Dallas Water Master Plan Update

August 2009

CH2MHILL

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Prepared for

City of Dallas, Oregon

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Prepared by

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1. Water System Description

CHAPTER 1

Water System Description

The City of Dallas (Dallas) owns and operates the water system within the Dallas Water System service boundary. Public Works Department staff performs the daily maintenance and operations of the water system.

The Dallas Water System has seven distinct service areas operating at three different service levels (hydraulic grades). Most of the distribution system is in Service Level 1 and is served from the Clay Street Reservoirs and Main Street Reservoir. Five closed-loop booster pump stations in the system serve developed areas at elevations above Service Level 1. Five of these developed areas are in Service Level 2 and one developed area is in Service Level 3. The Service Level 2 areas are served by the Orchard Drive, Church Street, Maple Street, and Elmwood Drive pump stations. The Service Level 3 area is served by the Upper Bridlewood Pump Station and works off of the Bridlewood Reservoir. The Douglas Street area is served by the high-pressure line from the 2-million-gallon (MG) reservoir at the water treatment plant (WTP).

Figure 1-1 shows the existing water system, city limits, and urban growth boundary (UGB). This master plan study includes an evaluation of the Dallas Water System.

Storage Reservoirs

Dallas has finished water reservoir facilities in four locations. The first of these facilities is a steel tank located at the WTP that holds approximately 2 MG. This reservoir serves as a wet well for the WTP. Finished water from the WTP is pumped to the tank and then supplied as needed by gravity into the Clay Street Reservoirs and Main Street Reservoir, from which the distribution system is served, again by gravity. The 2 MG WTP reservoir also supplies by gravity an 18-inch pipeline along West Ellendale Avenue to a pressure- reducing valve (PRV) near James Howe Road. Flow passes through both the WTP and Clay Street Reservoirs or Main Street Reservoir under normal conditions; however, when the PRV at West Ellendale Avenue and James Howe Road opens during periods of high demand, some water is supplied from the WTP reservoir directly to the distribution system. Prior to the PRV located on West Ellendale, an 8-inch high- pressure line branches off and supplies an area along Douglas Avenue via another PRV. The Clay Street location has two open circular concrete basins that were constructed in the 1930s and two open rectangular concrete basins that were constructed in 1954. Together, the four basins at the Clay Street Reservoirs contain 4 MG of storage. A 2 MG steel reservoir was constructed in 2008-2009 at the south end of Main Street. The Main Street Reservoir was identified as a project in the 2002 Water Master Plan. The fourth reservoir, located at Bridlewood, is a single steel tank with a storage volume of 0.135 MG. This tank is used to supply normal demands as well as emergency and fire protection storage in the Service Level 3 area serviced by the Upper Bridlewood Pump Station and the Service Level 2 area serviced by the Elmwood Drive Pump Station. Dallas has a total storage capacity of 8.135 MG. An evaluation of current and future storage capacity requirements is presented in Chapter 5. Table 1-1 presents a summary of the storage reservoirs in current operation.

Pump Stations

Dallas has a total of five booster pump stations, excluding the pumps located at the WTP. Table 1-2 provides a summary of the existing pump stations and equipment. The Orchard Drive, Maple Street, and Church Street pump stations are all closed-end pump stations that pump water from Service Level 1 to Service Level 2. The Elmwood Drive (Lower Bridlewood) Pump Station delivers water through the distribution system to the Bridlewood Reservoir in Service Level 2. The Upper Bridlewood Pump Station serves the Service Level 3 area by pumping from the Bridlewood Reservoir and using a hydropneumatic tank. The Orchard Drive and Upper Bridlewood pump stations each contain one smaller jockey pump for serving regular demands and one high-capacity booster pump to supply fire protection in the closed-end systems. The Maple Street and Elmwood Drive pump stations each have two identical-capacity pumps that alternate each time they are started. In addition to these two pumps, the Maple Street Pump Station has a third high-capacity fire flow pump to provide fire protection for the Oakdale School. The Church Street Pump Station contains a single jockey pump that runs continuously, serving approximately 30 homes both inside and outside the city limits. Table 1-2 presents a summary of the pump stations in current operation.

Pipeline

Dallas has approximately 61 miles of pipelines in its water transmission and distribution system. The system is predominantly looped and located within public rights-of-way, giving the city access for repairs and maintenance. The pipeline system has been upgraded and expanded annually to serve the city's growing demands. A large portion of the distribution system consists of 4- to 12-inch-diameter ductile iron (DI) and cast iron (CI) pipe. Additionally, a 16-inch-diameter steel pipe exists between the WTP reservoir and the Clay Street Reservoirs, and an 18-inch-diameter DI pipe between the WTP reservoir and the PRV on West Ellendale Avenue. A 14-inch-diameter DI and high-density polyethylene (HDPE) pipe feeds the Main Street Reservoir from a connection point at 14^{th} Street. The connection point at 14^{th} Street is fed by a 16-inch DI pipe from the Clay Street Reservoirs. A 16-inch-diameter concrete cylinder pipe serving as a supply main runs from the Clay Street Reservoirs to the east along Clay Street to Main Street.

Telemetry System

The Dallas Water System is monitored and controlled by a central telemetry system at the WTP. All telemetry system readings are available at the WTP only. The telemetry system allows water system operators to monitor and control WTP operations and the flow of finished water into the city's distribution system. In addition, reservoir levels at the WTP, Main Street Reservoir, and Clay Street Reservoirs can be monitored through the telemetry system. The ability to provide real-time system data and control from the telemetry system allows Dallas Water System operators flexible control of the WTP and distribution system.

TABLE 1-1
Existing Storage Reservoirs

Reservoir	Service Level	Volume (MG)	Elevation Overflow (ft)	Elevation Base (ft)	Diameter (ft)	Type	Year Built	Comments
Water Treatment Plant	Levels 1 and 2	2	631	600.5	106	Steel	1993	Good functional condition. External level indicator does not operate correctly at all times.
Clay Street Reservoirs	Levels 1 and 2	4	505.75	488.58	Circular–88 and 92 Rectangular–(2) 110 x 115	Concrete	1930s–Circular 1954–Rectangular	Good functional condition. The two rectangular reservoirs contain 3 MG total, and the two circular reservoirs contain 1 MG total. Each reservoir has floating covers installed in 2003
Main Street Reservoir	Levels 1 and 2	2	505.75	489.58	145	Steel	2008	Recently constructed.
Bridlewood Reservoir	Levels 2 and 3	0.135	637.5	614	32	Steel	1977	Good functional condition. There have been problems maintaining chlorine residual in the winter at this reservoir. Fill and discharge occur through the same pipe.
	Total Volume =	**8.135**						

MG = million gallons.

TABLE 1-2
Existing Pumping Facilities

Pump Station	Suction	Discharge	Pumps[1]	Year Built or Latest Rehabilitation	Comments[2]
Orchard Drive	Level 1	Level 2 (Closed-end pumping system)	1 – 20 hp -50 to 350 gpm 1 – 40 hp - 800 gpm	1999	Good functional condition. The below grade pump station was rebuilt in 1999. The station piping and valves were replaced. A variable frequency drive (VFD) was added for the jockey pump and a soft start for the large pump in 1999. Since then, a VFD was added for the fire pump. Service area enlarged and borders Douglas service area.
Maple Street	Level 1	Level 2 (Closed-end pumping system)	2 – 7.5 hp - 180 gpm 1 – 100 hp - 2300 gpm	1976	Good functional condition. During fire flow the pressure sustaining valve (PSV) causes the discharge pressure to fluctuate between 70 to 110 psi. The below grade pump station has a portable power generator connection.
Elmwood Drive (Lower Bridlewood)	Level 1	Level 2 - Bridlewood Reservoir	2 – 15 hp - 195 gpm	2009	Good functional condition. Above grade pump station houses two 15-hp pumps installed in 2009.
Church Street	Level 1	Level 2 (Closed-end pumping system)	1 – 7.5 hp - 240 gpm	Late 1970s	Good functional condition. The above grade pump station runs continuously and feeds 20 homes outside the city limits, in addition to 10 to 12 homes in the city. A 5-hp motor is kept at the city shops and is used as a backup. The pump station has a portable power generator connection.
Upper Bridlewood	Level 2	Level 3 (Closed-end pumping system)	1 – 5 hp - 150 gpm 1 – 20 hp - 850 gpm	1977	Good functional condition. The above grade pump station includes a hydropneumatic tank and a portable power generator connection.

[1] Pump curves were used to estimate pump capacity. A pump capacity flow range was estimated for pumps where the design point was not specified.

[2] Portable power generation connections are provided at four pump stations. Dallas owns and operates several portable power generators.

hp = horsepower

gpm = gallons per minute

psi = pounds per square inch

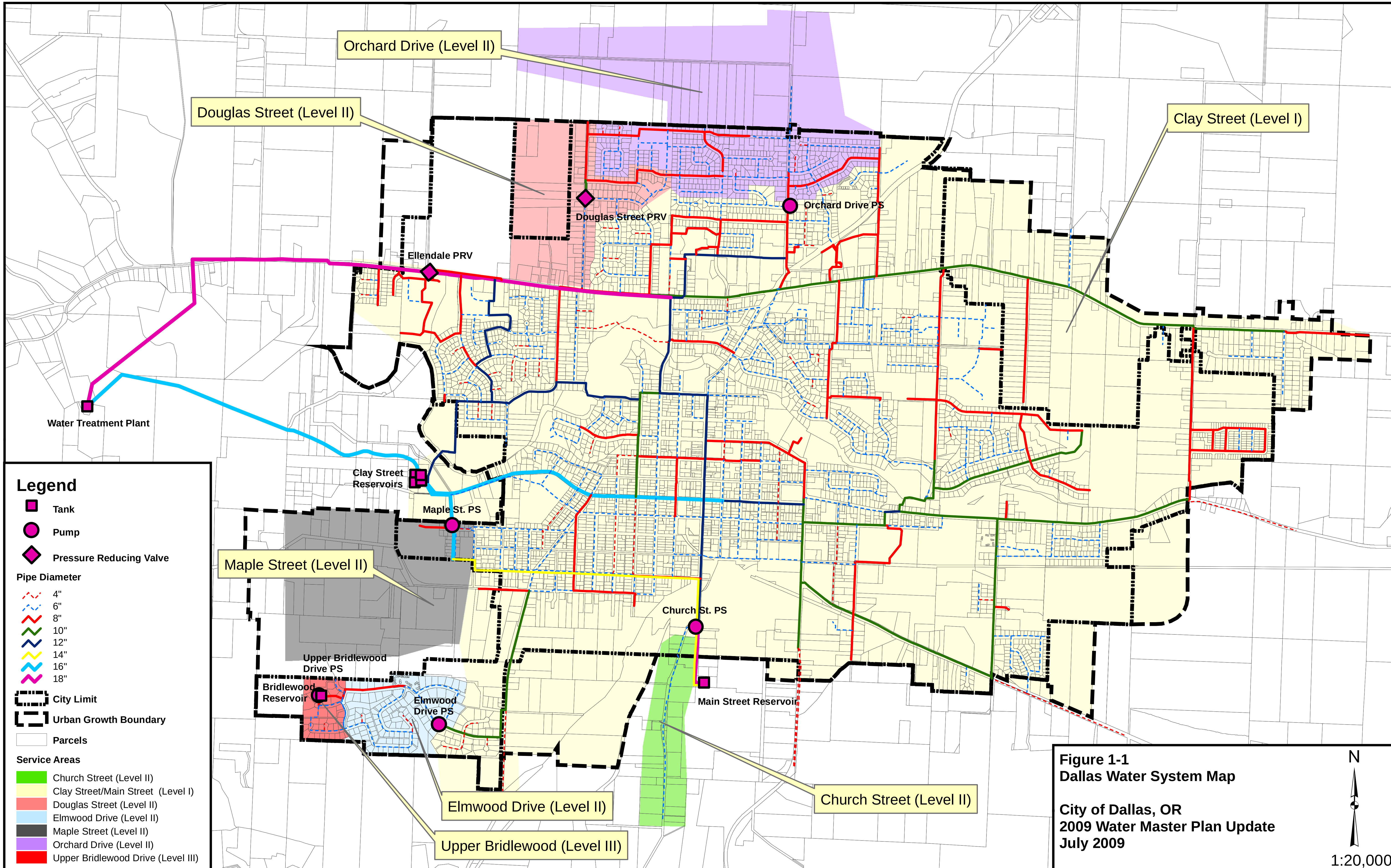


Figure 1-1
Dallas Water System Map

City of Dallas, OR
2009 Water Master Plan Update
July 2009

2. Water Requirements

CHAPTER 2

Water Requirements

This chapter discusses the planning data used in developing the population and water demand projections for the 2002 Dallas Water Master Plan and this 2008 Water Master Plan Update. Information from the Regional Water Supply and Transmission Expansion Project (RWSTEP) report completed for the Cities of Dallas, Monmouth, and Independence and the agricultural communities of Polk County, was also used in the demand projections.

Definition of Terms

The following definitions are used in the master plan:

Demand: The total quantity of water supplied for a given period of time to meet the various required uses. The various uses are residential, commercial, and industrial as well as fire fighting, system losses, other unaccounted-for and miscellaneous uses.

Unaccounted-for water: The difference between the total amount of water produced by the WTP and the total amount of water billed to customers.

The different levels of water demand used in this analysis are designated as average day demand (ADD), maximum day demand (MDD), and peak hour demand (PHD).

Average day demand: The total volume of water delivered to the system in 1 year, divided by 365 days.

Maximum day demand: The maximum volume of water delivered to the system in any single day of the year.

Peak hour demand: The maximum volume of water delivered to the system in any single hour of the year.

Historical Water Use Information

The historical water use information, current populations, and future population projections form the basis for projecting future water demands. Historical use and current population data are used to estimate per capita usage rates and peaking factors related to usage, and then these values are used with population projections to estimate future water use. Historical demands were obtained from Dallas water production data records. Current service population data were obtained from the Population Research Center and RWSP-2 reports. Table 2-1 summarizes population and historical water production data from 1996 to 2008.

Population Projections

As part of the 2002 Water Master Plan, population projections for Dallas were obtained through the Population Research Center at Portland State University (PSU) from a report prepared in October 2001 titled *Population Projections for Dallas City, Independence City, Monmouth City, and Unincorporated Areas of Polk County, Oregon: 2000 to 2100*. The projections are benchmarked to the 2000 census. The forecasts are based on an analysis of historical trends and expectations of the future, and do not assume any drastic changes to the population trends that have developed during the past three decades. Although past trends give an indication of what is likely to happen in the future, there always exists the possibility of unforeseen events that could have a significant impact on these projections.

Within the report from the Population Research Center, the ratio method was used to project population. This method projects population assuming the same influences of change for the city as the surrounding county population, and was used to project populations from 2000 to 2100 in 5-year intervals. Additionally, a range of “high,” “medium,” and “low” population forecasts was developed for the same time period. To be consistent with the RWSTEP report, the “high” population forecast was used. Projections for the years falling between the 5-year interval years were based on linear interpolations. Additional information regarding these population projections can be found in the Population Research Center report in Appendix A.

In addition to the population within the city limits, the Dallas Water System serves customers outside of the city. Consistent with the RWSTEP report, estimated populations outside of the city limits served with city water were taken from the Regional Water Supply Plan—Phase 2 (RWSP-2) report.

As part of the 2008 Water Master Plan Update, the flow projections were extended to 2028 using the same methodology as the 2002 Water Master Plan. The population projections presented in the *Population Projections for Dallas City, Independence City, Monmouth City, and Unincorporated Areas of Polk County, Oregon: 2000 to 2100* estimated a 2010 population of 15,175. The certified population presented by PSU’s Population Research Center estimated the actual population to be 15,360. Because the actual 2008 population was greater than predicted, the population projections after 2008 were modified by assuming the 2015 population projection is still valid at 16,616. The population projections between 2008 and 2015 were assumed to increase from the actual population of 15,360 in 2008 to the estimated population of 16,616 in 2015. A linear growth was assumed to occur between 2008 and 2015. Table 2-1 summarizes updated population and demand projections and Figure 2-1 illustrates population projections for the Dallas Water System.

TABLE 2-1
City of Dallas Population and System Demand Projections

Year	Dallas Water System Service Area Population			Total System Demands				
	Population [1] Within City Limits	Population [2] Outside City Limits	Total Service Area Population	ADD [3, 4] (mgd)	ADD (gpcpd)	MDD [3, 4] (mgd)	Peaking Factor [4] MDD/ADD	PHD [5] (mgd)
1996	11,154	832	11,986	2.68	224	6.30	2.35	--
1997	11,467	842	12,309	2.74	223	6.15	2.24	--
1998	11,789	852	12,641	3.02	239	6.39	2.12	--
1999	12,119	863	12,982	2.75	212	5.14	1.87	--
2000	12,459	873	13,332	2.6	195	4.86	1.87	--
2001	12,733	883	13,616	2.28	167	4.74	2.08	--
2002	12,850	893	13,743	2.23	162	5.67	2.54	--
2003	13,270	904	14,174	2.32	164	5.31	2.29	--
2004	13,500	914	14,414	2.31	160	4.96	2.15	--
2005	14,040	924	14,964	2.07	138	4.85	2.34	--
2006	14,585	935	15,520	2.42	156	5.57	2.30	--
2007	15,065	945	16,010	2.32	145	5.73	2.47	--
2008	15,360	956	16,316	2.73	167	5.73	2.30	--
2009	15,539	966	16,505	2.60	157	5.98	2.30	8.97
2010	15,719	977	16,696	2.63	157	6.05	2.30	9.07
2011	15,898	988	16,886	2.66	157	6.12	2.30	9.17
2012	16,078	999	17,077	2.69	157	6.18	2.30	9.28
2013	16,257	1,011	17,268	2.72	157	6.25	2.30	9.38
2014	16,437	1,022	17,459	2.75	157	6.32	2.30	9.48
2015	16,616	1,033	17,649	2.78	157	6.39	2.30	9.59
2016	16,916	1,045	17,961	2.83	157	6.50	2.30	9.76
2017	17,216	1,057	18,273	2.88	157	6.62	2.30	9.93
2018	17,515	1,068	18,583	2.93	157	6.73	2.30	10.10
2019	17,815	1,080	18,895	2.98	157	6.84	2.30	10.26
2020	18,115	1,092	19,207	3.02	157	6.96	2.30	10.43
2021	18,421	1,105	19,526	3.07	157	7.07	2.30	10.61
2022	18,727	1,119	19,846	3.12	157	7.19	2.30	10.78
2023	19,033	1,132	20,165	3.18	157	7.30	2.30	10.95
2024	19,339	1,146	20,485	3.23	157	7.42	2.30	11.13
2025	19,645	1,159	20,804	3.28	157	7.53	2.30	11.30
2026	19,951	1,173	21,124	3.33	157	7.65	2.30	11.48
2027	20,256	1,187	21,444	3.38	157	7.77	2.30	11.65
2028	20,562	1,202	21,763	3.43	157	7.88	2.30	11.82

[1] 1996 to 1999 estimated from 1995 in RWSP-2 report and 2000 Census results; 2000 to 2020 based on “Population Projections for Dallas City, Independence City, Monmouth City, and Unincorporated Areas of Polk County, Oregon: 2000 to 2100” prepared by Barry Edmonston, Portland State University, Population Research Center. Benchmarked to 2000 census results, ‘High Growth Assumption.’

[2] Estimated population outside of the city limits but served with city water (from the RWSP-2 report).

[3] 1996 through 2008 ADD and MDD data are based on raw water pumping records from the City of Dallas WTP.

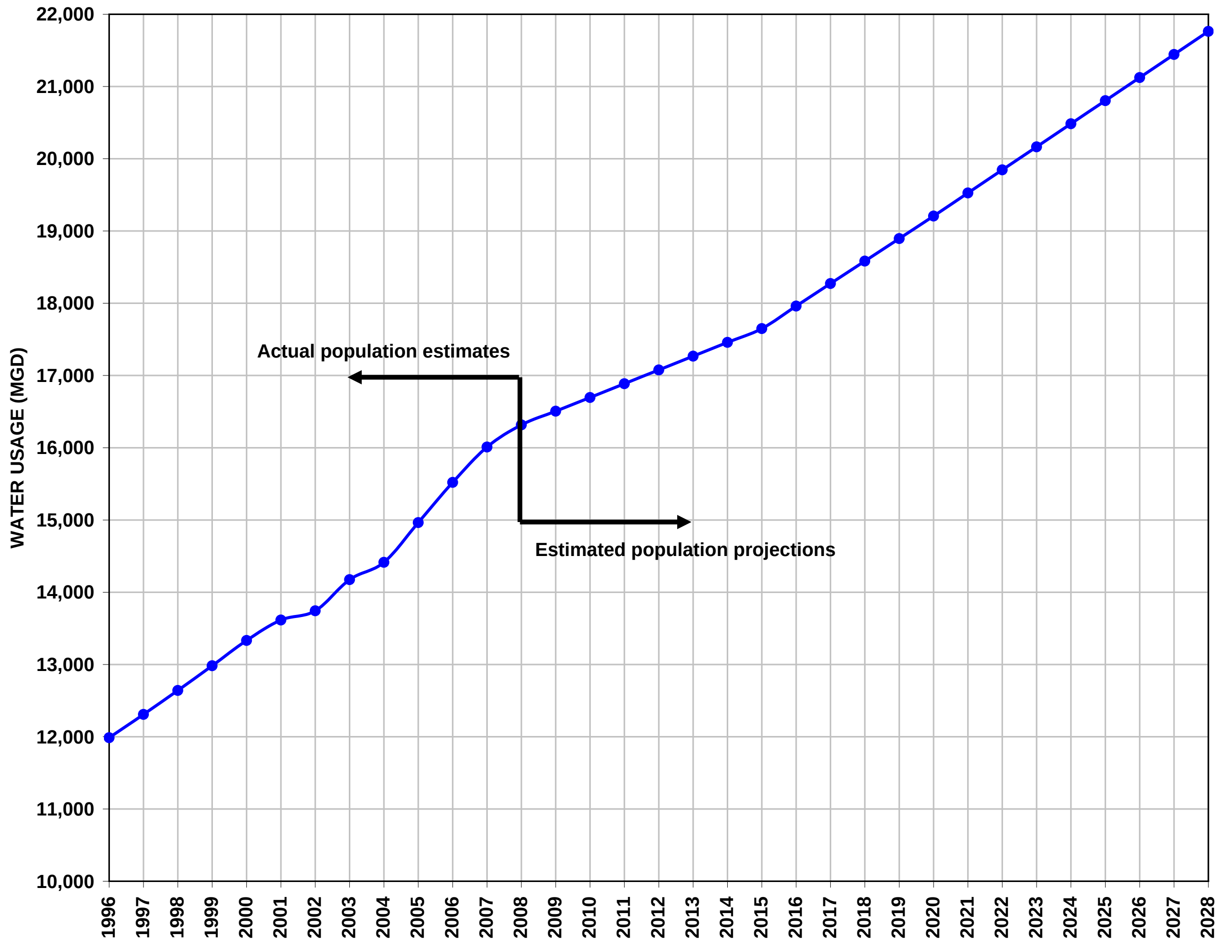
[4] Future projections assume an MDD/ADD peaking factor of 2.3 based on the past 6 years data.

[5] PHD is assumed to be 1.5 times the MDD.

Note: The line separating 2008 and 2009 represents where population and demand projections commence.

ADD = average day demand MDD = maximum day demand PHD = peak hour demand mgd = million gallons per day gpcpd = gallons per capita per day

FIGURE 2-1
City of Dallas Water System Population History and Projections



Water Demand Projections

Future water demand was projected based on the estimated per capita use and future population projections. It is assumed that the rate of increase in water use for industrial and other commercial users will follow a pattern similar to the residential population. This assumption provides a projection of future water needs in Dallas based on best information available and without knowledge of the elimination or addition of specific large industrial users. Therefore, projections for all future water use will be based on the rate of increase of the permanent residential population. Because the overall per capita consumption value includes water for industrial and commercial uses, it will appear to be high when compared to national averages of approximately 100 to 150 gallons per capita per day (gpcpd) for residential use only.

The average overall per capita water production from 1996 to 2008 was approximately 181 gpcpd. The per capita usage ranged between 138 and 239. Figure 2-2 shows the per capita usage from 1996 to 2008. Fluctuations in the per capita usage can be attributed to factors such as closure or reduction of existing users (for example, Willamette Industries, Tyco), weather influence on irrigation demand, water conservation, accuracy of population estimates, etc.

A sharp decrease occurs after 1998 as a result of the reduced usage by Willamette Industries. After this sharp decrease, the per capita usage is relatively flat. For the purpose of the flow projections, the future per capita usage was estimated to equal the average of per capita usage from 2001 to 2008. Existing commercial and industrial users are captured in the per capita usage values discussed above. It is assumed that the ratio of commercial and industrial use to residential use will remain similar in the future. Therefore, for planning purposes, increases in commercial and industrial use are captured by applying the overall per capita usage rates to increased population.

Conservation is an important component of demand projection, but the quantitative effects of conservation are sometimes difficult to predict. Public education and conservation programs must be implemented consistently to realize the benefits of conservation. Therefore, conservation was not accounted for in the flow projections.

