair Maple Street Reservoir Baffle and Improve Disinfection Performance

Project ID(s): *WTP05*

Stage(s): 1

Project Description: Repairs to the interior baffle wall and inlet/outlet piping to minimize short-circuiting through the Maple Street Reservoir are needed. With proper baffling, chlorine contact time (represented by T_{10} values greater than or equal to 0.60) should be achieved.

Proposed improvements include:

- Installation of new baffle curtain, and
- Improvements to existing inlet/outlet piping configuration – install new deflector pipe to divert flow away from new curtain.
- Close gap between the reservoir wall and baffle curtain that may be contributing to the short circuiting of the baffle.

Cost: $115,000

Chlorine System Safety Improvements

Project ID(s): *WTP06*

Stage(s): 1

Project Description: Modify the current chlorine storage and feed system at the plant to comply with current UBC/UFC requirements. Install liquid sodium hypochlorite delivery and storage facility, as well as a chemical metering facility. Cost estimates include integration with existing SCADA system.

Proposed improvements include:

- Installation of 2 new 6,000 gallon storage tanks, and
- Installation of 2 new chemical metering pumps (1 duty, 1 standby), and piping.

Cost: $140,000

Replace/Repair Control Room HVAC System

Project ID(s): *WTP07*

Stage(s): 1

Project Description: Install new heating and ventilation/air conditioning system in the Control Room Building.

Proposed improvements include:

- Detailed HVAC system evaluation, and
- Installation of heating, ventilation and air conditioning system.

Cost: $70,000

VFD Harmonics Evaluation

Project ID(s): *WTP08*

Stage(s): 1

Project Description: The Vine Street WTP's electrical capacity is adequate to meet current demands; however, harmonics, phasing, and grounding should be evaluated. This project provides a thorough evaluation of the plant's electrical system, including the grounding grid.

Proposed evaluation includes:

- Inspection and report.

Cost: $20,000

ADA/OSHA Compliance Upgrades

Project ID(s): *WTP09*

Stage(s): 1

Project Description: Although not required unless substantial structural modifications are made, an allowance of $50,000 has been included to complete access and safety improvements consistent with the historic nature of the structure.

Proposed upgrades include:

- Handrails
- Walkways
- Platforms
- Other facilities required to meet ADA and OSHA compliance.

Cost: $50,000

WTP Automation Upgrade – Plant Work

Project ID(s): *WTP10*

Stage(s): 1

Project Description: Replace the existing Siemens TI PLC (control hardware and software) system with a redundant Allen-Bradley PLC5 System.

Proposed improvements include:

- Preliminary Design
- System design and programming modifications,
- PLC/radio hardware and software,
- Electrical installation, and
- System start-up, testing and commissioning.

Cost: $535,000

WTP Automation Upgrade – Distribution Work

Project ID(s): *WTP11*

Stage(s): 1

Project Description: Replace the six on site communications and control units (located at the 34th Avenue Reservoir and Pump Station site, Queen Avenue Reservoir and Pump Station site, Broadway Reservoir, North Albany Pump Station, Gibson Hill Road Pump Station, and the Valley View Reservoir site.) with new, Scadapak RTU units. These six units would be configured to utilize DataRadio communications via the new DataRadio master installed at the WTP.

- Preliminary Design,
- System Design and programming modifications,
- PLC/radio hardware and software costs, and
- System start-up, testing and commissioning.

Cost: $127,000

WTP Security Upgrade

Project ID(s): *WTP12*

Stage(s): 1

Project Description: This project includes the evaluation and implementation of a cost effective security system at the Vine Street WTP based on the results of the system wide security assessment (Project Planning-1).

Proposed improvements include:

- Engineering investigation and report
- Installation of cameras, gate keypads and integration into the City's SCADA network.

Cost: $150,000

WTP Filter Gallery Maintenance

Project ID(s): *WTP13*

Stage(s): 1 & 2

Project Description: This project replace all valves and actuators associated with Filters 1-8; filter-to-waste valves in Filters 1-6 have been accounted for in project WTP02. In addition, the piping in both filter pipe galleries will be re-coated.

Proposed improvements include:

- Re-coat filter gallery piping,
- Replace filter influent valves and actuators (2 x 20-inch, 6 x 10-inch),
- Replace filter effluent valves and actuators (2 x 20-inch, 6 x 10-inch),
- Replace filter-to-waste valves and actuators (2 x 10-inch),
- Replace backwash inlet valves and actuators (2 x 30-inch, 6 x 20-inch), and
- Replace backwash outlet valves and actuators (2 x 30-inch, 6 x 20-inch).

Cost: $560,000

Clearwell Repairs

Project ID(s): *WTP14*

Stage(s): 2

Project Description: Investigate and confirm structural damage and cracks in the clearwell; repair if needed. Install new clearwell drain line and valve for dewatering.

Proposed improvements include:

- Structural investigation and possible repairs to the clearwell, and
- Installation of drain pipeline and valve for clearwell dewatering

Cost: $70,000

Chemical Storage Improvements

Project ID(s): *WTP15*

Stage(s): 2

Location: Chemical Building—2nd Floor

Project Description: Construction of containment area around the liquid alum storage tank to contain potential leaks.