The Dallas Water System ADD and MDD projections are summarized in Table 2-1. Figure 2-3 illustrates the demand projections.

Peaking Factor

The relationships between the ADD and other flow rate demands in the system are referred to as peaking factors. The average MDD/ADD peaking factor for 1996 through 2008 was 2.22. The highest MDD/ADD peaking factor of 2.54 occurred in 2002. An MDD/ADD peaking factor of 2.30 was used to project the MDD presented in Table 2-1. Using this conservative value in this study ensures that system improvements based on the results of this study will be designed to meet the highest potential MDD. No PHD data were available for estimating PHD/MDD peaking factors; therefore, a typical value of 1.5 was assumed for this study.

FIGURE 2-2
City of Dallas Water System Historical Per Capita Water Usage

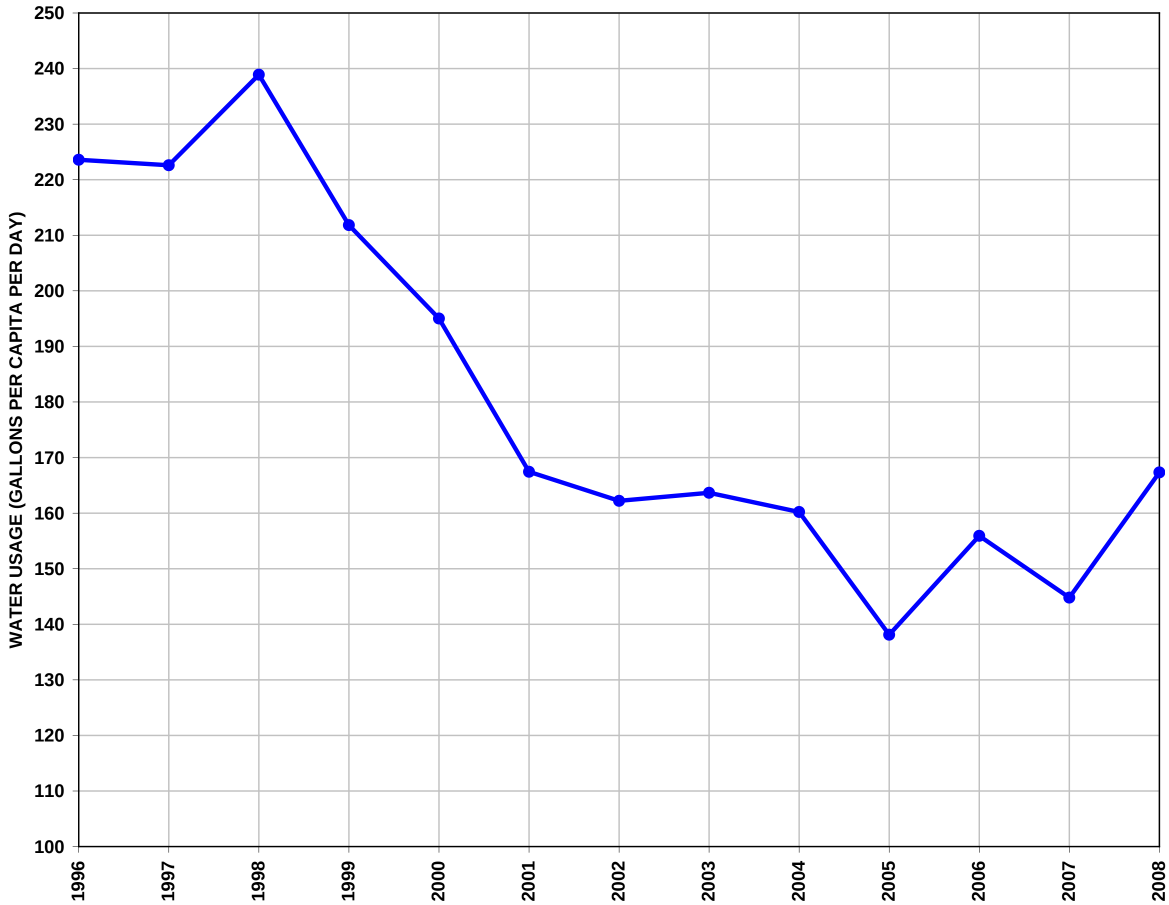
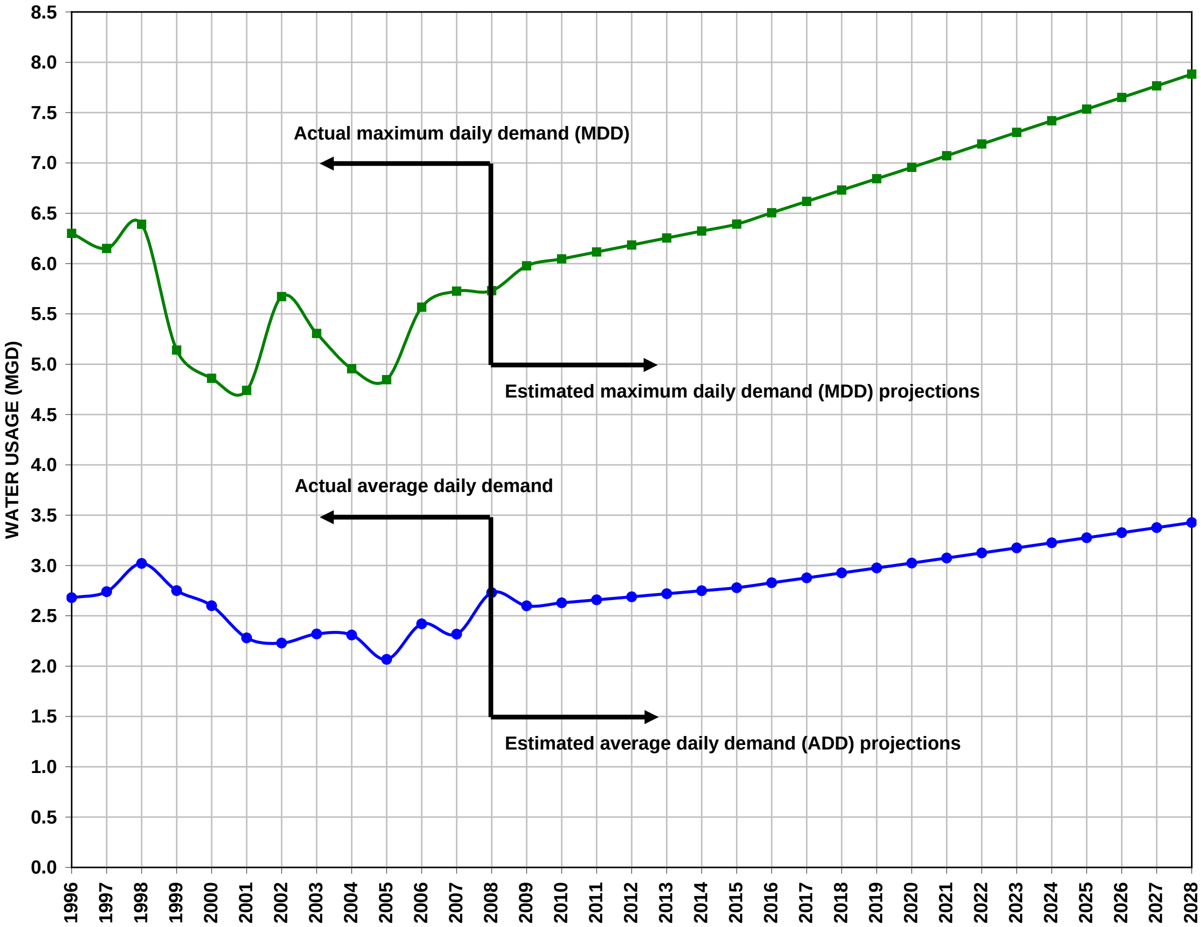


FIGURE 2-3
City of Dallas Water System Demand History and Projections



Unaccounted-for Water

Unaccounted-for water in the Dallas Water System is the difference between the total amount of water produced by the WTP and the total amount of water billed to customers. Unaccounted-for water from the Dallas Water System is water resulting from leakage losses, meter discrepancies, hydrant and main flushing, street sweeping, operation and maintenance uses, fire flow uses, unauthorized connections, and unmetered miscellaneous uses. Water used from hydrant connections for construction site activities is metered and billed, but is not included in the billing records and, therefore, is part of the unaccounted-for water reported in this analysis. In addition, water is sold to users from outside the system at the city shop in quantities ranging from several hundred to several thousand gallons at a time. This water also is not included in the billing records and is a part of the unaccounted-for water quantities estimated.

The average unaccounted-for water in the Dallas Water System has varied between 5.7 and 12.8 percent, with an average of 8.3 percent, between 1997 to 2001. Table 2-2 lists the historical unaccounted-for water for the system. Unaccounted-for water has not been tracked since 2001 because of the inaccuracy of the WTP effluent flow meter at low flow. The effluent flow meter at the WTP needs to be upgraded so the city can trend unaccounted-for water to measure conservation efforts. Conservation methods have been effective in reducing the unaccounted-for water as shown in Table 2-2. Included in the unaccounted-for water are routine monthly fire department fire hydrant flows for training, maintenance flushing of the distribution system, and other unmetered hydrant use by the state, county, and development contractors.

Fluctuation in unaccounted-for water also is affected by hydrant and main flushing. In a year when there are aggressive maintenance activities, unaccounted-for water would be expected to increase. Dallas should continue its successful water system maintenance program and attempt to meter all users to keep reducing the quantity of unaccounted-for water from the distribution system.

TABLE 2-2
Unaccounted-for Water

Year	Percentage
1997	12.8
1998	8.7
1999	5.9
2000	5.7
2001	7.1
Average	**8.0**

Note: Unaccounted-for water has decreased as distribution system improvements and other supply-side conservation methods have been implemented. Differences from year to year may not be accurate because of the timing of end-of-year customer meter readings.

3. Water Supply

CHAPTER 3

Water Supply

This chapter evaluates the Rickreall Creek watershed's ability to meet present and future city water needs.

Existing Water Supply System

The Rickreall (originally LaCreole) Creek Watershed, about 3.5 miles west of Dallas, supplies water for the Dallas Water System. The system has evolved from intakes on Rockhouse Creek, Applegate Creek, and Canyon Creek (tributaries to Rickreall Creek) in 1919, to the present dual intake system. In addition, water is stored behind an earthen dam about 4.5 miles upstream from the intake. Water is released from the dam when the natural stream flow is inadequate to meet the demand for water.

The dam was constructed in 1959 to store 760 acre-feet (247 MG) of water. In 1972 the dam was raised to provide a total raw storage of 1,550 acre-feet (505 MG). Field survey data gathered in 1998 indicate that sedimentation in the impoundment had decreased total raw storage capacity to approximately 1,050 acre-feet (342 MG). Construction of flashboards completed in April 2001 added 215 acre-feet (70 MG) of spring/summer storage. Currently, the intake pumps are capable of delivering about 8.6 million gallons per day (mgd) to the WTP.

Dallas has water rights totaling 3.45 mgd [5.33 cubic feet per second (cfs)] from stream flow and 12.92 mgd (20 cfs) from storage. Table 3-1 summarizes Dallas's current water right permits and certificates in the Rickreall Creek drainage basin.

TABLE 3-1
Existing Water Rights [1]

Priority Date	Permit	Certificate No.	Flow (cfs)	Flow (mgd)	Point of Diversion
Stream					
01/22/1919	4053	68474	1.00	0.65	Applegate Creek at Section 4, T8S, R6W, W.M. [2]
			3.00	1.94	Rockhouse/Rickreall confluence at Section 1, T8S, R7W, W.M. [2]
11/22/1967	33202	39181	0.06	0.04	At the Reservoir at NE1/2 SW1/4, Section 6, T8S, R6W; Rickreall Creek at SW 1/4 SE 1/4, Section 35, T7S, R6W; Reservoir 1,804.6 feet north and 2,405.3 feet east from SW corner, Section 6, T8S, R6W
03/23/1903	CD 38630	--	0.77	0.50	Canyon Creek at WTP in SW 1/4 SE 1/4, Section 35, T7S, R6W, W.M.; 870 feet north and 2,100 feet west from the SE corner, Section 35.
12/31/1909	38631	38631	0.50	0.32	In SW 1/4 SE 1/4, Section 35, T7S, R6W, W.M.; 1,880 feet south and 2,030 feet west from east 1/4 corner, Section 35.
		Total stream rights =	5.33	3.45	
Reservoir					
01/28/1958	R2283	--	--	--	1,200 acre-feet of storage on Rickreall Creek
01/28/1958	26397	--	10.00	6.46	Use of 1,200 acre-feet at up to 10 cfs
09/16/1971	R5755	--	--	--	790 acre-feet storage increase to existing 760 acre-feet = 1,550 acre-feet total
9/16/1971[3]	35718[3]	--	10.00	6.46	From reservoir storage 790 acre-feet R5755
		Permitted rate from storage =	20.00	12.92	

[1] Based on review of Oregon Water Resources Department (OWRD) files in fall 2001.

[2] Collected at new intake located on Rickreall Creek in SW 1/4 SE 1/4, Section 35, T7S, R6W, W.M.; 870 feet north and 2,100 feet west from the southeast corner of Section 35.

[3] This right is currently not certificated. Because this permit does not clearly specify a diversion rate, various diversion rates could be interpreted by others. Conversations with Polk County watermaster (personal communication, February 20, 2002) indicate that OWRD interprets the total maximum diversion rate from Mercer Reservoir to be 20 cfs, consisting of 10 cfs from this right and 10 cfs from 26397.

cfs = cubic feet per second

mgd = million gallons per day

Stream Hydrology

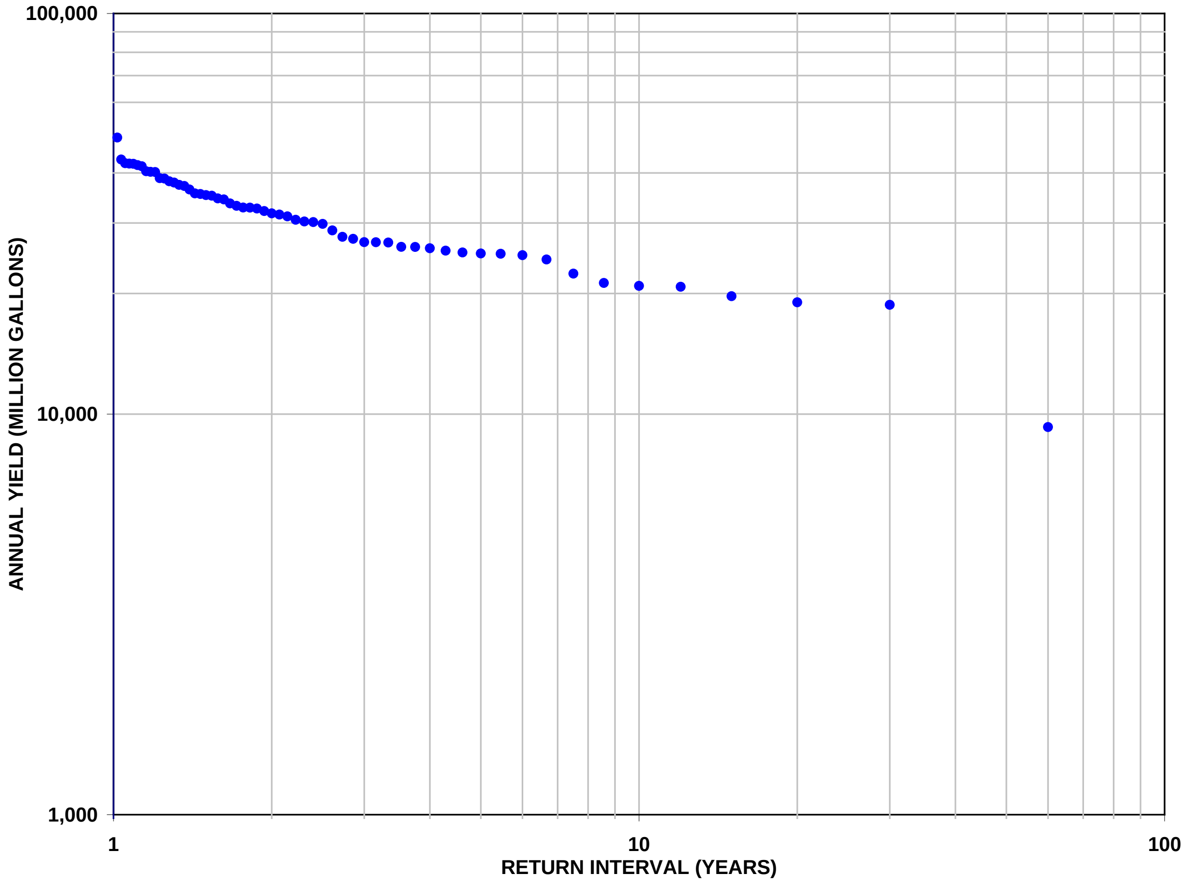
An analysis of the Rickreall watershed hydrology was made to determine the average basin yield, possible variations from this average, probability of low yields, and how the low yields compare to present and projected storage needs.

Yield

A gage (No. 14190700) was maintained near the present intake for 21 years, from August 1957 to September 1978. The records for this gage were artificially extended by comparing the records with other gages in the general vicinity over the same time period. The gage on the South Yamill River near Willamina (No. 14192500) was found to have a good correlation. The South Yamhill gage data allowed development of a 60-year stream flow record for Rickreall Creek (1934-1994). The 60-year constructed record was used to develop the frequency versus annual average stream flow estimates shown on Figure 3-1. As this figure shows, the annual stream yield is significantly larger than the reservoir storage volume (100 year drought yield = approximately 18,000 MG compared to a reservoir volume of 575 MG with flashboards).

It is estimated that 70 to 75 percent of the runoff occurs above the dam and is available for storage. Therefore, even in the 100-year drought condition the yield (and the potential for additional storage) is about 25 to 30 times the present reservoir volume. However, the stream flow patterns are highly seasonal. Summer and early fall stream flow alone is too low to meet the city’s needs.

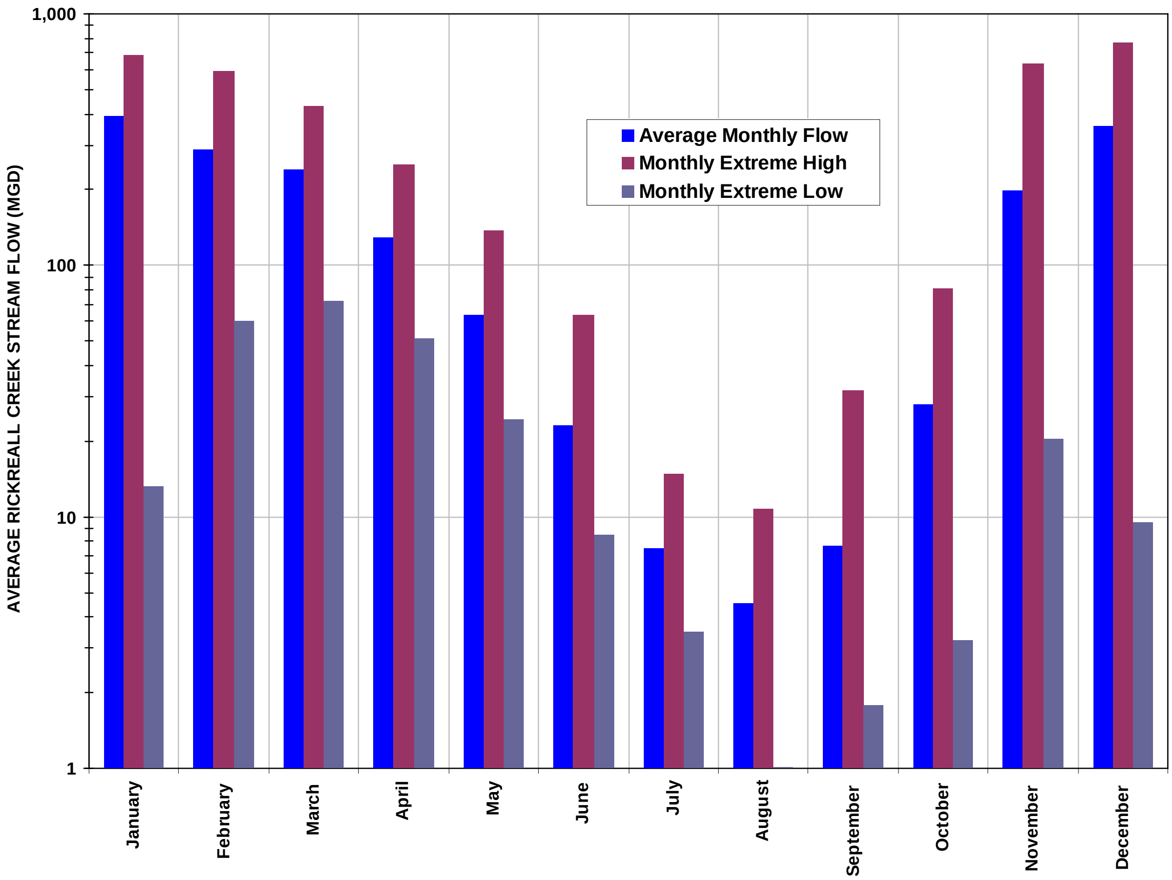
FIGURE 3-1
Rickreall Creek Yield Probability



Annual Distribution of Yield

The monthly average and extremes for the Rickreall Creek gage period of record are shown on Figure 3-2. The 100-year drought yield is estimated to be equal to the 1966-1967 water year. The 1966-1967 water year has been used as representative of the annual distribution that the 100-year drought might have. The distribution is characterized as having an early and late dry period during the dry season between July and October. Although lower flows have been recorded for single months in other years, the combination of low flow summer months experienced in the 1966-1967 water year constitutes the critical period of record for water supply planning. A 100-year drought has an 18 percent probability of occurring in any 20-year period.

FIGURE 3-2
Watershed Monthly Yields (based on USGS Gage No. 14190700)



Stream Flow Versus Demand

As the city's water needs have grown, the deficit that must be made up from stored water has increased. The relationship between runoff and demand is shown on Figure 3-3 and Figure 3-4 for the 100-year drought planning condition for 2007 and 2028 demands. The runoff deficit is indicated by the shaded area. This area between the unregulated stream flow and the demand curve represents the water that must be stored or supplemented by another source [for example, aquifer storage and recovery (ASR)] to meet demands during the low stream flow period.

FIGURE 3-3
Raw Water Storage Requirements (2007 Demands)

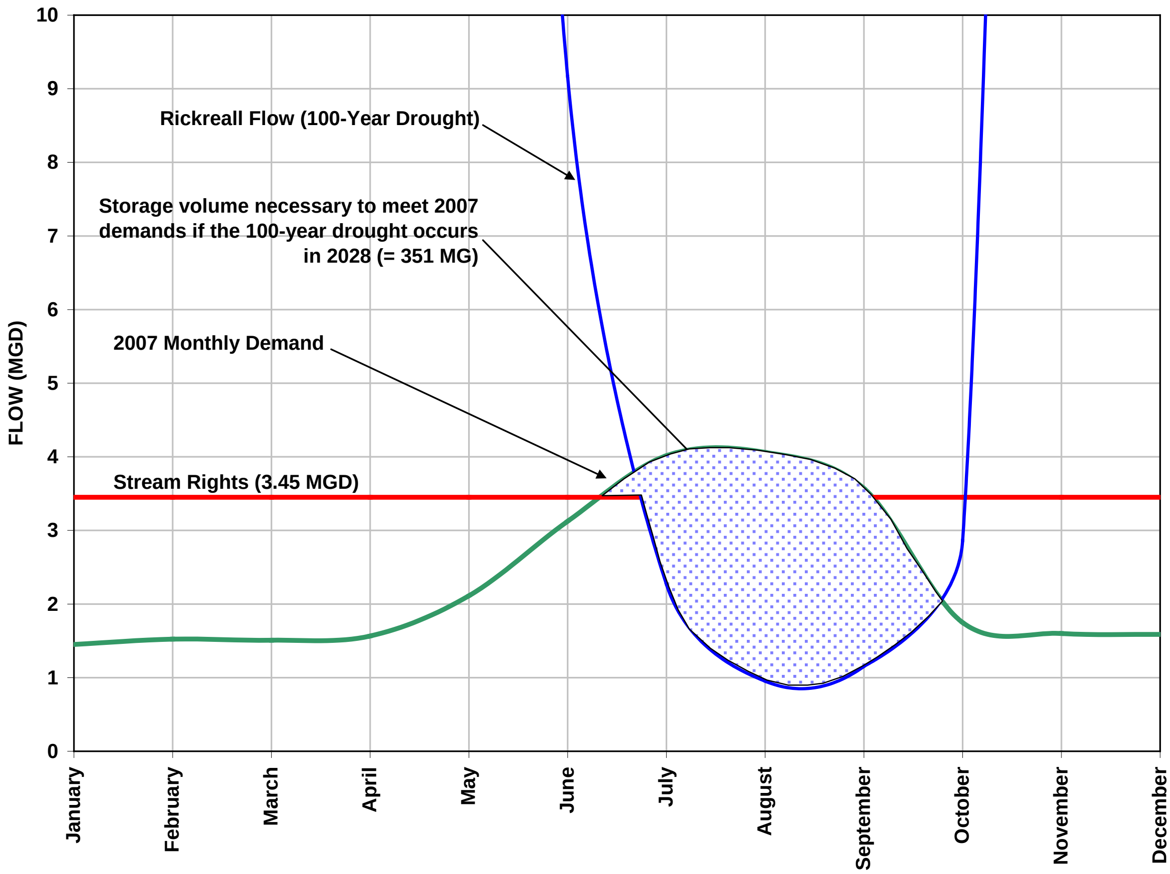
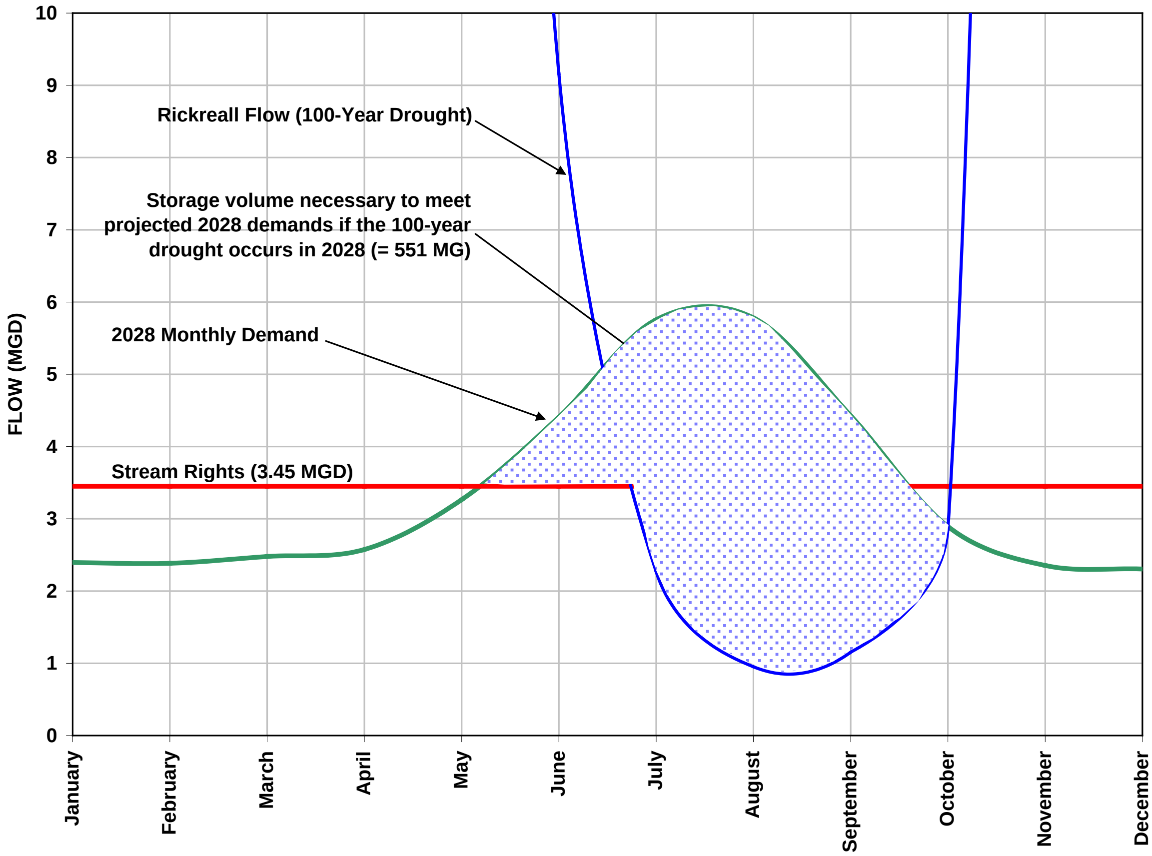


FIGURE 3-4
Raw Water Storage Requirements (2028 Demands)



Projections

The estimated future storage requirements to meet the 100-year drought needs are shown on Figures 3-5 and 3-6. Future requirements based on 100 percent capture of reservoir releases and the assumed present 70 percent capture is shown, including the impact of ASR, wastewater reuse, and conservation. Based on information provided by Golder and Associates, the existing ASR well can extract between 275 and 300 gallons per minute (gpm) with target storage of 50 MG and 80 percent recovery. For planning purposes, the ASR well is assumed to be capable of producing 275 gpm [400,000 gallons per day (gpd)] over a 3-month dry season. At this rate of extraction, the total volume recovered is approximately 36 MG. Figure 3-6 shows the storage requirements assuming a second ASR well is constructed with similar production rates as the current ASR well. The impact of wastewater reuse and conservation in addition to two ASR wells is also shown on Figure 3-6. Again, for planning purposes, it was assumed that the combined effect of wastewater reuse and conservation would reduce storage requirements at a rate of 0.50 mgd.

Although the need for storage is projected to grow, the storage provided by the present dam and reservoir is being reduced with time because of deposition of eroded soils carried into the reservoir by the contributing streams. The average rate of deposition is not known, but has been estimated to be between 5 and 12 acre-feet per year. For this evaluation, it has been assumed that 10 acre-feet per year is being reduced (from *An Engineering Study of the Water Supply for the City of Dallas, Oregon*, CH2M HILL, 1989). Annual deposits would vary considerably and would be greatest during years with large storms. Logging, road construction, and fires also create a greater potential for erosion and siltation of the reservoir.

As shown on Figure 3-5, the present storage appears to be adequate through the year 2015 when compared to the 100-year drought and with ASR. The need for additional storage is pushed out further when a second ASR well, wastewater reuse, and conservation are considered (see Figure 3-6); however, the need for additional storage also depends on the success of implementing ASR, wastewater reuse, and conservation. Because of the uncertainty of success, it is prudent to identify and obtain additional water supply and water rights. Considerations such as a second source or an emergency supply, if the primary source is made unusable, should be addressed. Even with long-range planning, meeting the environmental requirements, obtaining permits, and accomplishing the design and construction can take up to 10 years; perhaps longer, if the project is controversial (for example, a new dam).

FIGURE 3-5
Estimated Future Storage Requirements with One ASR Well (based on 2028 demands)

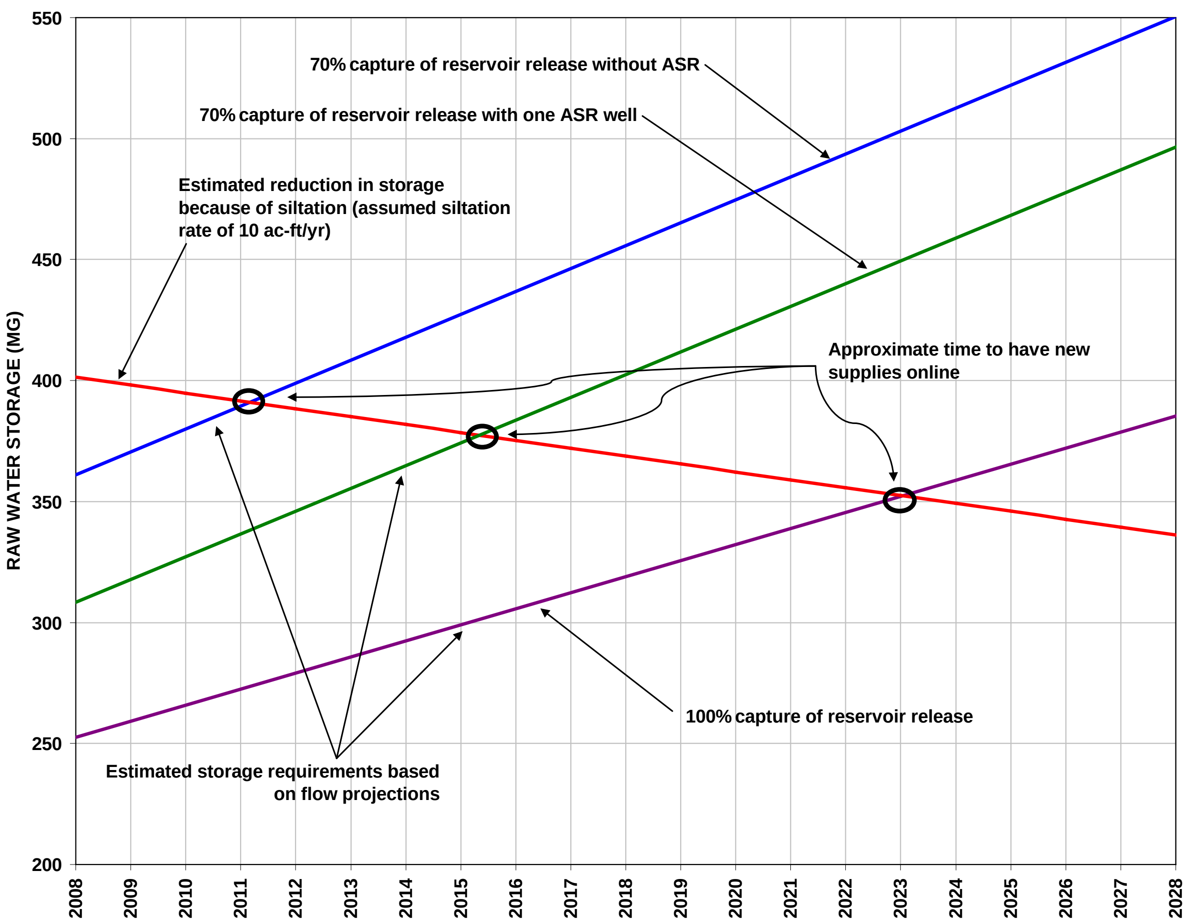
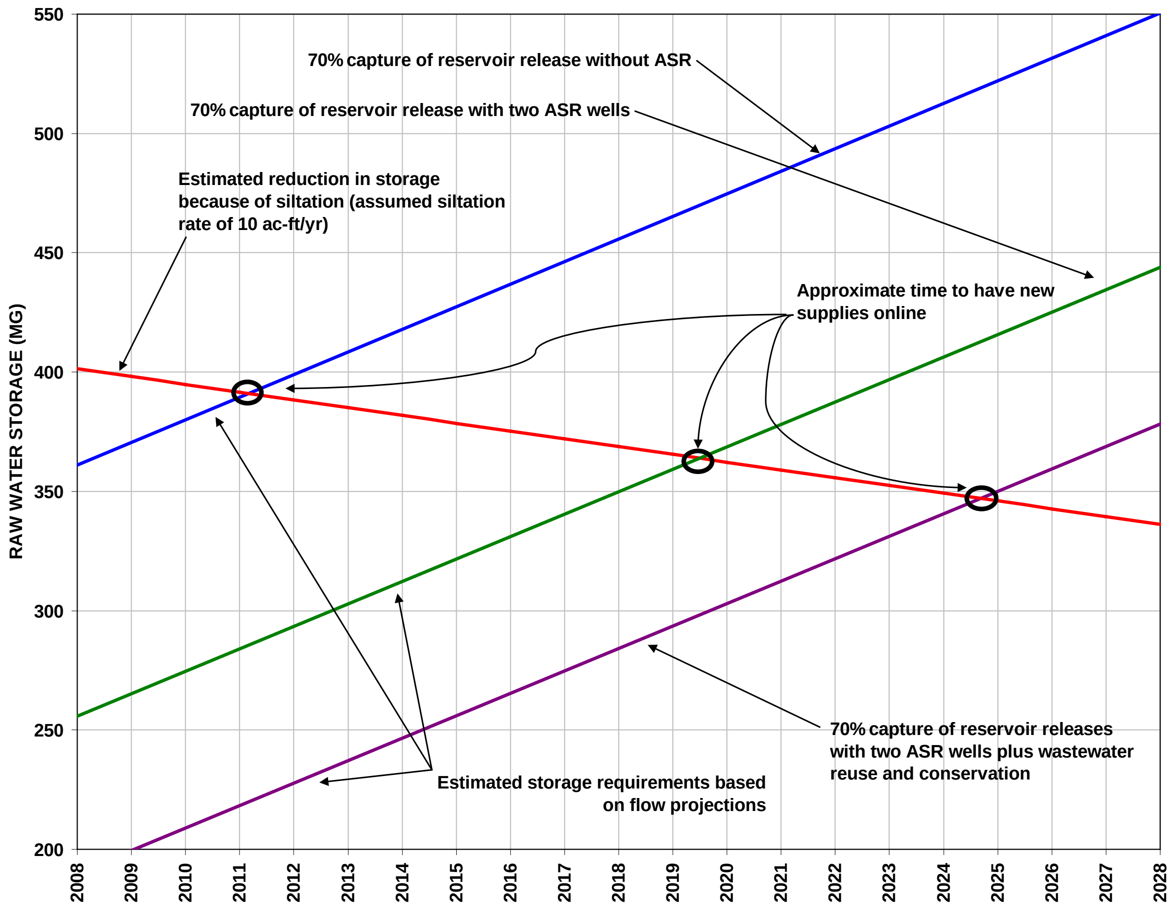


FIGURE 3-6
Estimated Future Storage Requirements with Two ASR Wells, Wastewater Reuse, and Conservation (based on 2028 demands)



Allowances for Losses

Additional storage is also necessary to make up for losses that occur. The losses fall into three categories: those that are part of the operation of the WTP, those that are part of the hydrologic cycle, and those that are operational. The hydrologic cycle losses include evaporation from the reservoir, evaporation from the stream after release from the reservoir, and transpiration from plants along the reservoir and stream banks.

The WTP discharges used water from the filters to backwash ponds prior to releasing to Rickreall Creek. The discharges are comprised of the water used to backwash the filters, and filter-to-waste water produced until the filters meet water quality criteria after the backwash. The amount of this water is approximately 4 percent of the finished water production and varies during the year.

Evaporation and transpiration losses vary with the weather conditions, reservoir stage, volume of water being released from the reservoir (stream width), and plant life along the reservoir and stream banks. Only those losses occurring during deficit stream flow conditions affect the storage requirements. Drought condition evaporation losses during the stream flow deficit period have been estimated from pan evaporation records from the

National Oceanic and Atmospheric Administration (NOAA). The dry season reservoir evaporation losses are estimated to be 10 MG (from *An Engineering Study of the Water Supply for the City of Dallas, Oregon*, CH2M HILL, 1989). Stream evapotranspiration losses during the same period are estimated to be 4 MG. Therefore, 14 MG has been added to the annual storage requirements estimated from Figure 3-5 to account for total natural hydrologic losses.

Operational losses result from the inability to coordinate reservoir releases to exactly match the amount of water that is needed at the intake. The travel time from the reservoir to the intake for released water varies with flow, the instream losses vary with the weather, and the intake pumping varies with demand. Therefore, it is not possible to exactly match release flows to demands. The present operation has improved over the past as a result of the installation of adjustable frequency drives for the intake pumps and operating the plant at a steady continuous rate through the day as opposed to intermittent plant operation. For the purpose of this evaluation, the assumed capture efficiency has been assumed to be 70 percent; that is, an average of 70 percent of the stored water released from the dam is captured at the intake.

Recommendations

The City of Dallas has reached a point where the implementation of an additional water supply source is needed to supplement Mercer Reservoir and the ASR well. The city has already investigated a regional water supply option with Monmouth and Independence (*Regional Water Supply Project Phase 1 & 2 Summary Report*, CH2M HILL, 2003) in which additional water supply was evaluated for the three cities. The two sources of water supply identified in the Summary Report were the Willamette River and an additional reservoir in the Coast Range west of Dallas. Additionally, Polk County is evaluating a regional water supply reservoir near Valsetz. Although a regional water supply project potentially will reduce the cost for the City of Dallas, the timing for other cities may not meet the needs for the City of Dallas. Additionally, a diversion from the Willamette River or a new dam in the Coast Range can take up to 10 years or more to bring online.

In anticipation of the need for an alternative water supply source, the city has been planning a wastewater effluent reuse project. Such a project allows the city to offset a limited amount of existing irrigation water demand supplied by Mercer Reservoir through retrofitting parks and school open spaces. Additionally, the city can supply new residential, commercial, and industrial customers with recycled wastewater to offset the demand on the Mercer Reservoir supply. Supplying recycled wastewater can potentially supply 1 to 2 mgd of irrigation water and offset the seasonal demand on Mercer Reservoir by 100 MG. The existing wastewater plant has been planned and designed to allow the addition of the necessary filtration process required to produce recycled wastewater effluent for public irrigation. Because the peak season demand is comprised of approximately two-thirds irrigation demand and one-third domestic use, the concept of recycling a low-water-quality source is attractive.

The following recommendations should be implemented by the city to meet its water demands in the future:

- Map the reservoir bottom so the present topography can be compared with the original topography to determine the actual change in reservoir volume. If the volume change indicates a deposition rate significantly different than estimated, the timing of future storage requirements would be affected.
- Make watershed improvements to minimize sedimentation.
- Track the capture efficiency by using data gathered from flow measurement points upstream and immediately downstream of the dam during the low flow months. This information will be useful for future planning purposes.
- Develop a conservation and curtailment plan that identifies when and how water restrictions are to be implemented.
- Apply for an operating permit or water right to make the installation of the flashboards an annual activity.
- Continue to develop the city's wastewater effluent recycling project by developing a conceptual design of the components required at the wastewater plant, as well as identifying likely areas of Dallas able to offset Mercer Reservoir water supply and the necessary conveyance infrastructure to distribute the recycled effluent. Additionally, the use of backwash and filter-to-waste water produced by the water treatment plant may be integrated with wastewater effluent recycling.
- Continue to develop the existing ASR well and explore new locations for ASR wells.
- Continue to explore the possibility of a regional water supply in the near term and decide if the conditions and timing meet the city's needs. In parallel with exploring regional opportunities, decide whether a long-term water supply from the Willamette River or Coast Range fits the goals for the City of Dallas as an option if the city needs to proceed without regional support. After an alternative has been selected, proceed with securing the appropriate land, easements, permits and agreements as needed. Table 3-2 lists advantages and disadvantages to consider with regards to the water supply sources available to the city.

TABLE 3-2
Future Water Supply Sources

Source	Advantages	Disadvantages
Wastewater Reuse	Wastewater recycling generally considered a sustainable and “green” solution Uses lowest-water-quality source available for irrigation and conserves high-quality water from Mercer Reservoir for domestic use Reduces pollutant and temperature loading to Rickreall Creek during critical creek low flow periods Wastewater treatment plant already planned and designed for filtration unit process required to produce recycled wastewater effluent for public-access irrigation Relatively low cost and minimal permitting effort when compared to alternative water supply sources. Cost to implement wastewater plant improvements and initial conveyance improvements estimated at $6,000,000 (excludes additional pipeline infrastructure needed to distribute recycled wastewater to areas of city).	Wastewater treatment plant improvements and pipeline infrastructure required to distribute recycled wastewater to areas of the city Additional WWTP operation and maintenance costs Recycled wastewater users need to be identified Public acceptance may be a challenge
Aquifer Storage and Recovery	Water treated and stored during abundant Rickreall Creek flows for later use during high water demand season Recovered water may be usable for landscape irrigation when not appropriate (e.g., high total dissolved solids) for drinking water	Existing ASR being developed and long-term feasibility uncertain
Willamette River	Redundant water supply source should Mercer Reservoir become unusable Potential to use lower-water-quality source for supplemental irrigation supply in conjunction with wastewater recycling	Water rights may be expensive and not available in future No control over flow regulation of upstream Army Corps of Engineers reservoirs Lengthy project to implement because of extensive permitting and land/easement acquisition Negative public perception of Willamette River water Additional treatment unit processes required beyond existing water treatment plant. Additional treatment includes flocculation and settling as well as granular activated carbon filters for taste and odor removal. A new plant would need to be constructed, most likely on the east side of the city.

TABLE 3-2
Future Water Supply Sources

Source	Advantages	Disadvantages
		Costly to implement. Construction cost estimated to range from $20M to $30M for a new intake, raw water pump station, transmission pipeline, water treatment plant, storage tank, high-service pump station, and finished water transmission pipeline
Reservoir in Coast Range	Potential dam and reservoir concepts already identified in *Regional Water Supply Project Phase 1 & 2 Summary Report,* including an off-line reservoir Location easily integrates with existing water system, with raw water conveyed to existing water treatment plant. Existing water treatment plant planned and designed for additional contact basin and filters for an ultimate capacity of approximately 12 mgd. Similar water source and quality as existing supply. Depending on location of new reservoir, a redundant water supply source should Mercer Reservoir become unusable.	Long-term unknown probability of reduction in reservoir capacity because of siltation if an off-line reservoir is not constructed. Lengthy project to implement because of extensive permitting and land/easement acquisition. Costly to implement. Construction cost estimated to range from $20M to $30M. Increased safety risk with a second dam/reservoir upstream of city.

4. Water Treatment

CHAPTER 4

Water Treatment

Facility Description

The Dallas WTP consists of raw water screening and pumping, coagulant rapid mixing, a contact basin, dual-media filtration, chlorine disinfection, and ammonia feed for chloramination. The original plant was constructed in 1972 and expanded in 1993 with the addition of two rapid sand filters and a 2 MG clearwell. The design capacity of the treatment plant after the 1993 expansion was 8.5 mgd maximum-day. The maximum treated at the plant to date is about 5.76 mgd.

The city intake along Rickreall Creek consists of a diversion structure that directs water into a channel with two tee-screens. Screened water from the tee-screens flows into a wet well where raw water is pumped to the WTP by three vertical turbine pumps with adjustable frequency drives. Raw water is conveyed to the WTP by 16-inch steel, 20-inch DI, and 28-inch HDPE pipelines.

At the plant, the rapid mix step uses a flash mixer to mix the raw water with coagulant that is then injected into the water. The city used aluminum sulfate as the coagulant until the past few years, when the city switched to polyaluminum chloride (PACl). The purpose of the flash mixer is to achieve the initial contact between the water and chemical and to begin the destabilization of particles to form a floc in the next steps in the treatment process.

The contact basin process is used to allow heavy particles to settle, which protects the filters from being overloaded by large solids loadings caused from heavy flows in Rickreall Creek during storms. To some extent, the basin promotes interparticle collisions and production of a pinpoint floc for filtration.

The plant uses four dual media filters with anthracite and sand. The dual media is a shallow bed with approximately 24 inches of anthracite followed by 12 inches of sand. The smaller sand media provides a barrier to particle breakthrough at higher loading rates or long filter run times. The filters are cleaned using backwashing and a surface scour system. Backwash water is discharged to two washwater lagoons where the solids are allowed to settle prior to discharging to the overflow weir. The overflow from the lagoons is piped to a discharge point in Rickreall Creek. There is no means to recycle backwash water from the lagoons.

Design Data

Table 4-1 summarizes the design data for the WTP. The design data shown on the table are within typical criteria to treat 8.5 mgd of raw Rickreall Creek water quality.

TABLE 4-1
Water Treatment Plant Design Data Summary (based on 8.5 mgd flow)

Component	Design Data
Raw Water Screening	
Number of Tee-Screens	2
Size	24”
Total Capacity	12.7 mgd (2 x 4,400 gpm each)
Raw Water Pumps	
Number of pumps	3
Total Firm Capacity (mgd)	8.64 mgd (2 x 3,000 gpm each)
Rapid Mixing	
Type	In-line mechanical
Size	24”
Velocity Gradient “G”	1,000 1/sec
Mixer Power	2 horsepower
Contact Basin	
Number of Basins	1
Volume per Basin	0.29 MG
Detention Time	49 minutes
Surface Loading	2,711 gpd/ft^2
Filters	
Type	Multimedia granular, gravity
Number of Filters	4
Filter Area	288 ft^2 each (1,152 ft^2 total)
Filtration Rate	5.1 gpm/ ft^2 (all filters online)
	6.8 gpm/ ft^2 (one filter offline)
Filter Effluent Pumps	
Number of pumps	3
Total Firm Capacity	8.64 mgd (2 x 3,000 gpm each)
Clearwell	
Capacity	1.93 MG
Contact Time	327 minutes at full capacity

Regulatory Evaluation

Three main regulations are applicable to the WTP: the Stage 2 Disinfectant/Disinfection Byproduct Rule (DBPR), Long- Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR), and the Lead and Copper Rule.

The DBPR is intended to reduce health risks from disinfection byproducts (DBPs) in drinking water, which form when chlorine is used to control microbial pathogens. This rule strengthens public health protection for customers of systems that deliver chlorinated water by requiring such systems to meet maximum contaminant levels as an average at each compliance monitoring location (instead of as a system-wide average as in previous rules) for two groups of DBPs: trihalomethanes (TTHMs) and five haloacetic acids (HAA5s). The rule targets systems with the greatest risk and builds incrementally on existing rules. This regulation will reduce DBP exposure and related potential health risks and provides more equitable public health protection. The Stage 2 DBPR was released simultaneously with the LT2ESWTR to address concerns about risk tradeoffs between pathogens and DBPs.

The purpose of LT2ESWTR is to reduce illness linked with the contaminant *Cryptosporidium* and other pathogenic microorganisms in drinking water. The LT2ESWTR will supplement existing regulations by targeting additional *Cryptosporidium* treatment requirements to higher risk systems. This rule also contains provisions to reduce risks from uncovered finished water reservoirs and provisions to ensure that systems maintain microbial protection when they take steps to decrease the formation of DBPs that result from chemical water treatment.