Proposed Improvements Include:

- Construction of a containment wall around the existing alum storage tank.

Cost: $28,000

Solids Handling

Project ID(s): *WTP16*

Stage(s): 2

Project Description: Improve the storage capacity and minimize leaching from the existing backwash ponds. Install a sump on each of the two existing ponds for dewatering.

Proposed improvements include:

- Reshape and re-line two existing backwash ponds.
- Install two sumps.

Cost: $220,000.

Seismic Upgrades

Project ID(s): *WTP17*

Stage(s): 2

Project Description: Conduct detailed evaluation and design of seismic compliance upgrades at the WTP to define plant vulnerability to seismic events.

Proposed evaluation includes:

- Engineering inspection and report, and
- Structural improvements to the WTP including:
  - Primary pipeline restraints
  - Structural reinforcement of brick structures.

Cost: $570,000

Distribution System Monitoring Improvements

Project ID(s): *WTP18*

Stage(s): 2

Project Description: Install 5 pressure transmitters on distribution system piping to increase overall system monitoring. Pressure transmitters should be installed in close proximity to existing wastewater pumping stations to facilitate ease of data acquisition and transfer. Data will be transmitted to the control room at the Vine Street WTP.

Proposed improvements include:

- 5 pressure transmitters, wiring and manholes
- Instrumentation and control upgrade.

Cost: $70,000

Replace Accelator Settling Tubes

Project ID(s): *WTP19*

Stage(s): 2

Project Description: This project replaces Accelator #1's worn settling tubes. Settling tubes will degrade due to normal wear and tear and light degradation of the plastic.

Proposed improvements include:

- Replace settling tubes on Accelator #1

Cost: $210,000

Repair/Replace Filter Media / Underdrain System

Project ID(s): *WTP20*

Stage(s): 2

Project Description: This project removes existing filter media and underdrains. The underdrains will be replaced with new, "gravel-less" plastic block underdrains. Install at least 30-inches of dual media (18-inches of anthracite over 12-inches of sand, for a total bed depth of 30-inches). Remove existing troughs and replace with new fiberglass troughs. Replace existing surface-wash system piping and agitators on filters 7 through 10 only.

Proposed equipment improvements include:

- Installation of 3,250 sf of "gravel-less" plastic block underdrains,
- Installation of 4,900 cf of 1.0 mm anthracite,
- Installation of 3,250 cf of 0.5 mm sand,
- New fiberglass troughs for both the new and old filters, and
- Four new surface wash systems for the larger filters.

Cost: $682,000

Add Granular Activated Carbon (GAC) to Filter Media

Project ID(s): *WTP21*

Stage(s): 2

Project Description: Incorporate granular activated carbon (GAC) to protect against organic contaminants which may be present in the raw water. GAC will increase the reliability and safety of the WTP. Note: costs associated with this CIP only include the additional incremental costs of GAC over anthracite as discussed in WTP20.

Proposed improvements includes:

- Installation of 4,900 cf of GAC.

Cost: $150,000

Valve Maintenance

Project ID(s): *WTP22*

Stage(s): 2 & 3

Project Description: This project replaces remaining plant valves and actuators not replaced through projects WTP02 and WTP13.

Proposed improvements include:

- Raw water isolation valves
- RWPS pump isolation, and check valves
- Settled water isolation valves
- Accelator drain valves
- Filter influent valves and actuators
- Filter effluent valves and actuators
- Filter-to-waste valves and actuators
- Backwash inlet valves and actuators
- Backwash outlet valves and actuators
- Backwash drain isolation valve
- Maple Street Reservoir inlet/outlet valves

Cost: $994,000

Plant Hydraulics

Project ID(s): *WTP23*

Stage(s): 3

Project Description: This project removes a hydraulic bottleneck that exists between the Accelators and filters. This bottleneck limits the flow to the six smaller filters, resulting in approximately 2,400 gpm (3.5 MGD) reduction in capacity. Costs associated with this improvement project include improvements to minimize air entrainment from the accelators in the pipeline, as well as related filter gallery-piping improvements.

Proposed Improvements Include:

- Replace the open channel that separates the old and new filters with a pipeline, and
- Install equipment to minimize air entrainment between the Accelators and filters.

Cost: $280,000

Instrumentation and Control Improvements

Project ID(s): *WTP24*

Stage(s): 3

Project Description: This project replaces the control hardware with a new control system. Costs associated with this improvement project include: preliminary design, upgrades to the remote telemetry units (RTUs), as well as replacement of the main plant PLC and upgrades to the SCADA system at the WTP.

Proposed instrumentation and control improvements include:

- Preliminary design,
- Upgrades to the remote telemetry units (RTUs)
- Replacement of the main plant PLC and upgrades to the SCADA system.

Cost: $840,000.

Replace Maple Street Reservoir Baffle

Project ID(s): *WTP25*

Stage(s): 3

Project Description: The useful life of the baffle replaced in project WTP05 is expected to be reached within the 2025 planning window. This project addresses the need for replacement of the baffle curtain inside the Maple Street Reservoir near the end of the Vine Street WTP planning window.