Current regulations require filtered water systems to reduce source water *Cryptosporidium* levels by 2-log (99 percent). Recent data on *Cryptosporidium* infectivity and occurrence indicate that this treatment requirement is sufficient for most systems, but additional treatment is necessary for certain higher-risk systems. These higher-risk systems include filtered water systems with high levels of *Cryptosporidium* in their water sources, and all unfiltered water systems, which do not treat for *Cryptosporidium*.

Currently, the city is undergoing distribution system testing for the Stage 2 DBPR and raw water quality testing for *Cryptosporidium* required by LT2ESWTR. Because of the use of chloramines, DBPs have not historically been a problem for the city and that trend is expected to continue. *Cryptosporidium* in the city source water supply is also not expected to be an issue because there are no farmland or wastewater plants upstream of the city water intake.

Lead and copper enter drinking water primarily through plumbing materials. Exposure to lead and copper may cause health problems. On June 7, 1991, the U.S. Environmental Protection Agency (EPA) published a regulation to control lead and copper in drinking water. This regulation is known as the Lead and Copper Rule (also referred to as the LCR or 1991 Rule).

The treatment technique for the rule requires systems to monitor drinking water at customer taps. If lead concentrations exceed an action level of 15 parts per billion (ppb) or copper concentrations exceed an action level of 1.3 ppm in more than 10 percent of customer taps sampled, the system must undertake a number of additional actions to control corrosion. If the action level for lead is exceeded, the system must also inform the public about steps they should take to protect their health, and may have to replace lead service lines under their control.

Historically, the city has not had an issue with lead and copper until recently. It is suspected that the change from aluminum sulfate to polyaluminum chloride as the coagulant has caused the corrosion potential to increase. The city is adding an orthophosphate (a corrosion inhibitor) feed system at the WTP to reduce the corrosion potential, in conjunction with maintaining increased alkalinity using soda ash.

Recommendations

The City of Dallas WTP has consistently been able to produce a high-quality water for its customers over the years. No major changes to the treatment process are required to meet current water quality regulations and no major changes are anticipated in the near future. The capacity of the plant is adequate to meet the needs up to the end of the study year (2028); however, with the implementation of a wastewater effluent recycling project, the capacity of the plant will be adequate beyond the study year. If the wastewater effluent recycling project is not implemented, then the WTP will need to undergo an expansion to include a second contact basin and two additional filters. The plant has been designed to accommodate these additions. Design of the plant expansion, if necessary, should occur a minimum of 5 years prior to the estimated capacity being exceeded, with construction started a minimum of 4 years prior.

The following non-capacity improvements are recommended to be completed:

- Effluent weirs are deteriorating and need to be replaced.
- Effluent flow meter cannot accurately read low flows and should be replaced in conjunction with the new 24-inch finished water pipeline.
- Because of possible future Department of Homeland Security regulations, consider switching from chlorine gas to bulk sodium hypochlorite or onsite generation of sodium hypochlorite. This has recently been evaluated and postponed for the next 5 years after the Risk Management Plan update is required.

5. Finished Water Transmission, Storage, and Distribution System Analysis

CHAPTER 5

Finished Water Transmission, Storage, and Distribution System Analysis

(Note: Figures 5-3 through 5-10 are presented at the end of this section.)

This chapter contains an analysis of the Dallas Water Transmission, Storage, and Distribution System for existing 2008 and future 2028 demands. The analysis includes the evaluation of the pumping, storage, transmission and distribution components of the Dallas Water System.

System Analysis Criteria and Hydraulic Model

This section presents the master plan system analysis criteria to be used in the existing and future system analysis. This section also contains a discussion of the hydraulic model and its development and verification process.

Analysis Criteria

The following criteria were used to evaluate the adequacy of the water system to meet existing 2008 and projected 2028 demands. The proposed criteria meet the Oregon Health Division (OHD) and the Oregon Water Resources Department (OWRD) requirements, and are accepted standards of practice in typical master plan studies. The analysis criteria contained herein are for water system master planning analysis and are not intended as specific development standards.

Source and Pumping

The source and pumping capacities must be adequate to supply water demands in each service area. For service areas with storage facilities for peaking equalization, the source and pumping capacity required is the MDD. Demands greater than MDD are served from the reservoir storage. However, for closed-end service areas served by small booster stations without storage facilities, the PHD will be used to evaluate additional capacity needs.

Typically, the capacity of a pumping facility is evaluated considering the largest pump being out of service. This is referred to as the firm pumping capacity of a facility. However, Dallas has several small booster pump stations serving small service areas and, except in the case of the Maple Street Pump Station (which has two equal-capacity pumps in a fixed lead/lag operation plus a third dedicated fire pump), and the Elmwood Drive Pump Station, which has two identical-capacity pumps that alternate each time they are started, they contain only one small jockey and one large booster pump in the booster stations. For these small booster stations, the small jockey pump normally handles up to the MDD and the large booster pump is used to meet short periods of PHD and provides backup service if the jockey pump is out of service. With the Orchard Drive, Upper Bridlewood, and Church Street pump stations, the jockey pump capacity will be compared to the MDD to determine if additional jockey pump capacity is required. Similarly, the large booster pump capacity in these stations will be compared to the PHD to determine if additional booster pumping capacity is required. For the purpose of analyzing pumping requirements for the Maple Street and Elmwood Drive pump stations, a single pump in operation is expected to meet MDD and PHD conditions, leaving the second pump for redundancy and fire flow conditions.

Storage

There is no storage criteria set by the OHD. A typical standard of practice in master plan studies, however, is to divide the total storage requirement into three components: peaking equalization, fire flow, and emergency storage. The total storage requirement for the Dallas Water System is recommended to be the sum of these three components, as follows:

- Peaking equalization storage is used when demands exceed the MDD supply or pumping capability of the system. Storage for peaking equalization is assumed in the master plan to be 25 percent of MDD.
- Fire flow storage is determined based on fire flows of 3,500 gpm for commercial areas and 1,500 gpm for residential areas at 3- and 2-hour durations, respectively.
- Emergency storage volume has the most flexibility in sizing and depends largely on the individual system makeup, lengths of historical emergency outages, and the level of risk the utility is willing to take. An emergency storage volume requirement of two times ADD is assumed in this master plan.

Pipeline

The distribution pipeline network must be able to meet MDD and maintain pressures above 35 pounds per square inch (psi) while maintaining fluid velocities in the pipeline less than 6 feet per second (fps). Water mains should be looped as much as possible to prevent dead ends, maintain high water quality, and increase reliability in the system. The sizing of water mains should be for the maximum potential demands and fire flow requirements according to the city zoning or planning area.

The pipeline network should provide the required flows for fire and MDD with a minimum residual pressure of 20 psi, which was established by OHD. The network also must be of adequate size for reservoir refill during low demand periods of the day. The pressures in the transmission system should not fluctuate by more than 20 to 30 psi from normal ADD pressures as sources or pumps refill the reservoirs.

Computer Simulation Model

The H2OMap network analysis software was selected to simulate the hydraulics of the Dallas Water System. H2OMap is a private domain software program developed by MWH Soft, Inc. The geographic information system (GIS) based H2OMap was used in this master plan study. The computer model was developed from digital pipeline system maps provided by Dallas. The model contains pipe, node, pump and reservoir data. Nodes are the interconnecting points of the pipeline network. The existing Dallas Water System model contains about 630 pipes and 500 nodes.

The computer model output provides an indication of how the system responds to various operation scenarios. The output lists the pressures and hydraulic grade lines at the nodes, velocity and friction losses through the pipes, and the operating conditions of all the facilities in the model.

For each service area, the fraction of the total system demand allocated to that area was estimated based on the fraction of total service connections attributed to each particular service area. Table 5-1 summarizes the resulting service area. Within each service area, the allocated demands were divided evenly to all nodes (except for the current large-volume water users summarized in Table 5-5). The demands of the large-volume water users were applied to the node closest to the demand. Existing and future ADD and MDD are presented in Chapter 2.

TABLE 5-1
Year 2008 Demand Distribution by Service Area

Service Area	Estimated ADD to Each Area (mgd)	Estimated ADD to Each Area (gpm)	Estimated MDD to Each Area (mgd)	Estimated MDD to Each Area (gpm)	Estimated PHD to Each Area (mgd)	Estimated PHD to Each Area (gpm)
Main Service Area—Level I	2.42	1,684	5.58	3,873	8.37	5,809
Orchard Drive Pump Station—Level 2	0.12	83	0.27	191	0.41	286
Douglas Street PRV—Level 2	0.06	45	0.15	104	0.22	155
Maple Street Pump Station—Level 2	0.02	11	0.04	25	0.05	38
Church Street Pump Station—Level 2	0.02	15	0.05	35	0.07	52
Elmwood Drive Pump Station—Level 2	0.06	43	0.14	99	0.21	148
Upper Bridlewood Pump Station—Level 3	0.02	15	0.05	35	0.07	52

mgd = million gallons per day; gpm = gallons per minute
ADD = average day demand; MDD = maximum day demand; PHD = peak hour demand
PRV = pressure-reducing valve

Hydraulic Computer Model Calibration

The model was calibrated by comparing field-measured pressures to computer model-predicted pressures. Flows at hydrants, reservoir levels, and pressures were measured and compared to model-predicted values. Obvious errors in pipe connections, pipe diameters, and demand allocations were corrected, and model computations were adjusted to match field-measured conditions within a reasonable range of tolerance.

Two model simulations were created in the hydraulic computer model to analyze the two field scenarios: static, and dynamic. The static scenario simulated the water system just prior to each test, with no hydrants flowing. The dynamic scenario simulated the system during the test while the hydrants were flowing. Table 5-2 and Table 5-3 compare field measurements with model calculations. Pressure recorder data collected in the field are presented in Figures 5-1 and 5-2.

The system demands during static conditions are significantly lower than the demands with the flowing hydrants included. With the relatively low flow conditions, pipe head losses were generally small. Therefore, static demands provide a condition under which the elevations of junction nodes can be verified. Dynamic calibration refers to a steady-state calibration when the system is being stressed by a significant and measurable demand at one or more fire hydrants.

Where significant differences were revealed between the model calculations and the field measurements, the model data were rechecked against known data. This primarily involved checking elevations, consistency of measured hydraulic grade lines (HGLs) with known HGLs from nearby tanks, pipe diameters, pipe lengths, and pipe connections.

TABLE 5-2
Field Measurements for Static Scenarios Compared to Model Calculations

Test No.	Location	Elevation feet	Field Static Pressure psi	Model Static Pressure psi	Static Pressure Difference psi	Static Percent Difference
1	Academy/Levens	330	73	71	1.6	2.2%
2	Birch/Brown	354	64	63	1.5	2.4%
3	Dogwood Ct.	358	61	61	0.4	0.7%
4	Glendover/Rhododendron	272	98	97	0.5	0.6%
5	Greenbriar/Cynthia	301	86	85	1.0	1.2%
6	Heath Ct.	404	42	41	1.4	3.4%
7	River/Richard	339	69	68	1.2	1.7%

TABLE 5-3
Field Measurements for Dynamic Scenarios Compared to Model Calculations

Test No.	Location	Hydrant Flow gpm	Field Dynamic Pressure Drop psi	Model Dynamic Pressure Drop psi	Dynamic Pressure Drop Difference psi	Dynamic Percent Difference
1	Brentwood/Orchard	1,185	4	4	0	0%
2	Rhododendron/St. Andrews	1,422	32	27	5	7%
3	Greenbriar/Cynthia	1,452	18	18	1	1%
4	Cedar/Cypress	1,351	20	26	6	9%
5	Gregory/Virginia	1,351	23	30	7	10%
6	Ellis/Birch	1,243	5	5	0	0%
7	Court/Lyle	1,330	5	7	2	2%

FIGURE 5-1
Pressure Monitoring Data (Recorder #274)

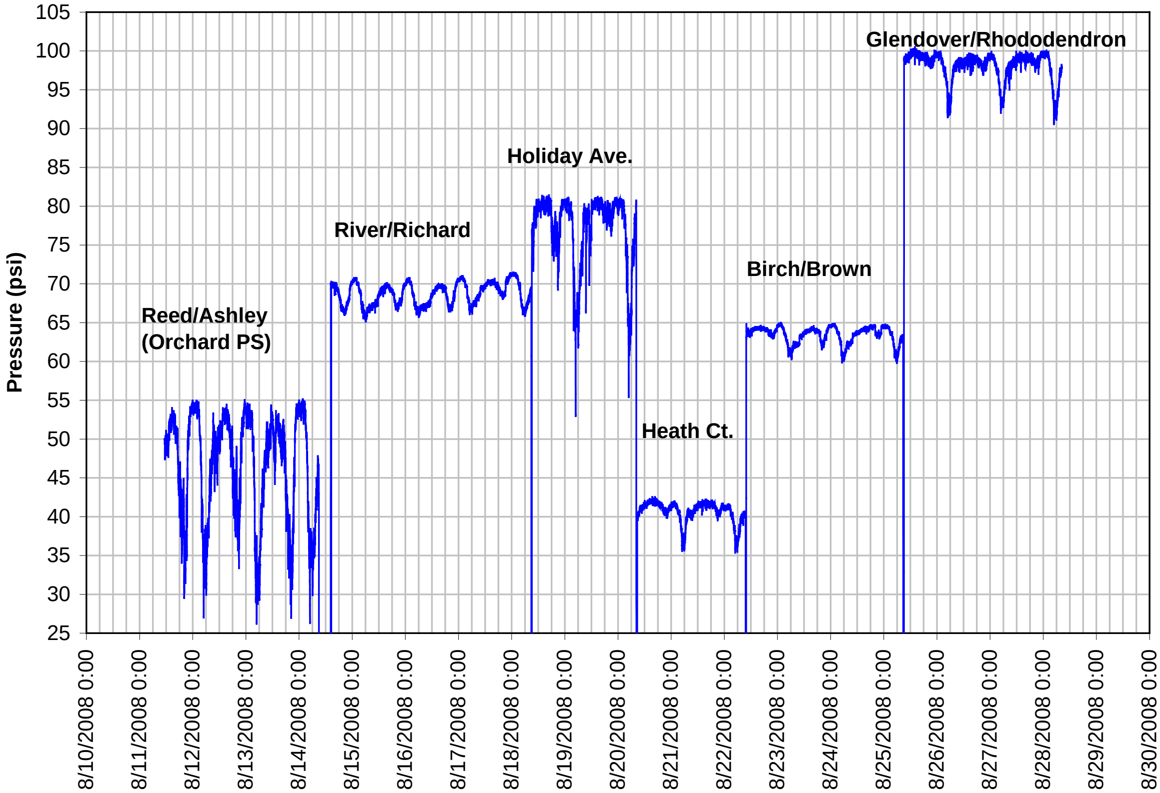
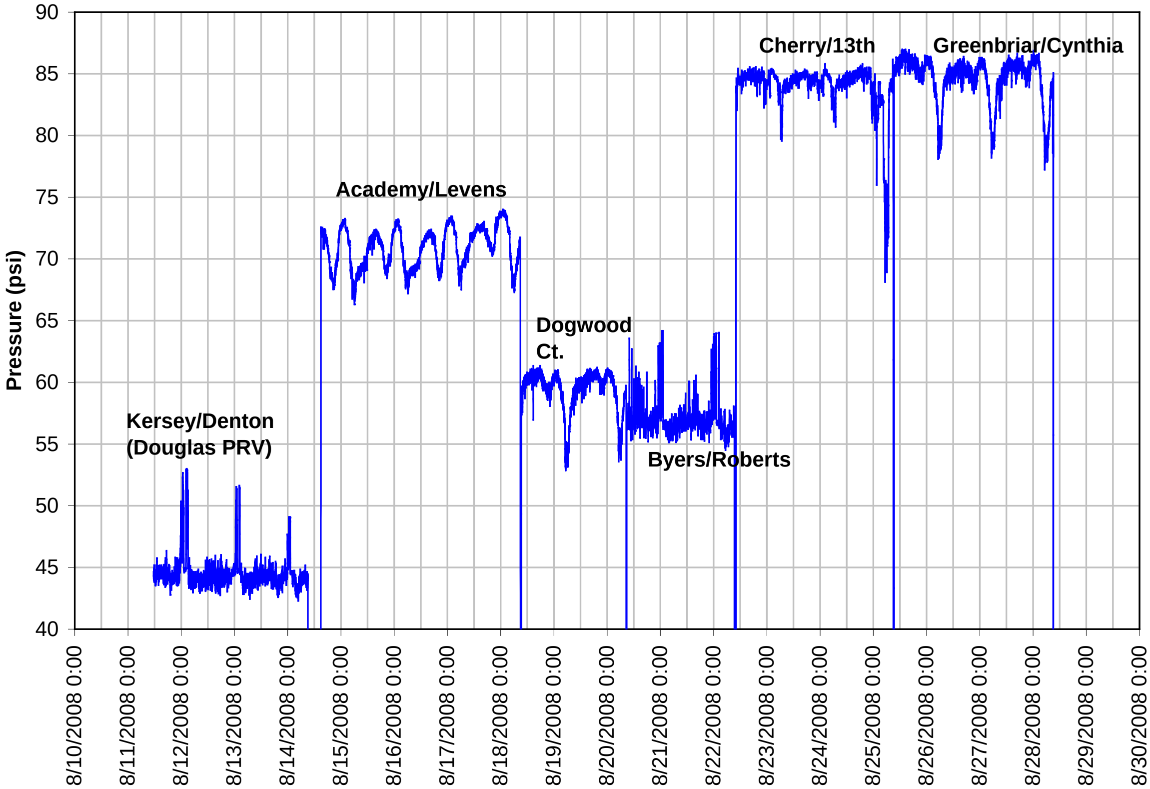


FIGURE 5-2
Pressure Monitoring Data (Recorder #275)



Friction factors were developed and provided in the original model that was developed for the 2002 Water Master Plan work. Potential major differences between field-measured pressures and model computations are typically not dependent on friction factors, but on bad pressure readings, bad flow readings, inaccurate elevation information, pipe diameters, pipe connections, or unintentionally closed valves.

Calibration Summary

This model update included development and verification of the physical components represented in the hydraulic model, review of system demands in the model, field testing and calibration of the updated model. Overall, the verification data indicate that the model accurately simulates the water system for the purpose of master planning analysis. Analysis results generally are within 5 percent of the field test data for static scenarios and 10 percent for dynamic scenarios. The conclusion of the model verification analysis is that the water system model is adequate for the water master plan analysis.

Model calibration for any water system is an ongoing effort. As changes in the system occur based on changing demands, new infrastructure development, or changing operational settings, the model must be periodically updated and checked with field measurements to ensure agreement.

Demand Allocation and Growth

For analysis purposes, total system demands were allocated to each area based on the fraction of total service connections attributed to each particular service area as discussed in Chapter 2. However, for the future system analysis, growth in demand was allocated both to existing nodes in the system and to new nodes assumed in areas expected to be developed. Based on the projected number of new connections in each service area provided by Dallas staff, 30 percent of the demand growth between 2008 and 2028 was allocated to new development and the remaining 70 percent was allocated to infill in the existing grid. New development was added to Service Level 1 in the Wyatt, LaCreole, and Barberry growth nodes, which are shown in Figure 5-3. Additional development in Service Level 2 was added to the Orchard Drive and Maple Street pump stations and Douglas PRV service areas. Growth in the Elmwood Drive and Upper Bridlewood Pump Station service areas was assumed to be infill within the existing grid and no new growth was assumed in the Church Street Pump Station service area. Table 5-4 presents a breakdown of this demand allocation for each service level and service area. For analysis under future conditions, demand is distributed to these areas to account for effects on the current piping system from allocation of future demand in these areas to be developed.

Large-Volume Users

Large-volume users create high point-loads on the system and need to be applied to the water system model to accurately analyze the system. Dallas currently has eight large-volume water users that have ADD of 5 gpm or higher. The ADD is based on annual usage. The actual daily and hourly peak use will vary depending on their specific use. A MDD peaking factor of 2.0 for these large-volume users was assumed for this study. Large-volume user peak hour usage is assumed to be the same as the maximum day usage. Table 5-5 summarizes the large-volume user locations and demands.

TABLE 5-5
Current Large-Volume Water Users

Customer	Location	2001 ADD (gpm)
Greenway Mobile Home Park	300 to 450 SE LaCreole	52
Meadow Creek Mobile Home Park	1401 W. Ellendale	22
Dallas High School	901 SE Ash	22
Whitworth Ball Complex	1100 Block of SE Miller	9
LaCreole Ball Complex	700 SE Academy	8
LaCreole Jr. High School	701 SE LaCreole	8

ADD = average day demand
gpm = gallons per minute

TABLE 5-4
Demand Allocation and Growth

	2008			2028		
	ADD (gpm)	**MDD (gpm)**	**PHD (gpm)**	**ADD (gpm)**	**MDD (gpm)**	**PHD (gpm)**
Service Level 1						
Existing 2008 Grid	1,684	3,873	5,809	1,939	4,437	6,584
Wyatt Growth Node	0	0	0	31	76	128
LaCreole Growth Node	0	0	0	62	152	257
Barberry Growth Node	0	0	0	58	142	241
Service Level 2						
Orchard Drive Pump Station Existing 2002 Grid	83	191	286	83	191	286
Orchard Drive Pump Station Expansion	0	0	0	23	53	79
Douglas Street PRV Existing 2002 Grid	45	104	155	45	104	155
Douglas Street PRV Expansion	0	0	0	16	37	55
Maple Street Pump Station Existing 2002 Grid	11	25	38	11	25	38
Maple Street Pump Station Expansion	0	0	0	34	78	117
Church Street Pump Station Existing 2002 Grid	15	35	52	15	35	52
Elmwood Drive Pump Station Existing 2002 Grid	43	99	148	48	110	166
Service Level 3						
Upper Bridlewood Pump Station Existing 2002 Grid	15	35	52	17	39	59
Total (gpm)	**1,896**	**4,360**	**6,541**	**2,382**	**5,478**	**8,218**
Total (mgd)	**2.73**	**6.28**	**9.42**	**3.43**	**7.89**	**11.83**

[1] 694.4 gpm = 1 mgd

gpm = gallons per minute; mgd = million gallons per day

ADD = average day demand; MDD = maximum day demand; PHD = peak hour demand

Future water demands for the current large-volume water users are not expected to change significantly; however, it is expected that other large-volume users will be developed during the study period within the planned industrial area of the city located in the southeast section of the city bounded generally by Fir Villa Road, Miller Avenue, Godsey Road and Monmouth Cut-off Road. For this reason, one large-volume water user will be assumed for the purpose of the future system analysis within the planned industrial area. A MDD of 0.50 mgd has been assumed.

Pump Stations

Table 5-6 provides analyses of pumping requirements for the Orchard Drive, Maple Street, Church Street, Elmwood Drive, and Upper Bridlewood pump stations. Table 5-6 compares the 2008 and 2028 service area demands and fire flow requirements to the available pumping capacity provided by the jockey and booster pumps at each station. The Maple Street and Elmwood Drive pump stations do not have the jockey/booster pump arrangement, but instead have two identical-capacity pumps. A single pump in operation at these stations is expected to meet ADD, MDD, and PHD conditions, leaving the second pump for redundancy and fire flow conditions. The Church Street Pump Station contains a single pump. Fire flow is not provided to the Church Street service area and the single pump production is compared to the PHD to determine if additional capacity is required. No redundant pumps are being provided at any of the booster pump stations, with the exception of the Elmwood Pump Station, which was upgraded with redundant pumps in 2009.

The Church Street service area is served by a single 240-gpm-capacity pump. No second pump is provided. Although this pump has adequate capacity to meet service area PHD, when the pump is out of service it is bypassed and the area is served off of the main service area. Under this condition the Church Street service area experiences low pressures until the pump station is online again. In addition, because there is no large-capacity pump available, it has been a practice that fire flow availability in this area is provided by fire tanker trucks. The Church Street service area is not expected to grow in the future.

In Table 5-6, the total pumping capacity available for fire flows is presented for each pump station service area. It should be noted that with the exception of the Maple Street service area, which has a 2,300-gpm fire pump, and the Elmwood service area, which has additional fire flow available from the Bridlewood Reservoir, all other pump stations (Orchard Drive, Church Street, and Upper Bridlewood) do not have adequate fire flow capacity. A total pumping capacity for fire flow of 1,500 gpm is recommended in these areas. The Orchard Drive Pump Station is currently (2008) deficient by about 650 gpm under a PHD plus fire flow condition and is projected to be deficient by more than 700 gpm in 2028. It should be noted that in the future, the Orchard Drive Pump Station would be used only in emergency or fire situations after a pipeline improvement, discussed later in this chapter, has been constructed that combines the existing Douglas Street service area with the Orchard Drive service area. Both service areas would then be served directly from the WTP reservoir. This pipeline improvement will allow for adequate fire flows to the Orchard Drive service area and no pumping improvements will be required. The Upper Bridlewood Pump Station currently (2008) needs an additional 550 gpm to provide PHD plus fire flow and is projected

TABLE 5-6
Pump Station Capacity (gpm)

Service Area	Orchard Drive	Maple Street	Church Street	Elmwood Drive	Upper Bridlewood
Supplied by Reservoir	No	No	No	Yes	No
Level of Service Provided by Pump Station[1]	PHD	PHD	PHD	MDD	PHD
2008 Domestic Requirements	286	39	52	100	52
2008 Fire Flow Requirements	1500	1500	NA[2]	0[3]	1500
Total 2008 Requirements	1786	1539	52	100	1552
Existing Capacity	1150	2660	240	195	1000
2002 Surplus/(Deficit)	**(636)**	**1121**	**188**	**95**	**(552)**
2028 Domestic Requirements	364	157	52	110	60
2028 Fire Flow Requirements	1500	1500	NA[2]	0[3]	1500
Total 2028 Requirements	1864	1657	52	110	1560
Existing Capacity	1150	2660	240	195	1000
2022 Surplus/(Deficit)	**(714)**	**1003**	**188**	**85**	**(560)**

[1] For zones without storage, PHD must be provided by pumping capacity.
[2] Fire flow is not provided to the Church Street zone by the city.
[3] Fire flow provided by Upper Bridlewood Reservoir.
gpm = gallons per minute; MDD = maximum day demand; PHD = peak hour demand

to need 560 gpm under 2028 conditions. Currently, fire protection in these service areas must rely on fire tanker truck pumping capacity for flows greater than each pump station capacity. It should be noted that to provide fire flows of 1,500 gpm to several of the upper service areas, upgrades to the existing piping will be required in addition to the pump station improvements.

Storage

The Dallas Water System currently contains seven storage facilities at four sites: the WTP reservoir, the Clay Street Reservoirs (four), Main Street Reservoir and the Bridlewood Reservoir. The WTP reservoir, Main Street Reservoir, and Clay Street Reservoirs provide storage volume for Service Level 1 and the Orchard Drive, Church Street, and Maple Street pump stations and Douglas PRV service areas in Service Level 2. Although fire flow storage for these pump station service areas is provided in the main level storage, actual fire flows in these areas are limited to the capacity of each respective pump station. The Bridlewood Reservoir provides storage for the Elmwood service area in Service Level 2 and the Upper Bridlewood service area in Service Level 3.

Service Level 1 system storage is adequate to meet the storage requirements given the recommended storage criteria. Table 5-7 shows the total system storage requirement based on the existing system demands. Required storage is based on the sum of 25 percent of MDD, two times ADD, and fire flow storage. Fire flow storage requirements for this analysis assumed a 3,500 gpm industrial/commercial fire flow for 3 hours in the main service level served by the WTP reservoir, Clay Street Reservoirs, and Main Street Reservoir; and a 1,500-gpm fire flow for 2 hours for the area served by the Bridlewood Reservoir.

Based on the demand distribution, the Bridlewood service area has a storage deficiency of 0.27 MG. During the next 20 years this storage deficiency will grow to 0.29 MG. Although this analysis indicates that additional storage is needed in the Bridlewood service area, it should be noted that the Bridlewood Reservoir can provide some residential fire flow in the area, although it will not meet fire flow, peaking equalization, and emergency storage needs simultaneously. Similar to the other pump station service areas in town, storage capacity for these needs can be provided from Service Level 1 and supplied to the area through the pump station. Therefore, it is critical for Dallas to ensure that the water system has adequate storage to meet the overall system needs and that individual pump stations have adequate capacity to serve the needs of the closed-end service areas.

The overall 2008 storage for the total Dallas Water System meets the requirements. Without additional storage facilities added to the system, a deficiency will grow to 1.74 MG during the next 20 years. Based on the current (2008) water use projections, a second 2- to 3-MG reservoir would be required in the system before 2028. A possible site in the Wyatt node has been identified for this reservoir.

TABLE 5-7
Dallas Storage Requirements

	Main Service Area Storage [1]					Bridlewood Service Area Storage [2]					Total Dallas Water System		
Year	ADD (mgd)	MDD (mgd)	Required Storage[3] (MG)	Existing Storage[4] (MG)	Additional Capacity Required (MG)	ADD (mgd)	MDD (mgd)	Required Storage[3] (MG)	Existing Storage (MG)	Additional Capacity Required (MG)	Total Required Storage (MG)	Existing Storage (MG)	Additional Capacity Required (MG)
2008	2.73	5.73	7.52	8	--	0.088	0.202	0.41	0.135	0.27	7.93	8.135	--
2009	2.60	5.98	7.32	8	--	0.088	0.202	0.41	0.135	0.27	7.73	8.135	--
2010	2.63	6.05	7.40	8	--	0.089	0.205	0.41	0.135	0.27	7.81	8.135	--
2011	2.66	6.12	7.48	8	--	0.089	0.205	0.41	0.135	0.27	7.89	8.135	--
2012	2.69	6.18	7.55	8	--	0.090	0.207	0.41	0.135	0.28	7.97	8.135	--
2013	2.72	6.25	7.63	8	--	0.090	0.207	0.41	0.135	0.28	8.04	8.135	--
2014	2.75	6.32	7.71	8	--	0.090	0.207	0.41	0.135	0.28	8.12	8.135	--
2015	2.78	6.39	7.79	8	--	0.091	0.209	0.41	0.135	0.28	8.20	8.135	0.07
2016	2.83	6.50	7.91	8	--	0.091	0.209	0.41	0.135	0.28	8.33	8.135	0.19
2017	2.88	6.62	8.04	8	0.04	0.092	0.212	0.42	0.135	0.28	8.46	8.135	0.32
2018	2.93	6.73	8.16	8	0.16	0.092	0.212	0.42	0.135	0.28	8.58	8.135	0.45
2019	2.98	6.84	8.29	8	0.29	0.093	0.214	0.42	0.135	0.28	8.71	8.135	0.58
2020	3.02	6.96	8.42	8	0.42	0.093	0.214	0.42	0.135	0.28	8.84	8.135	0.70
2021	3.07	7.07	8.55	8	0.55	0.094	0.216	0.42	0.135	0.29	8.97	8.135	0.83
2022	3.12	7.19	8.68	8	0.68	0.094	0.216	0.42	0.135	0.29	9.10	8.135	0.96
2023	3.18	7.30	8.81	8	0.81	0.094	0.216	0.42	0.135	0.29	9.23	8.135	1.09
2024	3.23	7.42	8.94	8	0.94	0.094	0.216	0.42	0.135	0.29	9.36	8.135	1.22
2025	3.28	7.53	9.07	8	1.07	0.094	0.216	0.42	0.135	0.29	9.49	8.135	1.35
2026	3.33	7.65	9.19	8	1.19	0.094	0.216	0.42	0.135	0.29	9.62	8.135	1.48
2027	3.38	7.77	9.32	8	1.32	0.094	0.216	0.42	0.135	0.29	9.75	8.135	1.61
2028	3.43	7.88	9.45	8	1.45	0.094	0.216	0.42	0.135	0.29	9.88	8.135	1.74

1 Main service area storage analysis includes demands and storage requirements for the Orchard Drive, Church Street, and Maple Street pump stations service areas because these systems do not have individual storage facilities. Storage for these higher elevation closed-end pump station service areas is provided by the storage facilities in the main Service Level 1 and, therefore, they are not analyzed separately. Although fire flow storage for these areas is provided in the main level storage, actual fire flows in these higher areas are limited to the capacity of each respective pump station.

2 Bridlewood service area storage analysis includes demands and storage requirements for the Elmwood Drive and Upper Bridlewood Pump Station service areas. Note that although this small storage tank does not meet the storage requirement for these areas, storage from the main Service Level 1 can be delivered to this system at a rate equal to the maximum capacity of the Elmwood Drive Pump Station.

3 Required storage calculation based on the sum of 25 percent of MDD, two times ADD, and fire flow storage. Fire flow storage for the main service area assumes a commercial/industrial fire flow requirement of 3,500 gpm for 3 hours. Fire flow storage for the Bridlewood service area assumes a residential fire flow requirement of 1,500 gpm for 2 hours.

4 Existing storage for the main service level includes 4 MG at the Clay Street Reservoirs, 2 MG at the Main Street Reservoir and the 2-MG Water Treatment Plant (WTP) reservoir. It should be noted that although the WTP reservoir contains 2 MG of storage, under normal operating conditions it is drained only 7 out of 30.5 feet and operates as the WTP's clearwell providing chlorine contact time for effective disinfection. Therefore, the resulting additional capacity calculations should be considered as a minimum requirement.

MG = million gallons; mgd = million gallons per day; gpm = gallons per minute
ADD = average day demand; MDD = maximum day demand

Pipeline

The piping system was analyzed for 2008 and 2028 ADD, MDD, and PHD conditions assuming reservoir levels 5 feet below overflow and that each pump station has the appropriate pumps on to meet the allocated demands in each service area. The existing piping system is generally adequate to serve the existing ADD, MDD, and PHD conditions for Dallas Water System customers in all pressure zones under 2008 and 2028 conditions. Minor deficiencies have been identified and are addressed later in this chapter.

The hydraulic model output data (system pressures and pipeline velocities) were analyzed relative to the pipeline criteria presented previously in this chapter. These criteria specified that the distribution pipeline network must be able to meet PHD and maintain pressures above 35 psi while maintaining fluid velocities less than 6 fps.

Figures 5-4 and 5-5 illustrate the PHD system pressures for 2008 and 2028 conditions, respectively. In Figure 5-4, it is clear that under current PHD conditions, most of the system operates within acceptable pressure ranges, generally more than 35 psi and less than 80 psi. Pressures between 20 and 35 psi can be found in the higher elevations of Service Level 1. Lower pressures are expected in these higher elevation areas and it should be noted that under ADD and MDD conditions, many of the low-pressure nodes in this area still maintain pressures more than 35 psi. Although other low-pressure nodes occur throughout the system, many of these nodes are found in areas in the vicinity of the reservoirs, high points in the system piping, and at dead-end extremities of the system with high elevations. These results are to be expected. Overall, current system pressures under domestic flow conditions are excellent throughout the city.

Under 2028 PHD conditions, as illustrated in Figure 5-5, several nodes in the model exhibit low pressures. The area north of West Ellendale Avenue and south of the Orchard Drive service area shows many more nodes between 20 and 35 psi, with a few nodes less than 20 psi. As shown in Figure 5-5, additional low-pressure areas occur near the suction side of the Upper Bridlewood Pump Station, near SW Maple Street and SW 11th Street, and sporadically along SE Hankel Street. Figure 5-6 identifies system pressure under 2028 MDD conditions. Many of the low-pressure areas identified under 2028 PHD conditions are reduced to extents similar to those found under 2008 PHD conditions.

Many of the low pressures found under 2028 PHD are caused by high elevations, as mentioned previously, and others are caused by high velocities occurring in pipes under increased demand resulting from future growth projections. To address the low pressures caused by high elevation, rezoning of the service areas to serve the lower pressure areas would be required. Because of the relatively small number of these locations, rezoning will not be evaluated as a solution. To address low pressures caused by high pipe velocities and the resulting high frictional losses, pipe replacement, pipe paralleling, or new pipeline alignments need to be evaluated. One of the best ways to ensure adequate water volume and pressure throughout the system is to develop a large-diameter pipeline grid. Dallas has been developing this grid with large-diameter pipes running from the Clay Street and Main Street reservoirs along Clay Street to Orr Corner Road and along West Ellendale Avenue and Kings Valley Highway. Several 10- and 12-inch-diameter pipelines from north-south connections are located between the east-west alignments. The first step in evaluating any areas where high velocities may be occurring is to ensure the continuity of the pipeline grid. Through conversations with the city and by using an iterative modeling approach, several pipeline improvements were identified to strengthen the overall pipeline grid in Service Level 1. Figure 5-7 shows 2028 PHD with numerous capital improvements. With these improvements, virtually all low pressures have been eliminated from the system. Improvement P-7 was identified as developer-financed improvements that will allow the area north of West Ellendale Avenue to be served by the WTP Reservoir and for the Orchard Drive Pump Station to be removed from service and used as a redundant fire flow pumping source. Improvement P-3 is associated with the Main Street Reservoir, which significantly improves pressure to the east portion of the city under peak demands.

It should be noted that the Orchard Drive service area can be served directly from the WTP Reservoir through the Douglas Street PRV from an 8-inch and 6-inch high-pressure pipeline that branches from the 18-inch pipeline along West Ellendale Avenue; however, the 8-inch and 6-inch pipelines from the WTP that feed the PRV are undersized to feed the Douglas PRV and Orchard Pump Station service areas. Improvement P-2 has been identified by the city as a potential future supply pipe alignment to replace the aging and difficult to access existing south supply line. The alignment shown is schematic and would need to be determined by assessing the topographic conditions and access easement availability.

Additional documentation concerning pipeline improvement recommendations to address existing and future deficiencies is presented later in this chapter.

Fire Flow

Fire flow demand is the amount of water required to fight a fire for a specified period of time. The Insurance Services Office, Inc. (ISO) classifies a city or municipality on a scale from 1 to 10 for insurance rating purposes on the basis of a maximum fire flow credit of 3,500 gpm flow from the water system, even though the ISO fire flow requirement may be more than 3,500 gpm. Part of this evaluation includes an analysis of the water supply system, specifically including fire flow availability and hydrant distribution. Fire flow requirements that are more than 3,500 gpm are evaluated individually and are not used by the ISO to determine the public protection classification of the municipality. Fire protection is not dependent on the water distribution system alone. Fire flows more than 3,500 gpm can be reduced with individual fire suppression systems, such as a sprinkler system or chemical system, and an alarm system, fire-resistant construction, onsite supply, and other methods.

Table 5-8 presents the recommended required fire flow durations according to the *Fire Protection Handbook* (National Fire Protection Association, 1997). Table 5-8 is used for determining storage requirements for fire suppression.

TABLE 5-8
Recommended Fire Flow Durations

Required Fire Flow (gpm)	Duration (hours)
2,500 or less	2
3,000 to 3,500	3
4,000 to 4,500	4
5,000 to 5,500	5

Source: National Fire Protection Association (1997).

The 14 locations from the ISO fire flow survey will be used in the water system model to analyze the fire flow capability of the Dallas Water System under 2008 and 2028 demand conditions. Table 5-9 shows the ISO fire flow analysis locations. In general, the overall water system will be evaluated for fire flow capability assuming fire flows of 3,500 gpm for commercial and industrial areas, and 1,500 gpm for residential areas under 2008 and 2028 demand conditions. All new construction should be designed to meet these fire flow requirements to the extent the existing infrastructure allows.

It is important to note that although fire flow calculations will be performed for all nodes in the model, most of the nodes are not fire hydrant locations and many are located on smaller pipes not designed to convey fire flows. As a result of smaller pipes and dead-ends, there will be many nodes with low fire flow availability scattered throughout the distribution system; however, most of these nodes are located in the vicinity of larger pipes with higher available fire flows where hydrants are located. The important part of this analysis will be locating areas where overall fire flow availability is inadequate and where nearby pipes with adequate fire flow are not available for fire protection.

TABLE 5-9
ISO Fire Flow Analysis

Location	ISO-Desired Fire Flow[1,2] (gpm)	Duration[3] (hours)
Orchard Drive and Reed Lane	1,000	2
Ellendale Avenue and Uglow Street	3,500	3
N. Levens Street and Harder Street	4,500	4
River Drive and Park Street	1,000	2
E. Ash Street and Mason Street	5,000	4
Birch Street and Uglow Street	3,000	3
Clay Street and Jefferson Street	2,000	2
Kings Valley Highway and Oakdale Avenue	500	2
Main Street and Academy Street	2,500	2
Main Street and Birch Street	2,000	2
LaCreole Drive South of School	5,000	4
West end of Maple Street	1,500	2
Monmouth Road and Godsey Road	1,250	2
Maplewood Drive and Oakwood Drive	1,500	2

[1] Required fire flows from 1994 ISO hydrant flow test data summary.

[2] Fire flow requirements that are more than 3,500 gpm are evaluated individually and are not used by the ISO to determine the public protection classification of a municipality. These numbers are presented for presentation purposes only and are not used to evaluate fire flow adequacy for the City of Dallas.

[3] Recommended required fire flow durations from the *Fire Protection Handbook* (National Fire Protection Association, 1997).

ISO = Insurance Services Office, Inc.

gpm = gallons per minute

The system was analyzed for fire flows under 2008 and 2028 MDD conditions. Fire flow criteria in the city were discussed previously in this section. The fire flows were analyzed assuming the water surface levels in the reservoirs were 5 feet below overflow elevation and that each pump station has all pumps on to produce the maximum available fire flow in each service area.

In addition to the 14 specific fire flow locations shown in Table 5-10 from the 1994 ISO survey, 76 additional locations were selected for fire flow availability analysis. These locations were selected such that an even distribution of locations was analyzed throughout the system. In addition, these locations were selected to represent actual fire hydrant locations (or the nearest intersection). For most of the Dallas Water System, fire flow availability has been analyzed at a node within a 3- to 4-block distance. For fire flow locations other than those identified by ISO, three flow rate thresholds have been identified. For residential locations a 1,500-gpm requirement has been identified, for multifamily a 2,500-gpm requirement, and for commercial/industrial a 3,500-gpm requirement was applied.

Table 5-10 presents the fire flow availability as calculated using the hydraulic model for the 90 modeled locations under 2008 and 2028 MDD conditions. Fire flow availability is calculated by the model, assuming a minimum residual pressure of 20 psi is maintained at each fire flow location. Figure 5-8 presents a graphical depiction of where adequate fire flows in the Dallas Water System under 2008 MDD conditions can be provided.

As shown in Figure 5-8 and Table 5-10, desired fire flows cannot be provided at several locations throughout Service Area 1. This is primarily a result of two factors: distance from sources, and undersized pipes. Several locations in the eastern portion of Service Level 1 are unable to meet fire flow requirements under 2008 conditions. The first problem is their relatively long distance from the source of water, in this case the Clay Street and Main Street reservoirs and the Ellendale PRV. The second issue is relatively small existing pipe diameter or incomplete pipeline grid. As noted in the domestic demand analysis, high pipe velocities and the resulting high frictional losses reduce the amount of fire flow available at 20 psi. By ensuring a contiguous and large-diameter pipeline grid, adequate fire flows can be provided to most locations in the system.

Figure 5-9 shows the same fire flow locations also under 2008 conditions with several pipe improvements. As shown in Table 5-10, significant improvements in fire flows can be achieved in the system. Each of these improvements is described in more detail later in this chapter. The purpose of the fire flow analysis was not to identify improvements for every hydrant location where flows could not be provided, but instead to provide a list of improvements that would increase the overall fire suppression capacity of the system. It should be emphasized that these are the same improvements that have been identified to improve low pressures within the system during 2008 and 2028 peak demands. The available fire flow values with and without improvements identified in Table 5-10 provide the city with the information needed to assess additional improvements for addressing specific locations where current fire flow criteria cannot be met.

TABLE 5-10
Fire Flow Availability

						Hydraulic Model Results			
						Year 2008 MDD		Year 2028 MDD	
No.	Location	Pressure Zone	Service Area	Node ID	ISO Desired Fire Flow [1] (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)
ISO Fire Flow Locations									
1	Ellendale Avenue and Uglow Street	1	Main	1-324	4,500	5,688	5,823	5,565	6,336
2	N. Levens Street and Harder Street	1	Main	1-1020	1,000	12,086	12,745	11,667	12,637
3	River Drive and Park Street	1	Main	1-876	5,000	11,432	11,834	11,118	11,645
4	E. Ash Street and Mason Street	1	Main	1-118	3,000	6,997	7,802	6,919	8,129
5	Birch Street and Uglow Street	1	Main	1-198	2,000	7,330	10,343	7,213	10,637
6	Clay Street and Jefferson Street	1	Main	1-366	500	15,361	17,297	14,874	17,223
7	Kings Valley Highway and Oakdale Avenue	1	Main	1-392	2,500	1,380	1,394	1,325	1,446
8	Main Street and Academy Street	1	Main	1-532	2,000	12,291	13,070	11,971	12,968
9	Main Street and Birch Street	1	Main	1-416	5,000	14,517	15,276	14,312	15,375
10	LaCreole Drive South of School	1	Main	1-88	1,250	4,519	4,678	4,870	5,396
11	Monmouth Road and Godsey Road	1	Main	1-172	1,500	4,607	5,484	4,642	6,367
12	Orchard Drive and Reed Lane	2	Orchard	2-670	3,500	1,578	2,295	1,605	1,179
13	West end of Maple Street	2	Maple	2-748	1,000	2,797	3,431	3,103	3,776
14	Maplewood Drove and Oakwood Drive	2	Elmwood	2-768	1,500	1,966	1,967	1,962	1,963
Additional Fire Flow Analysis Locations									
15	Edgewood Circle in Meadow Creek Mobile Home Park	1	Main	1-844	2,500	5,419	5,422	5,291	5,359
16	SW Sagebrush Court and SW Oregontrail Drive	1	Main	1-942	1,500	1,900	1,900	1,887	2,027
17	SW Applegate Trail Drive	1	Main	1-938	1,500	2,229	2,229	2,218	2,360
18	SW Solomon Court and SW Wyatt Street	1	Main	1-916	1,500	5,788	5,817	5,756	5,938
19	SW Marietta Lane and SW Wyatt Street	1	Main	1-886	1,500	4,028	4,038	4,007	4,169
20	SW Alexandra Drive and SW Augusto Drive	1	Main	1-922	1,500	3,833	3,842	3,811	4,000
21	SW Prince Place and SW Wyatt Street	1	Main	1-898	1,500	11,417	11,732	11,112	11,537
22	SW Bryson Street and SW Woodlawn Court	1	Main	1-854	1,500	4,504	4,519	4,470	4,669
23	SW Allgood Street and Hayter Street	1	Main	1-540	2,500	12,261	12,899	11,897	12,723

TABLE 5-10
Fire Flow Availability

						Hydraulic Model Results			
						Year 2008 MDD		Year 2028 MDD	
No.	Location	Pressure Zone	Service Area	Node ID	ISO Desired Fire Flow [1] (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)
24	SW Mill Street and SW Robinhood Drive	1	Main	1-484	1,500	4,265	4,303	4,215	4,402
25	SW Hunter - end of road	1	Main	1-460	1,500	1,254	1,258	1,245	1,276
26	SW Washington Street and SW Oregon Avenue	1	Main	1-456	1,500	3,312	3,333	3,271	3,384
27	SW 9th Street and SW Birch Street	1	Main	1-422	3,500	1,775	1,789	1,726	1,833
28	SW Bridlewood Drive and SW Alderwood Court	1	Main	1-960	1,500	1,358	1,364	1,328	1,385
29	SW Oakdale and SW Brown Street	1	Main	1-388	1,500	4,546	4,611	4,465	4,703
30	Church Street and SW Cherry Street	1	Main	1-396	1,500	7,373	7,527	7,296	7,647
31	SW Stump Street and SW Cherry Street	1	Main	1-404	1,500	4,405	4,465	4,334	4,557
32	SW Ellis Street and SW Ash Street	1	Main	1-354	1,500	5,659	5,746	5,590	5,849
33	Church Street and SW Ash Street	1	Main	1-348	1,500	8,796	9,078	8,664	9,227
34	SW Maple Street and SW Lewis Street	1	Main	1-434	2,500	4,213	4,468	4,138	4,563
35	SE Clay Street and SE Lyle Street	1	Main	1-362	2,500	11,675	13,634	11,443	13,725
36	Hayter Street and SW Clay Street	1	Main	1-374	1,500	16,664	18,043	16,115	17,936
37	SE Washington Street and SE Uglow Avenue	1	Main	1-222	3,500	7,508	8,178	7,367	8,344
38	SE Court Street and SE Lyle Street	1	Main	1-506	1,500	7,593	7,947	7,429	8,096
39	Mill Street and Jefferson Street	1	Main	1-472	3,500	8,409	8,755	8,261	8,907
40	Mill Street and Main Street	1	Main	1-474	3,500	12,997	13,928	12,560	13,828
41	SW Levons Street and Mill Street	1	Main	1-476	3,500	5,287	5,341	5,223	5,465
42	SE Academy Street - East of Jefferson Street	1	Main	1-528	1,500	1,684	1,692	1,665	1,722
43	Main Street and S Walnut Avenue	1	Main	1-310	3,500	6,266	6,351	6,151	6,503
44	SE Walnut Avenue and SE Walnut Circle	1	Main	1-296	1,500	2,594	2,610	2,545	2,676
45	SE Ironwood Avenue and SE Ironwood Court	1	Main	1-292	1,500	2,737	2,765	2,653	2,858
46	SE Davis Street and SE Hankel Street	1	Main	1-282	1,500	3,689	3,735	3,578	3,867
47	SE Hankel Street and SE Orchard Avenue	1	Main	1-314	3,500	3,941	3,989	3,831	4,139
48	SE Walnut Avenue and SE Cypress Avenue	1	Main	1-268	1,500	2,894	2,919	2,832	2,993
49	SE Cottonwood Lane and SE Deschutes Drive	1	Main	1-262	1,500	2,482	2,503	2,419	2,577
50	SE Alderson Drive and SE Juniper Avenue	1	Main	1-228	1,500	3,759	3,888	3,698	3,962