Proposed improvements include:

- Installation of new baffle curtain

Cost: $80,000

Planning Projects Worksheets

System Wide Security Assessment

Project ID(s): *Planning-1*

Stage(s): 1

Project Description: A system wide security assessment is required under the Public Health Security and Bioterrorism Preparedness Response Act amendment to the Safe Drinking Water Act. This project provides funding for preparation of a plan meeting the requirements of the act, including a system vulnerability assessment and follow-up emergency preparedness plan sufficient to meet regulatory requirements.

Proposed improvements include:

- System wide security assessment

Cost: $150,000

Facility Plan Updates

Project ID(s): *Planning-2*

Stage(s): 2 & 3

Project Description: Water facility plan updates are required periodically to reflect changes in expected growth patterns and demands, the regulatory environment and capital improvement needs. On average facility plan updates are completed on 10-year cycles. Two water facility plan updates have been included in the cost estimate below.

Proposed improvements include:

- 2 water facility plan updates by 2024

Cost: $600,000

WTF MINUTES

MWH
MONTGOMERY WATSON HARZA

CANAL FIELD INSPECTION DATA

MWH
MONTGOMERY WATSON HARZA

VINE STREET WTP DETAILED EVALUATION

MWH
MONTGOMERY WATSON HARZA

PUMP CURVE DATA

MWH
MONTGOMERY WATSON HARZA

FIELD INSPECTION DATA

MWH
MONTGOMERY WATSON HARZA

JWP INTERGOVERNMENTAL AGREEMENT

MWH
MONTGOMERY WATSON HARZA

JWP PREDESIGN MEMORANDUMS

MWH
MONTGOMERY WATSON HARZA

COUNCIL RESOLUTION #3363

MWH
MONTGOMERY WATSON HARZA

EXISTING SYSTEM EVALUATION RESULTS

MWH
MONTGOMERY WATSON HARZA

GLOSSARY

GLOSSARY OF TERMINOLOGY

Acre-Foot (AF) — A unit commonly used for measuring the volume of water; equal to the quantity of water required to cover one acre (43,560 square feet or 4,047 square meters) to a depth of 1 foot (0.30 meter) and equal to 43,560cubic feet (1,234 cubic meters), or 325,851 gallons.

Agricultural Water Use (Withdrawals) — Includes water used for irrigation and non-irrigation purposes. Irrigation water use includes the artificial application of water on lands to promote the growth of crops and pasture, or to maintain vegetative growth in recreational lands, parks, and golf courses. Non-irrigation water use includes water used for livestock, which includes water for stock watering, feedlots, and dairy operations, and fish farming and other farm needs.

Aquifer — (1) A geologic formation, a group of formations, or a part of a formation that is water bearing. (2) A geological formation or structure that stores or transmits water, or both, such as to wells and springs. (3) An underground layer of porous rock, sand, or gravel containing large amounts of water. Use of the term is usually restricted to those water-bearing structures capable of yielding water in sufficient quantity to constitute a usable supply.

Basin — A geographic area drained by a single major stream; consists of a drainage system comprised of streams and often natural or man-made lakes. Also referred to as *Drainage Basin*, *Watershed*, or *Hydrographic Region*.

Beneficial Use (of Water) — (1) A use of water resulting in appreciable gain or benefit to the user, consistent with state law, which varies from one state to another. Oregon recognizes but is not limited to the following general categories of beneficial use:

[1] municipal uses;

[2] industrial uses;

[3] commercial uses;

[4] domestic uses; and

[3] irrigation;

(2) The cardinal principle of the *(Prior) Appropriation Doctrine*. A use of water that is, in general, productive of public benefit, and which promotes the peace, health, safety and welfare of the people of the State. A certificated water right is obtained by putting water to a beneficial use. The right may be lost if beneficial use is discontinued. A beneficial use of water is a use that is of benefit to the appropriator and to society as well. The term encompasses considerations of social and economic value and efficiency of use. In the past, most reasonably efficient uses of water for economic purposes have been considered beneficial. Usually, challenges have only

been raised to wasteful use or use for some non-economic purpose, such as preserving instream values. Recent statutes in some states have expressly made the use of water for recreation, fish and wildlife purposes, or preservation of the environment a beneficial use.

Best Management Practices (BMP) — Water conservation measures that generally meet one of two criteria: (1) Constitutes an established and generally accepted practice that provides for the more efficient use of existing water supplies or contributes towards the conservation of water; or (2) Practices which provide sufficient data to clearly indicate their value, are technically and economically reasonable, are environmentally and socially acceptable, are reasonably capable of being implemented by water purveyors and users, and for which significant conservation or conservation-related benefits can be achieved.

Clean Water Act (CWA) [Public Law 92–500] — More formally referred to as the *Federal Water Pollution Control Act*, the Clean Water Act constitutes the basic federal water pollution control statute for the United States. Originally based on the *Water Quality Act* of 1965 that began setting water quality standards. The 1966 amendments to this act increased federal government funding for sewage treatment plants. Additional 1972 amendments established a goal of zero toxic discharges and "fishable" and "swimmable" surface waters. Enforceable provisions of the CWA include technology-based effluent standards for point sources of pollution, a state-run control program for nonpoint pollution sources, a construction grants program to build or upgrade municipal sewage treatment plants, a regulatory system for spills of oil and other hazardous wastes, and a *Wetlands* preservation program (Section 404).