TABLE 5-10
Fire Flow Availability

						Hydraulic Model Results			
						Year 2008 MDD		Year 2028 MDD	
No.	Location	Pressure Zone	Service Area	Node ID	ISO Desired Fire Flow [1] (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)	Available Fire Flow [2] Without Improvements (gpm)	Available Fire Flow [2] With Improvements (gpm)
51	Ash Street and Howe Street	1	Main	1-202	3,500	7,896	9,333	7,787	9,633
52	Holman Avenue and Monmouth Cutoff	1	Main	1-178	3,500	6,138	7,641	6,101	8,012
53	SE Virginia Drive and SE Gordon Court	1	Main	1-160	1,500	2,214	2,260	2,190	2,380
54	SE Gregory Drive and SE Ana Avenue	1	Main	1-164	1,500	2,020	2,054	1,997	2,155
55	SE Goodsey Road and SE Brookside Avenue	1	Main	1-152	1,500	4,675	4,950	4,814	5,699
56	SE Miller Avenue – east of LaCreole Drive	1	Main	1-116	2,500	6,107	6,551	6,178	7,044
57	SE Miller Avenue and SE Camella Drive	1	Main	1-122	3,500	5,062	5,333	5,570	6,258
58	SE Greening Drive and SE Jonathon Avenue	1	Main	1-132	1,500	3,661	3,768	3,870	4,189
59	SE Appleseed Drive and SE Miller Avenue	1	Main	1-140	1,500	4,364	4,891	5,529	6,286
60	SE Miller Avenue and Fir Villa Road	1	Main	1-142	1,500	3,730	4,939	5,027	6,650
61	East Ellendale Avenue and Fir Villa Road	1	Main	1-22	3,500	3,114	3,276	4,419	4,887
62	East Ellendale Avenue and Orchard View Lane	1	Main	1-12	1,500	2,597	2,680	3,102	3,327
63	Laura Lane and Pleasant Drive	1	Main	1-16	1,500	2,059	2,105	2,296	2,444
64	SE Barberry Avenue and SE LaCreole Drive	1	Main	1-92	1,500	6,037	6,317	6,746	7,658
65	SE Barberry Avenue and SE Muir Drive	1	Main	1-98	1,500	4,703	4,853	5,972	6,694
66	SE Greenlee Drive and SE Academy Street	1	Main	1-78	1,500	4,149	4,270	4,816	5,320
67	Greenway Mobile Home Park – south end	1	Main	1-60	2,500	2,414	2,427	2,346	2,515
68	Greenway Mobile Home Park – north end	1	Main	1-48	2,500	2,019	2,038	1,966	2,108
69	SE LaCreole Drive and SE Hankel Street	1	Main	1-66	2,500	4,207	4,319	4,121	4,553
70	SE Stone Lane and SE Stone Street	1	Main	1-252	2,500	1,788	1,803	1,736	1,866
71	East Ellendale Avenue and LaCreole Drive	1	Main	1-38	3,500	4,323	4,488	4,839	5,514
72	East Ellendale Avenue and Hawthorne Avenue	1	Main	1-34	3,500	3,428	3,558	4,947	5,597
73	West Ellendale Avenue and SW Jasper Street	1	Main	1-596	3,500	6,946	6,994	6,765	7,283
74	West Ellendale Avenue and SW Westwood Drive	1	Main	1-830	3,500	1,826	2,896	1,726	2,360
75	NW Brentwood Avenue and NW Sunny Drive	1	Main	1-816	1,500	3,228	3,248	3,149	3,468
76	NW Alameda Street and NW Robert Street	1	Main	1-812	1,500	3,897	3,920	3,864	4,107

TABLE 5-10
Fire Flow Availability

						Hydraulic Model Results			
						Year 2008 MDD		Year 2028 MDD	
No.	**Location**	**Pressure Zone**	**Service Area**	**Node ID**	**ISO Desired Fire Flow [1] (gpm)**	**Available Fire Flow [2] Without Improvements (gpm)**	**Available Fire Flow [2] With Improvements (gpm)**	**Available Fire Flow [2] Without Improvements (gpm)**	**Available Fire Flow [2] With Improvements (gpm)**
77	NW Denton Avenue and NW Heath Street	1	Main	1-682	2,500	3,164	3,195	3,131	3,548
78	NW Jasper Street and NW Bonanza Avenue	1	Main	1-602	1,500	6,251	6,334	6,026	6,718
79	NW Elderberry Lane and NW Gavin Drive	1	Main	1-786	1,500	2,167	2,161	2,177	2,416
80	Kings Valley Highway and NE Polk Station Road	1	Main	1-332	2,500	2,158	2,178	3,260	3,660
81	NW Hillcrest Drive and NW Byers Lane	2	Douglas	2-734	1,500	723	2,933	659	1,491
82	NW Ashley Street and NW Eve Avenue	2	Douglas	2-742	1,500	950	2,028	1,096	410
83	NW Douglas Street and NW Denton Street	2	Douglas	2-726	1,500	759	4,471	684	2,209
84	NW Fairhaven and NW Heath Street	2	Orchard	2-714	1,500	1,451	3,499	1,491	1,171
85	NE Gerlinger Lane and NE Gerlinger Place	2	Orchard	2-664	1,500	1,861	3,251	1,773	1,397
86	NE Fern Avenue – east of NE Dallas Drive	2	Orchard	2-692	1,500	1,829	2,993	1,754	1,315
87	Church Street – at city limits	2	Church	2-576	1,500	494	494	497	498
88	SW Bridlewood Drive and SW Maplewood Drive	2	Elmwood	2-758	2,500	2,847	2,847	2,839	2,843
89	SW Oakwood Drive and SW Oakwood Court	2	Elmwood	2-764	1,500	1,406	1,407	1,402	1,403
90	SW Boxwood Lane and SW Boxwood Court	3	U. Bridlewood	3-776	1,500	1,426	1,426	1,446	1,446

[1] Fire flow requirements that are larger than 3,500 gpm are evaluated individually and are not used by the ISO in determining the public protection classification of a municipality. These numbers are for presentation purposes only and are not used to evaluate fire flow adequacy for the City of Dallas.

[2] Available fire flow is calculated assuming that a minimum residual pressure of 20 psi is maintained at the fire flow location.

gpm = gallons per minute

MDD = maximum day demand

ISO = Insurance Services Office, Inc.

psi = pounds per square inch

Water Transmission, Storage, and Distribution System Capital Improvement Plan

This section contains a summary of the water system capital improvement projects developed from the master plan analysis. The improvements were developed to serve existing 2008 and projected 2028 demands for the Dallas Water System. It should be noted that the level of detail provided in the capital improvement plan (CIP) is intended to supply a general description and sizing of the project along with an order of magnitude cost estimate. Project-specific details will need to be verified through the design process. Cost estimates are included in this section for budget planning purposes only.

Capital Improvement Plan Summary

The CIP included in this study was developed on the basis of the analysis completed in this master plan to serve existing 2008 and future 2028 demands. The improvements have been separated into two planning horizons: those occurring in the period starting in 2009 and ending in 2014, and those extending from 2015 to 2028. This differentiation is intended to identify those improvements required in the near-term and those that may be needed at a later date if water use increases as expected. Improvements identified in the first 5 years primarily will address existing deficiencies and conditions, such as areas of low pressure, fire flows, storage volume and pumping capacity.

Figure 5-10 illustrates the project locations for the pump station, pipeline, and storage improvements developed in the CIP. Table 5-11 provides a description of each project including the purpose of the project, the required timeframe, and a cost estimate.

Storage Improvements

The existing total system storage is adequate to meet the criteria as described previously in this section. The Bridlewood service area is deficient by 0.27 MG under existing conditions. Improvement R-2 from the previous 2002 Water Master Plan has been completed (Main Street Reservoir) and has increased the total system storage by 2 MG in Service Level 1. The 2002 Water Master Plan identified a pump station (S-1) as part of the R-2 Main Street Reservoir project. A 14-inch high-pressure fill line was constructed instead of the pump station to fill the Main Street Reservoir. The high-pressure fill line is connected to the 2 MG clearwell at the WTP. A looped 16-inch-diameter pipeline (P-3, P-4) also will be required to connect the Main Street Reservoir to existing piping. Deficiencies in the Upper Bridlewood Reservoir volume may be addressed through increased pumping capacity at the Bridlewood Pump Station. If the WTP reservoir continues to be operated as part of the chlorine contact process for water treatment, then the city may want to consider only a portion its 2-MG volume as meeting the city’s storage requirements and provide additional storage to compensate.

TABLE 5-11
Water Transmission, Storage, and Distribution System Capital Improvement Plan (Updated from 2002 Water Master Plan)

Category	Project No.	Project Description	Project Purpose	Capital Improvement Schedule and Project Cost Summary by Fiscal Year	
				Phase 1 2009-2014	Phase 2 2015-2028
Storage	R-3	Construct a 2- to 3-MG storage reservoir to serve Service Level 1	To accommodate for future growth. The cost summary includes potential land acquisition expenses and assumes a 3 MG steel reservoir. Reservoir to be filled from connection to 18" pipeline along West Ellendale Avenue prior to PRV.		$4,014,000
Pumping	S-2	Upgrade to the existing Upper Bridlewood Pump Station	Installation of at least one additional pump at the Upper Bridlewood Pump Station allowing PHD and fire flow requirements to be supplied.	$209,000	
Transmission And Distribution System	P-2	Approximately 8,000 feet of 24-inch-diameter pipe running from the Water Treatment Plant (WTP) to the main service area at the Clay Street Reservoirs	Provides redundancy to the system and allows for the potential decommissioning of the existing south supply line.	$2,290,000	
	P-3	2,500 feet of 16-inch-diameter pipe that will connect to the Main Street Reservoir to the southeast area of the City	This improvement will allow for peak and fire flow volumes to be served to the city by the Main Street Reservoir.		$691,000
	P-5	400 feet of 8-inch-diameter pipe installed between the Douglas Street service area and the Orchard Drive service area along Reed Lane	This improvement will combine the Douglas Street and Orchard Drive service areas into a single pressure zone served from the WTP reservoir.	$100,000	
	P-7	4,800 feet of 12-inch-diameter pipe installed between West Ellendale Avenue and Fairhaven Lane	This improvement will allow a second connection to the WTP reservoir service area, thereby completing a transmission loop north of West Ellendate Avenue. Additionally, this improvement will allow removal of the Orchard Drive Pump Station from service except as an emergency supply source.	$883,000	
	P-8	6,000 feet of 12-inch-diameter pipe installed in the proposed SE Fir Villa Road right-of-way extension. 2,100 feet of 12-inch diameter pipe installed in Clow Corner Road from the proposed SE Fir Villa Road intersection to SE Godsey Road.	This improvement will improve the distribution capacity of the southeast portion of the water system, allowing for future growth.		$1,326,000
	P-10	WTP flow meter bypass, upgrade to effluent flow meter and piping to accommodate P-2	This improvement will allow the existing flow meter vault to be bypassed, improve the accuracy of effluent flow readings, and prepare for the connection to the new 24" finished water pipeline (P-2)	$300,000	
Misc.	M-1	Small-diameter pipeline replacement program (see Table 5-12)	Incremental replacement of older small-diameter system piping. Budgets for the pipe replacement program are established by the Public Works Department.		

Note: This order of magnitude estimate is in April 2009 dollars and does not include escalation, financing, construction management or operations and maintenance costs. Costs based on 2002 Water Master Plan and escalated to 2009 at an annual rate of 7%.

Under 2028 conditions, total storage deficiency is projected to increase to 1.74 MG. Improvement R-3, a 2- to 3-MG steel reservoir, has been identified to provide the additional storage required based on 2028 conditions. A possible site in the Wyatt Node has been identified for the reservoir location. Accurate sizing and siting of this facility will depend on the extent and location of future system development. It should be noted that total required volume of storage planned for in the next 20 years should take into account any future plans for removing portions or all of the Clay Street Reservoirs from service.

Pump Station Improvements

Based on the pumping capacity analysis, improvements are required to the Orchard Drive and Upper Bridlewood (S-2) pump stations to provide adequate fire flows to their respective service areas. However, the Orchard Drive zone will have adequate fire fighting capacity after pipeline improvement P-7 has been constructed. This improvement will connect the Orchard Drive service area directly to the WTP reservoir. The existing Orchard Drive Pump Station capacity, in addition to water provided through the Douglas Street service area from the WTP reservoir, will be adequate for fire suppression flows. The Upper Bridlewood Pump Station improvement should be sized to serve projected 2028 demands, because little growth is expected in this service area and the useful life of the pumps will extend beyond 2028. It should be noted that pipe upgrades will be necessary in conjunction with the pump station improvements to effectively transfer MDD plus fire flow rates in most areas of the water system.

Pipeline Improvements

Several pipeline improvements have been identified that will strengthen the overall transmission capacity and fire fighting capability in the system. The projects listed are from the 2002 Water Master Plan. Projects from the 2002 Water Master Plan that have been completed, replaced, or eliminated are not described below.

Improvement P-2 is a 24-inch-diameter supply line from the WTP connecting to the Service Level 1 area near the Clay Street Reservoirs. The length of the pipeline is estimated to be 8,000 feet. It should be noted that the alignment shown in Figure 5-10 is schematic and is not intended to be used as a final route. The actual timing of P-2 may coincide with the abandonment of the existing southern supply line, which is a steel pipe that was constructed about 50 years ago.

Improvement P-3 includes about 2,500 feet of 16-inch-diameter pipe installed near the Main Street Reservoir and connecting to Uglow Street. A 16-inch-diameter pipe is warranted to allow fire flows of 3,500 gpm and larger to be delivered from this reservoir with low head losses.

Improvement P-7 is intended to be developer-financed. This improvement, about 4,800 feet of 12-inch-diameter pipe, will provide another connection to the WTP Reservoir service area and will strengthen the pipe grid north of Ellendale Avenue. Construction of this improvement will allow the city to remove the Orchard Drive Pump Station from service except as an emergency water source. Currently, the northwest area of the Orchard Drive Pump Station service area is experiencing low pressure (~ 25-30 psi) from high demands (refer to Figure 5-1). The low pressures make this project critical to complete within the next year prior to further development in the service area.

Improvement P-8, timed for construction with the extension of Fir Villa Road, will improve distribution capacity of the southeast portion of the water system, allowing for future growth. This improvement involves the installation of approximately 6,000 feet of 12-inch-diameter pipe within the proposed SE Fir Villa Road right-of-way extension, in addition to 2,100 feet of 12-inch-diameter water main within Clow Corner Road, replacing the existing 4-inch-diameter main.

Improvement P-10 includes improvements at the WTP associated with the effluent flow meter. The existing flow meter does not accurately read low flows and there is no bypass to allow repair of the flow meter and keep the 2 MG reservoir on line. Under this project, a bypass will be added and the flow meter replaced along with piping improvements for connecting the future 24-inch finished water pipeline (P-2).

Other pipeline improvements will be necessary to address all of the specific fire flow requirements in the city. The CIP outlined in this section is intended to provide the large improvements that allow the transfer of adequate quantities of water throughout the system. The city also has an ongoing main replacement program, noted as improvement M-1, which is budgeted for yearly with the goal of replacing about 1,500 feet of pipeline per year. Table 5-12 lists the main replacement projects that have been identified by the city.

TABLE 5-12
City Main Replacement Program

Location	Length (ft)	Type/Size	Proposed	Hydrants
Academy Ball Field to Deschutes	300	1 1/2” OD	8” DI	1
Arizona Street, Birchwood North	200	4” OD	4” DI	
Ash Street, Lyle to Shelton	350	5” OD	6” DI	Replace
Birchwood, Walnut to Needham	450	6” OD	6” DI	1
Stump Street, Ash to Oakdale	1,500	6” OD	6” DI	1
Godsey, Monmouth Cutoff south	300	2” ID	2” PVC	
Cherry to Oakdale	400	6” CI	8” DI	
Water Treatment Plant	900	Various	8” DI	

Cost Estimate Summary

The cost estimates developed for the proposed improvements are order-of-magnitude estimates and should be updated for specific project conditions when implementation is imminent. The cost estimates are based on CH2M HILL’s cost curves, database, and estimating resources, expressed in 2008 dollars. The curves are based on historical data and past projects and estimates completed by CH2M HILL. Detailed cost estimates should be developed during the design phase of the facilities.

Pump Station Costs

The cost estimate for the proposed pump station includes a building that houses the pumps, site work, piping, controls, pumps, and accessories. Land acquisition costs are included with the associated reservoir projects and not with the pump station because it is assumed that the pump stations will be sited on the same property as the reservoirs. Indirect costs were included in the estimate for administration, legal, engineering and a 30 percent contingency.

Pipeline Costs

Cost estimates for pipeline projects were based on using cement-lined DI pipe. The cost estimate included fittings, valves, and fire hydrants located about every 500 lineal feet. Imported backfill was assumed for the pipeline improvements. Asphalt surface restoration was assumed for pipelines in existing roads. Indirect costs were included in the estimate for administration, legal, engineering and a 30 percent contingency.

Storage Costs

The cost estimate for each reservoir was based on constructing a steel tank. The reservoir cost estimate includes site work, foundation, site piping, controls, privacy screening, miscellaneous appurtenances, and the tank itself. Indirect costs were included in the estimate for administration, legal, engineering and a 30 percent contingency. A land purchase allowance of $200,000 is assumed for each reservoir. Actual costs will vary based on market conditions and land value assessments.

Schedule

The CIP presented in Figure 5-10 and Table 5-11 shows individual projects, project priorities, project purpose, construction schedule, and estimated costs. The project priorities were assessed based on the modeling analysis and through discussions with City of Dallas staff. The improvements are prioritized to occur either in the 5 fiscal years from 2009 to 2014 (Phase 1), or the subsequent 15 years from 2015 to 2028 (Phase 2). The actual growth in demand should be monitored and available funding should be evaluated to verify the recommended implementation period of the improvements. Improvements that are dependent on new development should be constructed only when the developments actually occur or are imminent.

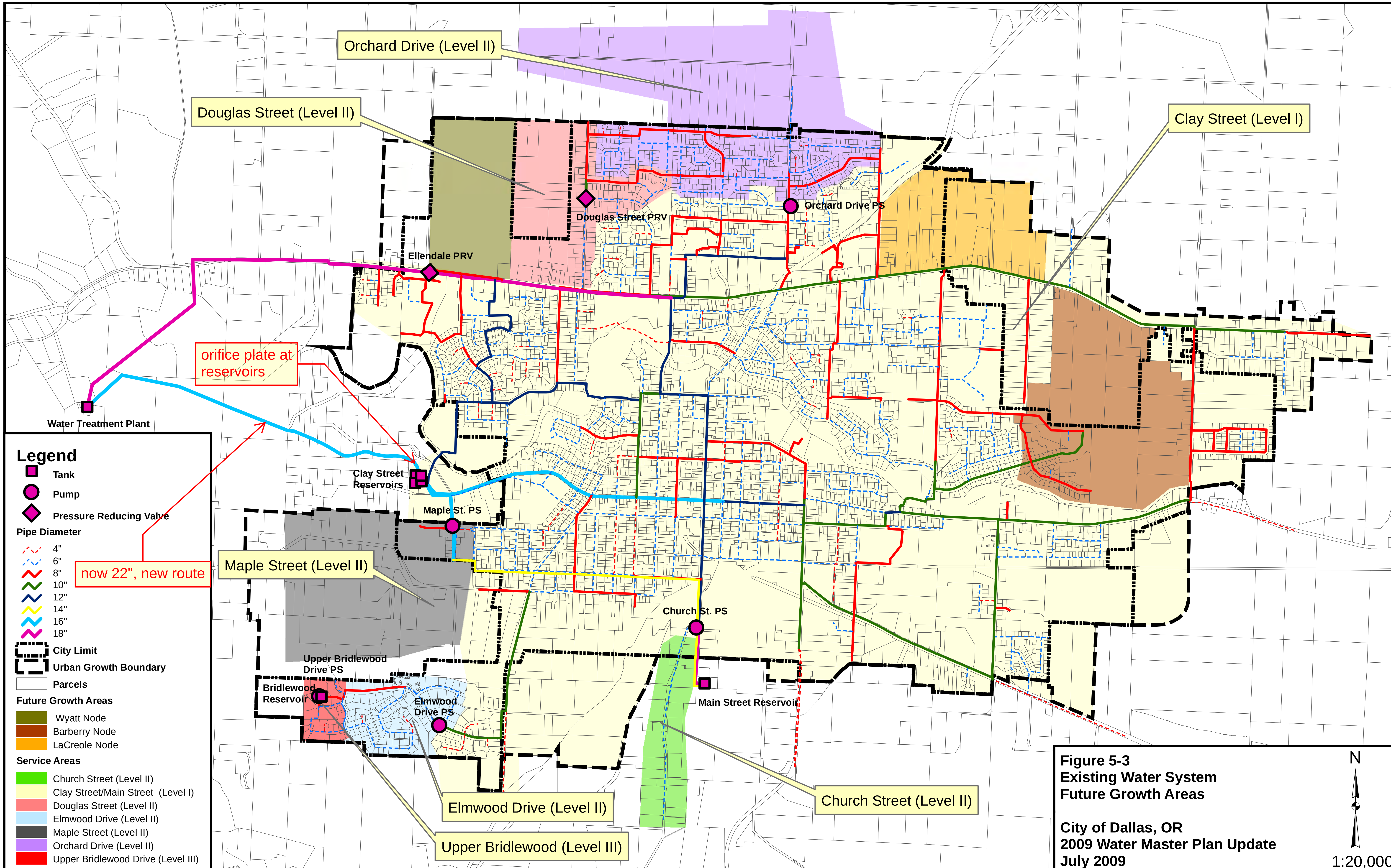


Figure 5-3
Existing Water System
Future Growth Areas

City of Dallas, OR
2009 Water Master Plan Update
July 2009

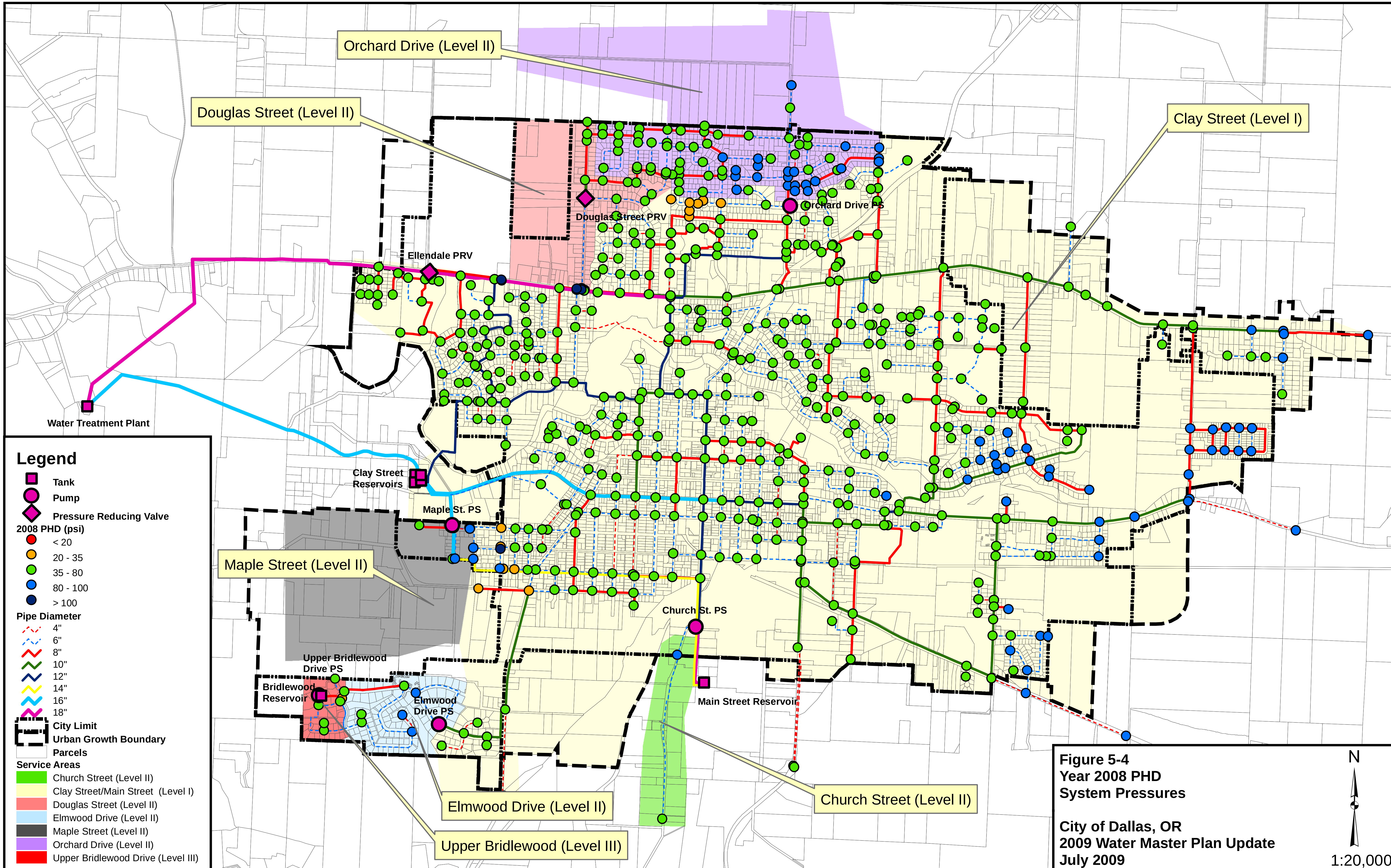


Figure 5-4
Year 2008 PHD
System Pressures

City of Dallas, OR
2009 Water Master Plan Update
July 2009

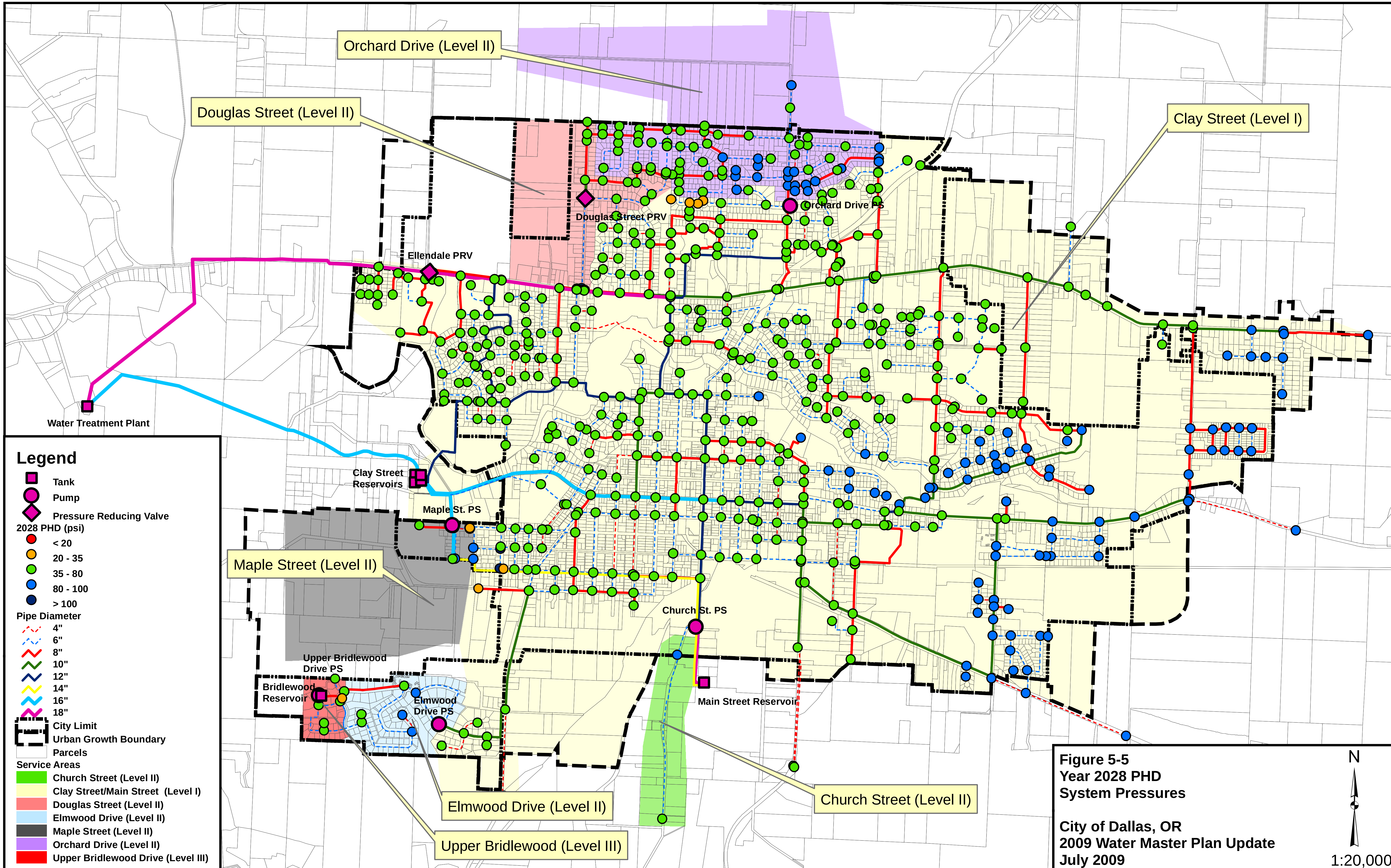


Figure 5-5
Year 2028 PHD
System Pressures

City of Dallas, OR
2009 Water Master Plan Update
July 2009

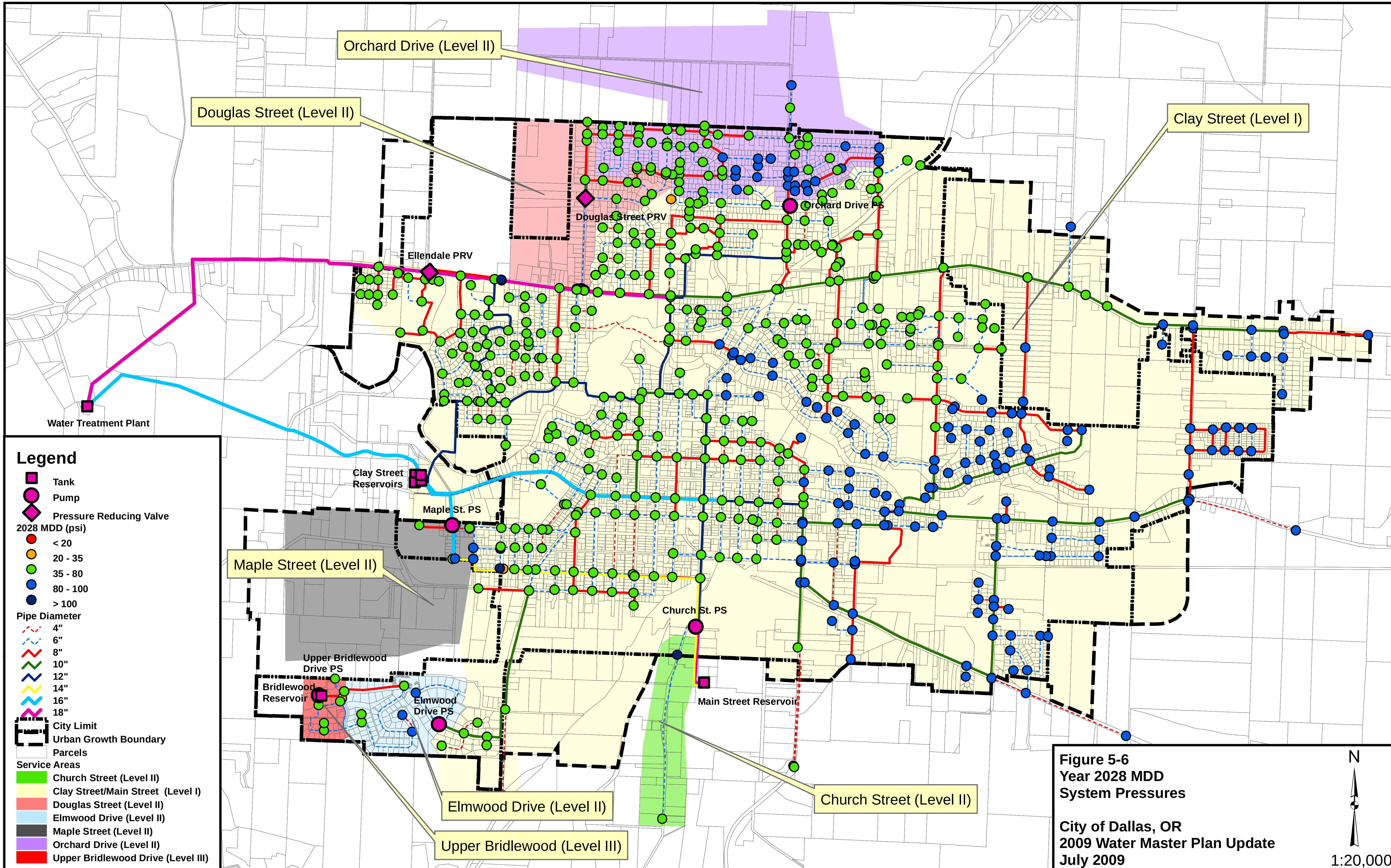


Figure 5-6
Year 2028 MDD
System Pressures

City of Dallas, OR
2009 Water Master Plan Update
July 2009

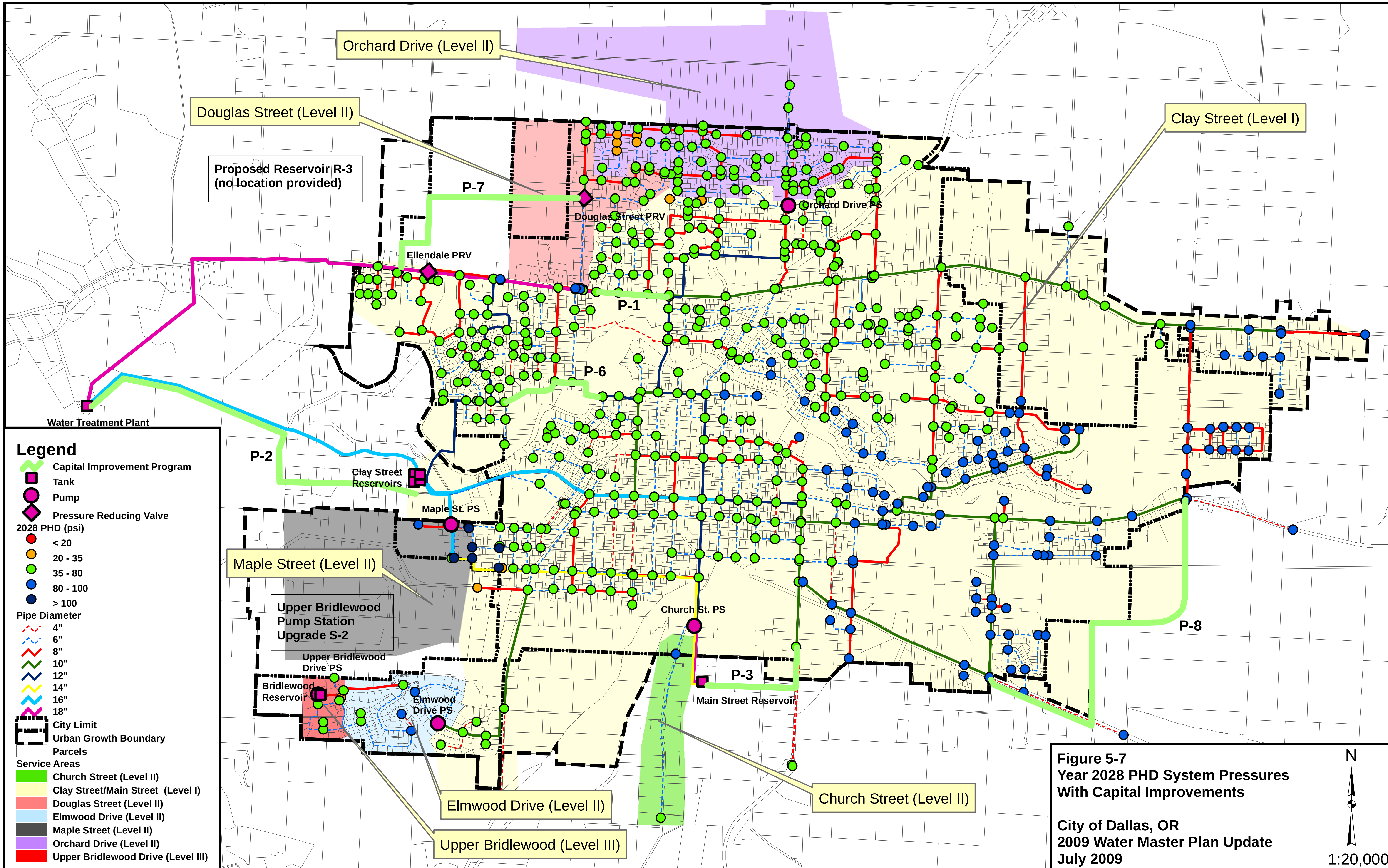


Figure 5-7
Year 2028 PHD System Pressures
With Capital Improvements

City of Dallas, OR
2009 Water Master Plan Update
July 2009

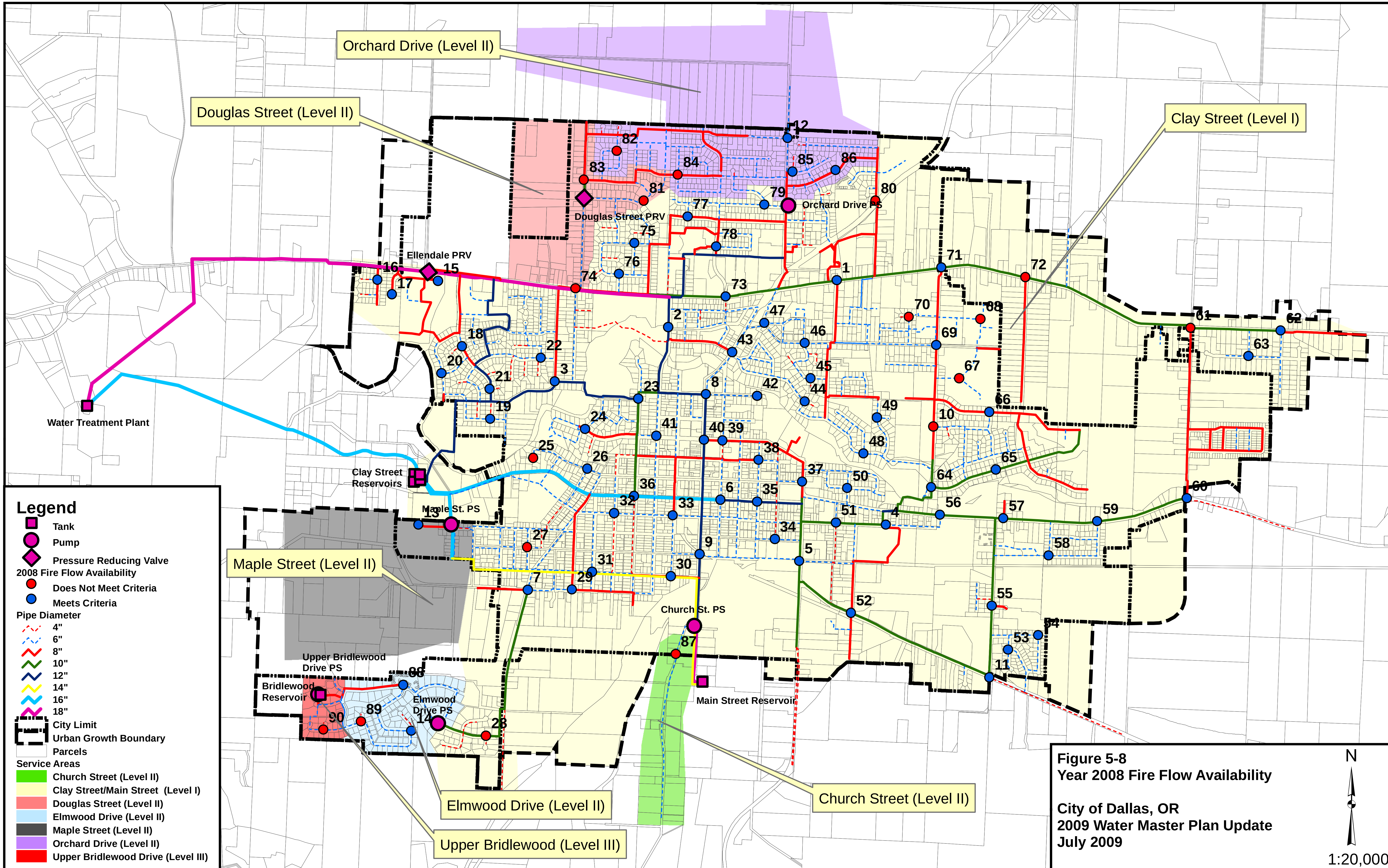
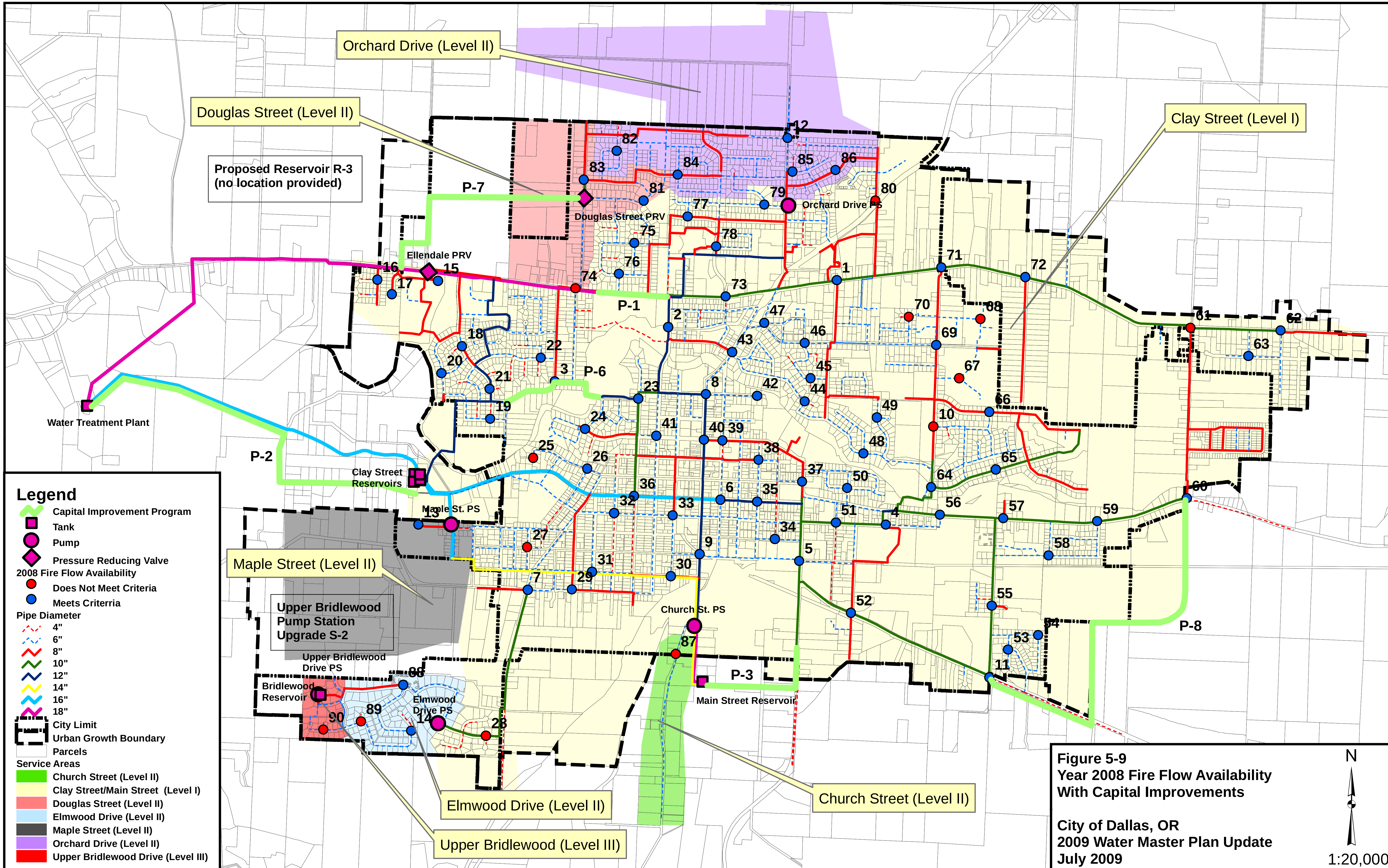


Figure 5-8
Year 2008 Fire Flow Availability

City of Dallas, OR
2009 Water Master Plan Update
July 2009



Orchard Drive (Level II)
Douglas Street (Level II)
Clay Street (Level I)
Proposed Reservoir R-3
(no location provided)
P-7
P-1
P-6
P-2
P-3
P-8
Douglas Street PRV
Orchard Drive PS
Ellendale PRV
Water Treatment Plant
Clay Street
Reservoirs
Maple St. PS
Maple Street (Level II)
Upper Bridlewood
Pump Station
Upgrade S-2
Upper Bridlewood
Drive PS
Bridlewood
Reservoir
Elmwood
Drive PS
Church St. PS
Main Street Reservoir
Elmwood Drive (Level II)
Upper Bridlewood (Level III)
Church Street (Level II)
Legend
Capital Improvement Program
Tank
Pump
Pressure Reducing Valve
2008 Fire Flow Availability
Does Not Meet Criteria
Meets Criteria
Pipe Diameter
4"
6"
8"
10"
12"
14"
16"
18"
City Limit
Urban Growth Boundary
Parcels
Service Areas
Church Street (Level II)
Clay Street/Main Street (Level I)
Douglas Street (Level II)
Elmwood Drive (Level II)
Maple Street (Level II)
Orchard Drive (Level II)
Upper Bridlewood Drive (Level III)
Figure 5-9
Year 2008 Fire Flow Availability
With Capital Improvements
City of Dallas, OR
2009 Water Master Plan Update
July 2009
N
1:20,000

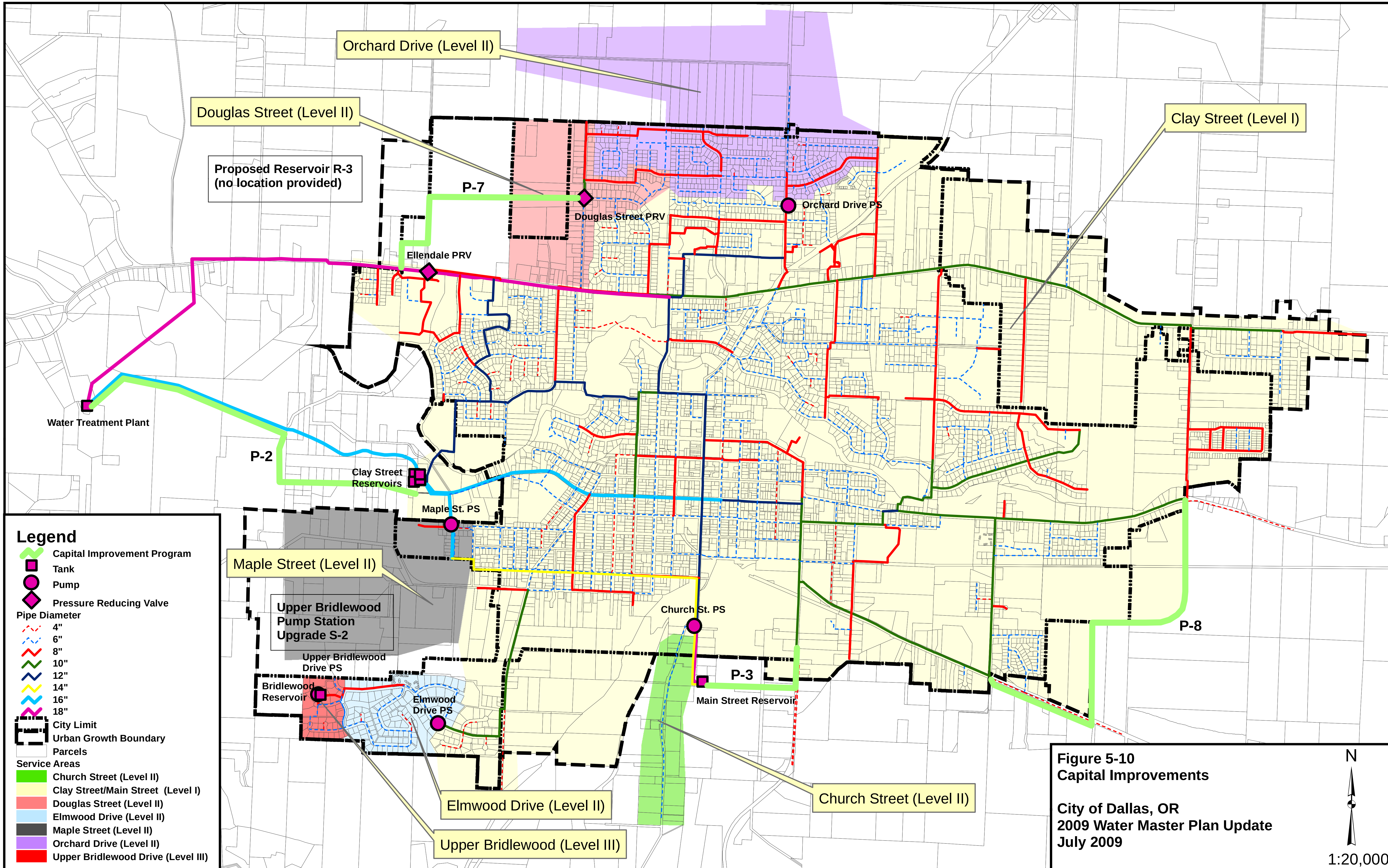


Figure 5-10
Capital Improvements

City of Dallas, OR
2009 Water Master Plan Update
July 2009

APPENDIX A

Population Projections for Dallas City, Independence City, Monmouth City, and Unincorporated Areas of Polk County, Oregon: 2000 to 2100

**Population Projections for
Dallas City,
Independence City,
Monmouth City,
and Unincorporated Areas of
Polk County, Oregon:
2000 to 2100**

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Portland State University
Portland, Oregon 97207

October 15, 2001

Introduction

This report presents population projections for the cities of Dallas, Independence, and Monmouth, Oregon and for the unincorporated areas of Polk County, Oregon for the period 2000 to 2100. The projections are benchmarked to 2000 census results for the city and county populations.