Clean Water Act (CWA), Section 319 — A federal grant program added by Congress to the CWA in 1987 and managed by the *U.S. Environmental Protection Agency (EPA)*, Section 319 is specifically designed to develop and implement state *Nonpoint Source (NPS) Pollution* management programs, and to maximize the focus of such programs on a watershed or waterbasin basis with each state. Today, all 50 states and U.S. territories receive Section 319 grant funds and are encouraged to use the funding to conduct nonpoint source assessments and revise and strengthen their nonpoint source management programs. Before a grant is provided under Section 319, states are required to: (1) complete a Nonpoint Source (NPS) Assessment Report identifying state waters that require nonpoint source control and their pollution sources; and (2) develop Nonpoint Source Management Programs that outline four-year strategies to address these identified sources.

Commercial Water Use (Withdrawals) — Water for motels, hotels, restaurants, office buildings, and other commercial facilities and institutions, both civilian and military. The water may be obtained from a public supply or may be self supplied. The terms "water use" and "water withdrawals" are equivalent, but not the same as *Consumptive Use* as they do not account for return flows. Also see *Industrial Water Use (Withdrawals)* and *Public Supply Water*.

Consumptive (Water) Use — (1) A use which lessens the amount of water available for another use (e.g., water that is used for development and growth of plant tissue or consumed by humans or renders it no longer available because it has been evaporated, transpired by plants, incorporated into products or corps, consumed by people or livestock, or otherwise removed from water supplies. (3) The portion of water withdrawn from a surface or groundwater source that is consumed for a particular use (e.g., irrigation, domestic needs, and industry), and does not return to its original source or another body of water. The terms *Consumptive Use* and *Nonconsumptive Use* are traditionally associated with water rights and water use studies, but they are not completely definitive. No typical consumptive use is 100 percent efficient; there is always some return flow associated with such use either in the form of a return to surface flows or as a ground water recharge. Nor are typically nonconsumptive uses of water entirely nonconsumptive. There are evaporation losses, for instance, associated with maintaining a reservoir at a specified elevation to support fish, recreation, or hydropower, and there are conveyance losses associated with maintaining a minimum streamflow in a river, diversion canal, or irrigation ditch.

Cubic Feet Per Second (CFS) — A unit expressing rate of discharge, typically used in measuring streamflow. One cubic foot per second is equal to the discharge of a stream having a cross section of 1 square foot and flowing at an average velocity of 1 foot per second. It also equals a rate of approximately 7.48 gallons per second, 448.83 gallons per minute. 1.9835 acre-feet per day, or 723.97 acre-feet per year.

Cubic Feet Per Second Day (CFS-Day) — The volume of water represented by a flow of one cubic foot per second for 24 hours. It equals 86,400 cubic feet, 1.983471 acre-feet, or 646,317 gallons.

Curtailment Program. – A system of incentives or mandatory restrictions intended to encourage conservation and/or forcibly restrict water use as a means of reducing the peak day demand. Also called WATER RESTRICTIONS.

Diurnal — Refers to a cycle that repeats daily. Specifically, the diurnal water demand curve for a water distribution system specifies the relative water demand of the system as a function of the time of day and thus shows times of peak demand and low demand. This diurnal curve may be determined by monitoring the entire system demand as a function of time, and it is important in the proper design and accurate modeling the water system.

Domestic Water Use (Withdrawals) — Water used normally for residential purposes, including household use, personal hygiene, drinking, washing clothes and dishes, flushing toilets, watering of domestic animals, and outside uses such as car washing, swimming pools, and for lawns, gardens, trees and shrubs. The water may be obtained from a public supply or may be self supplied. The terms "water use" and "water withdrawals" are equivalent, but not the same as *Consumptive Use* as they do not account for return flows. Also referred to as *Residential Water Use*. Also see *Public Supply Water*.

Drought — There is no universally accepted quantitative definition of drought. Generally, the term is applied to periods of less than average or normal precipitation over a certain period of time sufficiently prolonged to cause a serious hydrological imbalance resulting in biological losses (impact flora and fauna ecosystems) and/or economic losses (affecting man). In a less precise sense, it can also signify nature's failure to fulfill the water wants and needs of man.

Duty (of Water) — (1) The total volume of water per year that may be diverted under a vested water right. (2) The total volume of irrigation water required for irrigation in order to mature a particular type of crop. In stating the duty, the crop, and usually the location of the land in question, as well as the type of soil, should be specified. It also includes consumptive use, evaporation and seepage from on-farm ditches and canals, and the water that is eventually returned to streams by percolation and surface runoff.

Endangered Species — Any plant or animal species threatened with extinction by man-made or natural changes throughout all or a significant area of its range; identified by the Secretary of the Interior as "endangered", in accordance with the 1973 *Endangered Species Act (ESA)*, below.

Endangered Species Act (ESA) — An act passed by Congress in 1973 intended to protect species and subspecies of plants and animals that are of "aesthetic, ecological, educational, historical, recreational and scientific value." It may also protect the listed species' "critical habitat", the geographic area occupied by, or essential to, the protected species. The *U.S. Fish and Wildlife Service (USFWS)* and the *National Marine Fisheries Service (NMFS)* share authority to list endangered species, determine critical habitat and develop recovery plans for listed species. Currently, approximately 830 animals and 270 plants are listed as endangered or threatened nationwide at Title 50, Part 17, sections 11 and 12 of the Code of Federal Regulations. Further, under a settlement with environmental groups, USFWS has agreed to propose listing another 400 species over the next few years. The 1973 Endangered Species Act superseded and strengthened the *Endangered Species Preservation Act* of 1966 and the *Endangered Species Conservation Act* of 1969. The 1973 provisions required that the act be re-authorized by Congress every five years.

Firm Capacity — The capacity of a facility when the largest component of the facility is not on-line. For example the firm capacity of a pumps station would be the maximum pumping capacity of the station when the largest pump is turned off, or in the case of equal capacity pumps, when any one of the pumps is turned off. In a water treatment plant the components used as the basis for the firm capacity rating would be the filters.

Fiscal Year (FY) — The 12-month period, from July 1 through June 30, used by the City of Albany in budget formulation and execution. The fiscal year is designated by the calendar year in which it ends.

Flood, or Flood Waters — (1) An overflow of water onto lands that are used or usable by man and not normally covered by water. Floods have two essential characteristics: The inundation of land is temporary; and the land is adjacent to and inundated by overflow from a river, stream, lake, or ocean. (2) As defined, in part, in the *Standard Flood Insurance Policy (SFIP)*: "A general and temporary condition of partial or complete inundation of normally dry land areas from overflow of inland or tidal waters or from the unusual and rapid accumulation or runoff of surface waters from any source."

Flood, 100-Year — A 100-year flood does not refer to a flood that occurs once every 100 years, but rather to a flood level with a 1 percent or greater chance of being equaled or exceeded in any given year. Areas below the 100 year flood level are termed special flood hazard areas. Areas between the 100-year and the 500-year flood boundaries are termed *Moderate Flood Hazard Areas*. The remaining areas are above the 500-year flood level and are termed *Minimal Flood Hazard Areas*.

Gage, or Gauge — (1) An instrument used to measure magnitude or position; gages may be used to measure the elevation of a water surface, the velocity of flowing water, the pressure of water, the amount of intensity of precipitation, the depth of snowfall, etc. (2) The act or operation of registering or measuring magnitude or position. (3) The operation, including both field and office work, of measuring the discharge of a stream of water in a waterway.

Gallons per Capita (Person) per Day (GPCD) — An expression of the average rate of domestic and commercial water demand, usually computed for public water supply systems. Depending on the size of the system, the climate, whether the system is metered, the cost of water, and other factors, *Public Water Supply Systems (PWSS)* in the United States experience a demand rate of approximately 60 to 150 gallons per capita per day. Also see *Gallons per Employee per Day (GED)* for information on the application of this concept to commercial water use by *Standard Industrial Classification (SIC) Code*.

Gallons per Employee (Worker) per Day (GED, or GPED) — A measure or coefficient expressing an area's commercial water use per worker (employee), typically for distinct industry sectors. It is based on an analytical technique for measuring and forecasting commercial water use in a service area based upon the unique, seasonal, business-related water use by specific industrial sectors. GED commercial water-use coefficients are typically developed based upon Standard Industrial Classifications (SIC) codes for which comparable commercial water use and employment data are available. For forecasting more frequently than annually, GED coefficients will incorporate seasonal patterns (monthly or quarterly) as well. By deriving forecasts of trends in industry sector employment and combining them with appropriate, industry-specific GED coefficients, relatively accurate forecasts of the corresponding commercial water use may be obtained.

Ground Water, also Groundwater — (1) Generally, all subsurface water as distinct from *Surface Water*; specifically, the part that is in the saturated zone of a defined aquifer. (2) Water that flows or seeps downward and saturates soil or rock, supplying springs and wells. The upper level of the saturate zone is called the Water Table. (3) Water stored underground in rock crevices and in the pores of geologic materials that make up the earth's crust. Ground water lies under the surface in the ground's *Zone of Saturation*, and is also referred to as *Phreatic Water*.

HAA5 — Refers to the five regulated haloacetic acids, Dibromoacetic Acids (DBAA), Dichloroacetic Acids (DCAA), Monobromoacetic Acid (MBAA), Monochloroacetic Acid (MCAA), and Trichloroacetic Acid (TCAA). When water containing natural organic matter is treated with any disinfectant, disinfection by-products including HAAs form. These by-products are thought to cause cancer and thus health-based standards have been set by the US Environmental Protection Agency (EPA) for the maximum allowable concentration of HAAs in public water systems. The content of HAAs is usually measured in parts per million (ppb) or mg/L.

HEC-RAS — Acronym for Army Corps of Engineer's Hydraulic Engineering Center (HEC) River Analysis System (RAS) which is a computer aided water surface profile model for modeling of steady, one-dimensional, gradually varied flow in both natural and man-made river channels.

Industrial Water Use (Withdrawals) — Industrial water use includes water used for processing activities, washing, and cooling. Major water-using manufacturing industries include food processing, textile and apparel products, lumber, furniture and wood products, paper production, printing and publishing, chemicals, petroleum, rubber products, stone, clay, glass and concrete products, primary and fabricated metal industries, industrial and commercial equipment and electrical, electronic and measuring equipment and transportation equipment. The terms "water use" and "water withdrawals" are equivalent, but not the same as *Consumptive Use* as consumptive uses do not account for return flows. Also see *Commercial Water Use (Withdrawals)*.

Instream Flow or Instream Use — (1) The amount of water remaining in a stream, without diversions, that is required to satisfy a particular aquatic environment or water use. (2) Nonconsumptive water requirements that do not reduce the water supply; water flows for uses within a defined stream channel. Examples of instream flows include:

[1] ***Aesthetics*** — Water required for maintaining flowing steams, lakes, and bodies of water for visual enjoyment;

[2] ***Fish and Wildlife*** — Water required for fish and wildlife;

[3] ***Navigation*** — Water required to maintain minimum flow for waterborne commerce;

[4] ***Quality Dilution*** — Water required for diluting salt and pollution loading to acceptable concentrations; and

[5] ***Recreation*** — Water required for outdoor water recreation such as fishing, boating, water skiing, and swimming.

Irrigation Water Use (Withdrawals) — Artificial application of water on lands to assist in the growing of crops and pastures or to maintain vegetative growth on recreational lands, such as parks and golf courses. The terms "water use" and "water withdrawals" are equivalent, but not the same as *Consumptive Use* as consumptive uses do not account for return flows.

Junior (Water) Rights — A junior water rights holder is one who holds rights that are temporarily more recent than senior rights holders. All water rights are defined in relation to other users, and a water rights holder only acquires the right to use a specific quantity of water under specified conditions. Therefore, when limited water is available, junior rights are not met until all senior rights have been satisfied.

Livestock Water Use — Water use for stock watering, feed lots, dairy operations, fish farming, and other on-farm needs. Livestock as used here includes cattle, sheep, goats, hogs, and poultry. Also included are such animal specialties as horses, rabbits, bees, pets, fur-bearing animals in captivity, and fish in captivity.

Municipal and Industrial (M & I) Water Withdrawals (Use) — Water supplied for municipal and industrial uses provided through a municipal distribution system for rural domestic use, stock water, steam electric powerplants, and water used in industry and commerce.

National Environmental Policy Act (NEPA) — A 1970 Act of Congress that requires all federal agencies to incorporate environmental considerations into their decision-making processes. The act requires an *Environmental Impact Statement (EIS)* for any "major federal action significantly affecting the quality of the human environment."

Non-Point Source (NPS) Pollution — (1) Pollution discharged over a wide land area, not from one specific location. (2) Water pollution caused by diffuse sources with no discernible distinct point of source, often referred to as runoff or polluted runoff from agriculture, urban areas, mining, construction sites and other sites. These are forms of diffuse pollution caused by sediment, nutrients, organic and toxic substances originating from land use activities, which are carried to lakes and streams by surface runoff.

Perfected Water Right — (1) A completed or fully executed water right. A water right is said to have been perfected when all terms and conditions associated with it have been fully accomplished, e.g., the diversion has been effected and the water applied to beneficial use. (2) A water right to which the owner has applied for and obtained a permit, has complied with the conditions of the permit, and has obtained a license or certification of appropriation. (3) A water right that indicates that the uses anticipated by an applicant, and made under permit, were made for *Beneficial Use*. Usually it is irrevocable unless voluntarily canceled or forfeited due to several consecutive years of nonuse. Also referred to as a *Certified Water Right*.

Permit — (1) (Water Right) A written document that grants authority to take unused water and put it to *Beneficial Use*. If all requirements of the permit are satisfied, then the permit for water appropriation can mature into a license or *Perfected Water Right*. (2) (Discharge) A legally binding document issued by a state or federal permit agency to the owner or manager of a point source discharge. The permit document contains a schedule of compliance requiring the permit holder to achieve a specified standard or limitation (by constructing treatment facilities or modifying plant processes) by a specified date. Permit documents typically specify monitoring and reporting requirements to be conducted by the applicant as well as the maximum time period over which the permit is valid. Also see *Water Right*.

pH — A measure of the relative acidity or alkalinity of water. It is defined as the negative log (base 10) of the hydrogen ion concentration and is also known as the hydrogen potential of a solution. Pure water with a pH of 7 is neutral; lower pH levels indicate increasing acidity, while pH levels above 7 indicate increasingly basic solutions.

Pig (cleaning) — Refers to a poly pig , which is a bullet-shaped device made of hard rubber or similar material. This device is used to clean pipes. It is inserted in one end of a pipe, moves through the pipe under pressure, and is removed from the other end of the pipe.

Planning — A comprehensive study of present trends and of probable future developments, together with recommendations of policies to be pursued. Planning embraces such subjects as population growth and distribution; social forces; availability of land, water, minerals, and other natural resources; technological progress; and probable future revenues, expenditures, and financial policies. Planning must be responsive to rapidly changing conditions.

Planning Horizon — The overall time period considered in the planning process that spans all activities covered in or associated with the analysis or plan and all future conditions and effects or proposed actions which would influence the planning decisions.

Point Source (PS) Pollution — (1) Pollution originating from any discrete source. (2) Pollutants discharged from any distinct, identifiable point or source, including pipes, ditches, channels, sewers, tunnels, wells, containers of various types, concentrated animal-feeding operations, or floating craft. Also referred to as *Point Source of Pollution*. Also see *Non-Point Source (NPS) Pollution*.

Priority — The concept that the person first using water has a better right to it than those commencing their use later. An appropriator is usually assigned a "priority date". However, the date is not significant in and of itself, but only in relation to the dates assigned other water users from the same source of water. Priority is only important when the quantity of available water is insufficient to meet the needs of all those having a right to use water.

Public Interest, or Public Welfare — An interest or benefit accruing to society generally, rather than to any individuals or groups of individuals in the society. In many states, a permit to appropriate water must be denied if the appropriation would be contrary to the public interest or public welfare. These terms are sometimes vague and state engineers or others administering the water permit systems generally have viewed narrowly the authority granted under such provisions. In some cases they have restricted their consideration to matters of economic efficiency or the effects of the proposed appropriation on existing or future use for the water and have not considered such things as the environmental effects. However, recent developments, such as state environmental policy acts or legislation addressing specific public interest criteria, have placed new emphasis on this issue.

Public Supply Water — (1) Water withdrawn for all users by public and private water suppliers and delivered to users that do not supply their own water. (2) Water withdrawn by and delivered to a public water system regardless of the use made of the water. Includes water supplied both by large municipal systems and by smaller quasi-municipal or privately-owned water companies. Water suppliers provide water for a variety of uses, such as *Domestic Water Use* (also referred to as *Residential Water Use*), *Commercial Water Use*, *Industrial Water Use*, and *Public Water Use*

Public Water Use — Water supplied from a *Public Water Supply System (PWSS)* and used for such purposes as fire fighting, street washing, and municipal parks, golf courses, and swimming pools. Public water use also includes system water losses (water lost to leakage) and brine water discharged from desalination facilities. Also referred to as *Utility Water Use*.

Rate. - A unit of measure or reference of quantity. These terms are used frequently throughout the document, and may typically mean either the amount of water that is treated or consumed. The rate of production may be expressed as 300 gallons per minute and the rate of demand or consumption might be 246 gallons per household per day. Carefully note the context the term is used in.

Residential Water Use — Water used normally for residential purposes, including household use, personal hygiene, and drinking, watering of domestic animals, and outside uses such as car washing, swimming pools, and for lawns, gardens, trees and shrubs. The water may be obtained from a public supply or may be self supplied. Also referred to as *Domestic Water Use*. Also see *Public Supply Water*.

Riparian — Pertaining to the banks of a river, stream, waterway, or other, typically, flowing body of water as well as to plant and animal communities along such bodies of water.

Riparian Areas (Habitat) — (1) Land areas directly influenced by a body of water. Usually such areas have visible vegetation or physical characteristics showing this water influence. Stream sides, lake borders, and marshes are typical riparian areas. Generally refers to such areas along flowing bodies of water.

River Mile (RM): Specific location along a river or stream that designates how far that point is upstream from the mouth of the river.

Safe Drinking Water Act (SDWA) (Public Law 93–523) — An amendment to the *Public Health Service Act* that established primary and secondary quality standards for drinking water. The SDWA was passed in 1976 to protect public health by establishing uniform drinking water standards for the nation. In 1986 SDWA Amendments were passed that mandated the *U.S. Environmental Protection Agency (EPA)* to establish standards for 83 drinking water contaminants by 1992 and identify an additional 25 contaminants for regulation every 3 years thereafter.

SDC (Systems Development Charge) — A fee charged to new development to help pay for the capital costs associated with new growth. This fee is usually assessed as part of the permitting fees and is based on development's demand on the community's infrastructure.

Senior Rights — A senior rights holder is one who holds rights that are older (more senior) than those of junior rights holders. All water rights are defined in relation to other users, and a water rights holder only acquires the right to use a specific quantity of water under specified conditions. Thus, when limited water is available, senior rights are satisfied first in the order of their *Priority Date*.

Stream — A general term for a body of flowing water; natural water course containing water at least part of the year.

Surface Water — (1) An open body of water such as a stream, lake, or reservoir. (2) Water that remains on the earth's surface; all waters whose surface is naturally exposed to the atmosphere, for example, rivers, lakes, reservoirs, ponds, streams, impoundments, seas, estuaries, etc., and all springs, wells, or other collectors directly influenced by surface water. (3) A source of drinking water that originates in rivers, lakes and run-off from melting snow. It is either drawn directly from a river or captured behind dams and stored in reservoirs.

Total Dissolved Solids (TDS) — (Water Quality) A measure of the amount of material dissolved in water (mostly inorganic salts). Typically aggregates of carbonates, bicarbonates, chlorides, sulfates, phosphates, nitrates, etc. of calcium, magnesium, manganese, sodium, potassium, and other actions that form salts. The inorganic salts are measured by filtering a water sample to remove any suspended particulate material, evaporating the water, and weighing the solids that remain. An important use of the measure involves the examination of the quality of drinking water. Water that has a high content of inorganic material frequently has taste problems and/or water hardness problems. The common and synonymously used term for TDS is "salt". Usually expressed in milligrams per liter.

TOC (Total Organic Carbon) — Refers to the total content of organically bonded carbon in a sample of water. This carbon is differentiated from inorganically bonded carbon such as that occurring in bicarbonate groups.

Transfer (Water Right) — (1) The process of transferring a water right from one person to another. (2) A passing or conveyance of title to a water right; a permanent assignment as opposed to a temporary lease or disposal of water. Most states require that some formal notice or filing be made with an appropriate state agency so that the transaction is officially recorded and the new owner is recorded as the owner of the water right.

TTHM (Total Trihalomethanes)— Refers to the total content of trihalomethanes in water. Trihalomethanes are derivatives of methane, CH_4, in which three halogen atoms are substituted for three of the hydrogen atoms. These molecules, known as THMs, are Trichloromethane (chloroform), Dibromochloromethane, Bromodichloromethane, and Tribromomethane (bromoform). When water containing organic material is treated with any disinfectant, by-products including THMs form. These chemicals are suspected of causing cancer and thus are regulated by the US Environmental Protection Agency. The maximum allowable annual average of TTHMs is set by the EPA and is usually measured in parts per billion (ppb).

Turbidity — A measure of the reduced transparency of water due to suspended material that carries water quality implications. The term "turbid" is applied to waters containing suspended matter that interferes with the passage of light through the water or in which visual depth is restricted. The turbidity may be caused by a wide variety of suspended materials, such as clay, silt, finely divided organic and inorganic matter, soluble colored organic compounds, plankton and other microscopic organisms and similar substances.

UGB (Urban Growth Boundary) — A site-specific line on a map that separates existing and future urban development from rural lands. Urban levels and densities of development, complete with urban levels of services, are planned within the UGB. Outside the UGB, rural lands are planned for farm and forest uses or for rural levels of development with accompanying rural levels of services.

Water Conservation — The physical control, protection, management, and use of water resources in such a way as to maintain crop, grazing, and forest lands, vegetative cover, wildlife, and wildlife habitat for maximum sustained benefits to people, agriculture, industry, commerce, and other segments of the national economy. The extent to which these actions actually create a savings in water supply depends on how they affect new water use and depletion.

Water Management — (1) (General) Application of practices to obtain added benefits from precipitation, water, or water flow in any of a number of areas, such as irrigation, drainage, wildlife and recreation, water supply, watershed management, and water storage in soil for crop production. Includes *Irrigation Water Management* and *Watershed Management*. (2) (Irrigation Water Management) The use and management of irrigation water where the quantity of water used for each irrigation is determined by the water-holding capacity of the soil and the need for the crop, and where the water is applied at a rate and in such a manner that the crop can use it efficiently and significant erosion does not occur. (3) (Watershed Management) The analysis, protection, development, operation, or maintenance of the land, vegetation, and water resources of a drainage basin for the conservation of all its resources for the benefit of its residents. Watershed management for water production is concerned with the quality, quantity, and timing of the water which is produced.

Water Plan — A document of issues, policies, strategies and action plans intended to effectively and economically execute a *Water Planning* process.

Water Planning — Water planning is an analytical planning process developed and continually modified to address the physical, economic, and sociological dimensions of water use. As a planning process it must assess and quantify the available supply of water resources and the future demands anticipated to be levied upon those resources. Based upon this continuous supply and demand evaluation, water planning must also give direction for moving water supplies to points of use while encouraging users to be good and effective stewards of available water resources. The water planning process requires constant re-evaluation and updating to address changing social, political, economic, and environmental parameters. While the ultimate objective of such efforts is typically the development of a comprehensive, publicly-supported *Water Plan*, it is also critical to develop and maintain a comprehensive and viable water planning process that covers various aspects of water resource development, transport, water treatment, allocation among various competing uses, conservation, waste-water treatment, re-use, and disposal.

Water Right — (1) The legal right to use a specific quantity of water, on a specific time schedule, at a specific place, and for a specific purpose. (2) A legally-protected right, granted by law, to take possession of water occurring in a water supply and to put it to *Beneficial Use*. (3) A legal right to divert state waters for a beneficial purpose.

Water Rights — (1) The legal rights to the use of water. (2) A grant, permit, decree, appropriation, or claim to the use of water for beneficial purposes, and subject to other rights of earlier date or use, called *Priority* or *Prior Appropriation*. They consist of *Riparian Water Rights*, *Appropriative Water Rights*, *Prescribed Water Rights*, and *Reserved Water Rights*.

Water Use — The amount of water needed or used for a variety of purposes including drinking, irrigation, processing of goods, power generation, and other uses. The amount of water used may not equal the amount of water withdrawn due to water transfers or the recirculation or recycling of the same water. For example, a power plant may use the same water a multiple of times but withdraw a significantly different amount.