Policy makers should view population projections as one of several available sources of information about likely future conditions. The forecasts in this report are based on assumptions developed from analysis of historical trends and expectations of the future. While the past gives some indication of what is likely to happen in the future, there always the possibility of unforeseen events that could have a significant impact of population change. Thus, users of these projections should be aware that new changes could occur and that it is wise to evaluate projections periodically in future years.

Given that these projections are developed for long-term trends, they are conservative. This means that they do not assume drastic changes to the population trends that have developed over the past thirty years.

Methods and Assumptions

This report relies on the ratio method approach for projecting city population size. It provides a projection for the total city population, by five years intervals, from 2000 to 2100.

The basic idea of the ratio method is that local city populations are under the same influences of change as the surrounding county population. In particular, we assume here that the influences of population change (fertility, mortality, and migration) are similar in Dallas, Independence, and Monmouth cities and surrounding Polk County. So, rather than make detailed assumptions about local mortality, fertility, and migration levels for the city populations, we can presume a link between population changes in Polk County and the cities located in Polk County.

We base the forecasts on two assumptions.

- First, we rely on the population forecasts for Polk County, from 2000 to 2040, that have been prepared by Oregon's Office of Economic Analysis (OEA). OEA's forecast assumes that annual population growth for the county diminishes from its current level of 2.00 percent (for the 1995 to 2000 period) to 0.86 percent in 2035-2040. The OEA forecast is based on analysis of population trends in the county for the past thirty years. We extrapolate the OEA forecast by assuming that Polk County's population continues to increase at an annual rate of 0.85 percent from 2040 to 2100.

- Second, we note that the proportion of Polk County's population that resides in the three cities has changed over time.

  The City of Dallas has increased from 17.99 percent in 1970, to 18.87 percent in 1980, to 18.81 percent in 1990, and to 19.97 percent in 2000. Based on a fitted trend to the 1970-2000 data, we assume that there is linear increase in the proportion of the county's population residing in Dallas, and that the percent reaches 22.38 percent in 2050 and

24.95 percent in 2100. We assume that this increase will result from increasing urbanization, with moderately faster population growth in Dallas than in the surrounding county, and from city annexations.

The City of Independence has increased from 7.34 percent in 1970, to 8.90 percent in 1980, to 8.83 percent in 1990, and to 9.66 percent in 2000. Based on a fitted trend to the 1970-2000 data, we assume that there is linear increase in the proportion of the county's population residing in Independence, and that the percent reaches 12.36 percent in 2050 and 15.17 percent in 2100. We assume that this increase will result from increasing urbanization, with moderately faster population growth in Independence than in the surrounding county, and from city annexations.

The City of Monmouth has decreased slightly from 14.82 percent in 1970, to 12.38 percent in 1980, to 12.55 percent in 1990, and to 12.41 percent in 2000. Based on a fitted trend to the 1970-2000 data, we assume that there is linear decrease in the proportion of the county's population residing in Monmouth, and that the percent reaches 9.83 percent in 2050 and 7.25 percent in 2100. We assume that these decreases will result from increasing urbanization, but with slightly slower population growth in Monmouth than in the surrounding county.

We combine the two assumptions above, multiplying the forecast for Polk County times the forecast for the proportion of the population residing in each city to obtain a forecast for the city population. We call this the "medium" population forecast.

In order to take into account variation in the two assumptions above, we make further assumptions about the low and high ranges that they might take. For the low assumptions, we assume that (a) annual population growth in Polk County is 15 percent slower than forecast by OEA and (b) the increase in the percentage of Polk County that resides in each city is at a 10 percent slower rate. For the high assumptions, we assume that (a) population growth in Polk County is 15 percent faster than forecast by OEA and (b) the increase in the percentage of Polk County that resides in each city is at a 10 percent faster rate. Based on experience in preparing city population forecasts, I believe that these are reasonable assumptions to bracket the range of possible future population growth.

Results

Table 1 shows the overall results for population growth in the cities of Dallas, Independence, and Monmouth, as well as for the three cities combined. For the three cities combined, the population increased from 14,192 in 1970 to 26,225 in 2000. Continued future population growth is likely, with the population almost doubling to 47,594 in 2050, and to 77,391 in 2100 under the medium growth assumptions. For the lower growth assumptions, the population in the three cities combined is 58,581 in 2100. For the higher growth assumptions, the population in the three cities combined is 101,543 in 2100.

As discussed later, there is likely to be modest population growth in unincorporated areas of Polk County, primarily because more rapidly growth areas near urban areas will be annexed into the incorporated cities of the county. As population growth occurs in the unincorporated areas outside the three cities, the city population projections assume that this growth will be included through the process of annexation.

Dallas City

Table 2 presents results for population change in Dallas for 1970 to 2100. The population figures for 1970 to 2000 are based on census data. The figures for 2000 to 2100 are forecasted values. The medium values for Dallas assume (a) the county population projections from OEA for 2000 to 2040 and our extrapolated values for 2040 to 2100 and (b) a continuation in current trends for an increasing proportion of the county population residing in Dallas. Table 2 shows figures for Polk County and the proportion of the county residing in Dallas that are used to produce the population forecasts.

- For the medium projection, Dallas is expected to almost double from 12,459 in 2000 to 23,901 in 2050. Growth is expected to be more moderate in the 2050 to 2100 period, with a further increase to 40,763 in 2100.
- For the low projection, Dallas is forecasted to increase from 12,459 in 2000 to 20,697 in 2050 and 31,873 in 2100. Even in the low projection, there will be substantial population increase over the next century.
- For the high projection, Dallas is expected to more than double from 12,459 in 2000 to 27,566 in 2050, and then almost double again to 51,992 in 2100. This projection assumes that Polk County increases by more than 1 percent a year over the next century and that Dallas increases its share of the county’s population to more than 26 percent by 2100.

Independence City

Table 3 presents results for population change in Independence for 1970 to 2100. The population figures for 1970 to 2000 are based on census data. The figures for 2000 to 2100 are forecasted values. The medium values for Independence assume (a) the county population projections from OEA for 2000 to 2040 and our extrapolated values for 2040 to 2100 and (b) a continuation in current trends for an increasing proportion of the county population residing in Independence. Table 3 shows figures for Polk County and the proportion of the county residing in Independence that are used to produce the population forecasts.

- For the medium projection, Independence is expected to more than double from 6,025 in 2000 to 13,196 in 2050. Growth is expected to be more moderate in the 2050 to 2100 period, with a further increase to 24,786 in 2100.
- For the low projection, Independence is forecasted to increase from 6,025 in 2000 to 11,083 in 2050 and 18,694 in 2100. Even in the low projection, there will be substantial population increase over the next century.
- For the high projection, Independence is expected to more than double from 6,025 in 2000 to 15,645 in 2050, and then almost double again to 32,623 in 2100. This projection assumes that Polk County increases by more than 1 percent a year over the next century and that Independence increases its share of the county’s population to more than 16 percent by 2100.

Monmouth City

Table 4 presents results for population change in Monmouth for 1970 to 2100. The population figures for 1970 to 2000 are based on census data. The figures for 2000 to 2100 are forecasted values. The medium values for Monmouth assume (a) the county population projections from OEA for 2000 to 2040 and our extrapolated values for 2040 to 2100 and (b) a continuation in current trends for an increasing proportion of the county population residing in Monmouth. Table 4 shows figures for Polk County and the proportion of the county residing in Monmouth that are used to produce the population forecasts.

- For the medium projection, Monmouth is expected to increase from 7,741 in 2000 to 10,497 in 2050. Growth is expected to be more moderate in the 2050 to 2100 period, with a further increase to 11,842 in 2100.
- For the low projection, Monmouth is forecasted to increase modestly from 7,741 in 2000 to 8,658 in 2050 and then decrease slightly to 8,014 in 2100. In the low projection assumptions, there will be little overall population change over the next century.
- For the high projection, Monmouth is expected to increase moderately from 7,741 in 2000 to 12,638 in 2050, and then increase to 16,928 in 2100. This projection assumes that Polk County increases by more than 1 percent a year over the next century and that Monmouth decreases its share of the county's population to about 8.6 percent by 2100.

Unincorporated Areas of Polk County

Table 5 presents results for population change in unincorporated areas of Polk County for 1970 to 2100. The population figures for 1970 to 2000 are based on census data. The figures for 2000 to 2100 are forecasted values. The medium values for Polk County's unincorporated areas assume (a) the county population projections from OEA for 2000 to 2040 and our extrapolated values for 2040 to 2100 and (b) a continuation in current trends for a decreasing proportion of the county population residing in unincorporated areas. Table 5 shows figures for Polk County and the proportion of the county residing in unincorporated areas that are used to produce the population forecasts.

- For the medium projection, the population in unincorporated areas is expected to increase from 16,610 in 2000 to 20,747 in 2050. Growth is expected to be more moderate in the 2050 to 2100 period, with slight decreases to 19,973 in 2100.
- For the low projection, the population in unincorporated areas is forecasted to increase modestly from 16,610 in 2000 to 17,864 in 2050 and then decrease slightly to 14,723 in 2100. In the low projection assumptions, there will be moderate overall population declines over the next century.
- For the high projection, the population in unincorporated areas is expected to increase moderately from 16,610 in 2000 to 24,053 in 2050, and then increase to 26,788 in 2100. This projection assumes that Polk County increases by more than 1 percent a year over the next century and that the population in unincorporated areas decreases its share of the county's population to about 13.5 percent by 2100.

TABLE 1
Population Projection for Dallas City, Independence City, and Monmouth City, and Combined Population of the Three Cities, Polk County, Oregon: Observed Population from 1970 to 2000; Projected Population from 2000 to 2100

	Low Growth Assumptions				Medium Growth Assumptions				High Growth Assumptions			
Year	Dallas	Independence	Monmouth	Combined Population	Dallas	Independence	Monmouth	Combined Population	Dallas	Independence	Monmouth	Combined Population
1970	6,361	2,594	5,237	14,192	6,361	2,594	5,237	14,192	6,361	2,594	5,237	14,192
1975	7,366	3,231	5,413	16,010	7,366	3,231	5,413	16,010	7,366	3,231	5,413	16,010
1980	8,530	4,024	5,594	18,148	8,530	4,024	5,594	18,148	8,530	4,024	5,594	18,148
1985	8,965	4,230	5,931	19,126	8,965	4,230	5,931	19,126	8,965	4,230	5,931	19,126
1990	9,422	4,425	6,288	20,135	9,422	4,425	6,288	20,135	9,422	4,425	6,288	20,135
1995	10,850	4,875	7,225	22,950	10,850	4,875	7,225	22,950	10,850	4,875	7,225	22,950
2000	12,459	6,025	7,741	26,225	12,459	6,025	7,741	26,225	12,459	6,025	7,741	26,225
2005	12,993	6,242	7,777	27,012	13,407	6,562	8,119	28,089	13,830	6,890	8,468	29,188
2010	13,830	6,736	7,978	28,543	14,490	7,203	8,480	30,173	15,175	7,691	9,002	31,868
2015	14,700	7,255	8,167	30,122	15,632	7,888	8,839	32,359	16,616	8,561	9,548	34,725
2020	15,580	7,788	8,332	31,701	16,805	8,602	9,176	34,583	18,115	9,480	10,083	37,678
2025	16,456	8,329	8,465	33,249	17,986	9,335	9,481	36,802	19,645	10,436	10,591	40,672
2030	17,309	8,866	8,558	34,733	19,151	10,073	9,742	38,966	21,173	11,410	11,053	43,636
2035	18,145	9,403	8,616	36,165	20,305	10,817	9,964	41,087	22,702	12,404	11,476	46,583
2040	18,965	9,940	8,641	37,546	21,449	11,568	10,148	43,164	24,233	13,416	11,860	49,508
2045	19,813	10,498	8,654	38,966	22,643	12,359	10,324	45,326	25,848	14,493	12,245	52,586
2050	20,697	11,083	8,658	40,439	23,901	13,196	10,497	47,594	27,566	15,645	12,638	55,850
2055	21,619	11,697	8,652	41,968	25,226	14,083	10,665	49,974	29,393	16,879	13,039	59,311
2060	22,579	12,339	8,635	43,554	26,620	15,022	10,828	52,470	31,335	18,199	13,448	62,982
2065	23,580	13,012	8,607	45,199	28,088	16,016	10,986	55,089	33,401	19,611	13,863	66,875
2070	24,624	13,716	8,566	46,906	29,633	17,068	11,136	57,837	35,597	21,121	14,285	71,003
2075	25,711	14,454	8,512	48,678	31,260	18,181	11,279	60,719	37,931	22,735	14,714	75,380
2080	26,844	15,227	8,444	50,515	32,972	19,358	11,413	63,743	40,412	24,460	15,147	80,020
2085	28,025	16,035	8,362	52,422	34,773	20,604	11,537	66,915	43,049	26,304	15,587	84,939
2090	29,255	16,882	8,263	54,400	36,669	21,921	11,651	70,241	45,851	28,273	16,030	90,154
2095	30,537	17,767	8,148	56,452	38,664	23,314	11,753	73,731	48,828	30,377	16,477	95,683
2100	31,873	18,694	8,014	58,581	40,763	24,786	11,842	77,391	51,992	32,623	16,928	101,543

TABLE 2
Population Projection for Dallas City, Polk County, Oregon: Observed Population from 1970 to 2000; Projected Population from 2000 to 2100

	Polk County						Dallas City					
	Low		Medium		High		Low		Medium		High	
Year	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	%Dallas of Polk	Pop.No.	%Dallas of Polk	Pop.No.	%Dallas of Polk	Pop.No.
1970	35,349	---	35,349	---	35,349	---		6,361	17.99%	6,361		6,361
1975	39,700	2.32%	39,700	2.32%	39,700	2.32%		7,366	18.55%	7,366		7,366
1980	45,203	2.60%	45,203	2.60%	45,203	2.60%		8,530	18.87%	8,530		8,530
1985	45,231	0.01%	45,231	0.01%	45,231	0.01%		8,965	19.82%	8,965		8,965
1990	50,088	2.04%	50,088	2.04%	50,088	2.04%		9,422	18.81%	9,422		9,422
1995	56,450	2.39%	56,450	2.39%	56,450	2.39%		10,850	19.22%	10,850		10,850
2000	62,380	2.00%	62,380	2.00%	62,380	2.00%		12,459	19.97%	12,459		12,459
2005	65,907	1.10%	66,819	1.37%	67,744	1.65%	19.71%	12,993	20.06%	13,407	20.41%	13,830
2010	69,420	1.04%	71,301	1.30%	73,232	1.56%	19.92%	13,830	20.32%	14,490	20.72%	15,175
2015	73,028	1.01%	75,963	1.27%	79,015	1.52%	20.13%	14,700	20.58%	15,632	21.03%	16,616
2020	76,611	0.96%	80,649	1.20%	84,901	1.44%	20.34%	15,580	20.84%	16,805	21.34%	18,115
2025	80,100	0.89%	85,266	1.11%	90,766	1.34%	20.54%	16,456	21.09%	17,986	21.64%	19,645
2030	83,411	0.81%	89,695	1.01%	96,453	1.22%	20.75%	17,309	21.35%	19,151	21.95%	21,173
2035	86,576	0.74%	93,969	0.93%	101,994	1.12%	20.96%	18,145	21.61%	20,305	22.26%	22,702
2040	89,601	0.69%	98,091	0.86%	107,385	1.03%	21.17%	18,965	21.87%	21,449	22.57%	24,233
2045	92,700	0.68%	102,350	0.85%	113,004	1.02%	21.37%	19,813	22.12%	22,643	22.87%	25,848
2050	95,906	0.68%	106,793	0.85%	118,917	1.02%	21.58%	20,697	22.38%	23,901	23.18%	27,566
2055	99,222	0.68%	111,430	0.85%	125,139	1.02%	21.79%	21,619	22.64%	25,226	23.49%	29,393
2060	102,654	0.68%	116,268	0.85%	131,687	1.02%	22.00%	22,579	22.90%	26,620	23.80%	31,335
2065	106,204	0.68%	121,315	0.85%	138,577	1.02%	22.20%	23,580	23.15%	28,088	24.10%	33,401
2070	109,877	0.68%	126,583	0.85%	145,827	1.02%	22.41%	24,624	23.41%	29,633	24.41%	35,597
2075	113,677	0.68%	132,078	0.85%	153,458	1.02%	22.62%	25,711	23.67%	31,260	24.72%	37,931
2080	117,609	0.68%	137,813	0.85%	161,487	1.02%	22.82%	26,844	23.92%	32,972	25.02%	40,412
2085	121,676	0.68%	143,796	0.85%	169,936	1.02%	23.03%	28,025	24.18%	34,773	25.33%	43,049
2090	125,884	0.68%	150,039	0.85%	178,828	1.02%	23.24%	29,255	24.44%	36,669	25.64%	45,851
2095	130,238	0.68%	156,553	0.85%	188,185	1.02%	23.45%	30,537	24.70%	38,664	25.95%	48,828
2100	134,742	0.68%	163,350	0.85%	198,031	1.02%	23.65%	31,873	24.95%	40,763	26.25%	51,992

TABLE 3
Population Projection for Independence City, Polk County, Oregon: Observed Population from 1970 to 2000; Projected Population from 2000 to 2010

	Polk County						Independence City					
	Low		Medium		High		Low		Medium		High	
Year	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	%Independence of Polk	Pop.No.	%Independence of Polk	Pop.No.	%Independence of Polk	Pop.No.
1970	35,349	---	35,349	---	35,349	---		2,594	7.34%	2,594		2,594
1975	39,700	2.32%	39,700	2.32%	39,700	2.32%		3,231	8.14%	3,231		3,231
1980	45,203	2.60%	45,203	2.60%	45,203	2.60%		4,024	8.90%	4,024		4,024
1985	45,231	0.01%	45,231	0.01%	45,231	0.01%		4,230	9.35%	4,230		4,230
1990	50,088	2.04%	50,088	2.04%	50,088	2.04%		4,425	8.83%	4,425		4,425
1995	56,450	2.39%	56,450	2.39%	56,450	2.39%		4,875	8.64%	4,875		4,875
2000	62,380	2.00%	62,380	2.00%	62,380	2.00%		6,025	9.66%	6,025		6,025
2005	65,907	1.10%	66,819	1.37%	67,744	1.65%	9.47%	6,242	9.82%	6,562	10.17%	6,890
2010	69,420	1.04%	71,301	1.30%	73,232	1.56%	9.70%	6,736	10.10%	7,203	10.50%	7,691
2015	73,028	1.01%	75,963	1.27%	79,015	1.52%	9.93%	7,255	10.38%	7,888	10.83%	8,561
2020	76,611	0.96%	80,649	1.20%	84,901	1.44%	10.17%	7,788	10.67%	8,602	11.17%	9,480
2025	80,100	0.89%	85,266	1.11%	90,766	1.34%	10.40%	8,329	10.95%	9,335	11.50%	10,436
2030	83,411	0.81%	89,695	1.01%	96,453	1.22%	10.63%	8,866	11.23%	10,073	11.83%	11,410
2035	86,576	0.74%	93,969	0.93%	101,994	1.12%	10.86%	9,403	11.51%	10,817	12.16%	12,404
2040	89,601	0.69%	98,091	0.86%	107,385	1.03%	11.09%	9,940	11.79%	11,568	12.49%	13,416
2045	92,700	0.68%	102,350	0.85%	113,004	1.02%	11.32%	10,498	12.07%	12,359	12.82%	14,493
2050	95,906	0.68%	106,793	0.85%	118,917	1.02%	11.56%	11,083	12.36%	13,196	13.16%	15,645
2055	99,222	0.68%	111,430	0.85%	125,139	1.02%	11.79%	11,697	12.64%	14,083	13.49%	16,879
2060	102,654	0.68%	116,268	0.85%	131,687	1.02%	12.02%	12,339	12.92%	15,022	13.82%	18,199
2065	106,204	0.68%	121,315	0.85%	138,577	1.02%	12.25%	13,012	13.20%	16,016	14.15%	19,611
2070	109,877	0.68%	126,583	0.85%	145,827	1.02%	12.48%	13,716	13.48%	17,068	14.48%	21,121
2075	113,677	0.68%	132,078	0.85%	153,458	1.02%	12.72%	14,454	13.77%	18,181	14.82%	22,735
2080	117,609	0.68%	137,813	0.85%	161,487	1.02%	12.95%	15,227	14.05%	19,358	15.15%	24,460
2085	121,676	0.68%	143,796	0.85%	169,936	1.02%	13.18%	16,035	14.33%	20,604	15.48%	26,304
2090	125,884	0.68%	150,039	0.85%	178,828	1.02%	13.41%	16,882	14.61%	21,921	15.81%	28,273
2095	130,238	0.68%	156,553	0.85%	188,185	1.02%	13.64%	17,767	14.89%	23,314	16.14%	30,377
2100	134,742	0.68%	163,350	0.85%	198,031	1.02%	13.87%	18,694	15.17%	24,786	16.47%	32,623

TABLE 4
Population Projection for Monmouth City, Polk County, Oregon: Observed Population from 1970 to 2000; Projected Population from 2000 to 2100

	Polk County											
	Low		Medium		High		Low		Medium		High	
Year	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	%Monmouth of Polk	Pop.No.	%Monmouth of Polk	Pop.No.	%Monmouth of Polk	Pop.No.
1970	35,349	---	35,349	---	35,349	---		5,237	14.82%	5,237		5,237
1975	39,700	2.32%	39,700	2.32%	39,700	2.32%		5,413	13.63%	5,413		5,413
1980	45,203	2.60%	45,203	2.60%	45,203	2.60%		5,594	12.38%	5,594		5,594
1985	45,231	0.01%	45,231	0.01%	45,231	0.01%		5,931	13.11%	5,931		5,931
1990	50,088	2.04%	50,088	2.04%	50,088	2.04%		6,288	12.55%	6,288		6,288
1995	56,450	2.39%	56,450	2.39%	56,450	2.39%		7,225	12.80%	7,225		7,225
2000	62,380	2.00%	62,380	2.00%	62,380	2.00%		7,741	12.41%	7,741		7,741
2005	65,907	1.10%	66,819	1.37%	67,744	1.65%	11.80%	7,777	12.15%	8,119	12.50%	8,468
2010	69,420	1.04%	71,301	1.30%	73,232	1.56%	11.49%	7,978	11.89%	8,480	12.29%	9,002
2015	73,028	1.01%	75,963	1.27%	79,015	1.52%	11.18%	8,167	11.64%	8,839	12.08%	9,548
2020	76,611	0.96%	80,649	1.20%	84,901	1.44%	10.88%	8,332	11.38%	9,176	11.88%	10,083
2025	80,100	0.89%	85,266	1.11%	90,766	1.34%	10.57%	8,465	11.12%	9,481	11.67%	10,591
2030	83,411	0.81%	89,695	1.01%	96,453	1.22%	10.26%	8,558	10.86%	9,742	11.46%	11,053
2035	86,576	0.74%	93,969	0.93%	101,994	1.12%	9.95%	8,616	10.60%	9,964	11.25%	11,476
2040	89,601	0.69%	98,091	0.86%	107,385	1.03%	9.64%	8,641	10.35%	10,148	11.04%	11,860
2045	92,700	0.68%	102,350	0.85%	113,004	1.02%	9.34%	8,654	10.09%	10,324	10.84%	12,245
2050	95,906	0.68%	106,793	0.85%	118,917	1.02%	9.03%	8,658	9.83%	10,497	10.63%	12,638
2055	99,222	0.68%	111,430	0.85%	125,139	1.02%	8.72%	8,652	9.57%	10,665	10.42%	13,039
2060	102,654	0.68%	116,268	0.85%	131,687	1.02%	8.41%	8,635	9.31%	10,828	10.21%	13,448
2065	106,204	0.68%	121,315	0.85%	138,577	1.02%	8.10%	8,607	9.06%	10,986	10.00%	13,863
2070	109,877	0.68%	126,583	0.85%	145,827	1.02%	7.80%	8,566	8.80%	11,136	9.80%	14,285
2075	113,677	0.68%	132,078	0.85%	153,458	1.02%	7.49%	8,512	8.54%	11,279	9.59%	14,714
2080	117,609	0.68%	137,813	0.85%	161,487	1.02%	7.18%	8,444	8.28%	11,413	9.38%	15,147
2085	121,676	0.68%	143,796	0.85%	169,936	1.02%	6.87%	8,362	8.02%	11,537	9.17%	15,587
2090	125,884	0.68%	150,039	0.85%	178,828	1.02%	6.56%	8,263	7.77%	11,651	8.96%	16,030
2095	130,238	0.68%	156,553	0.85%	188,185	1.02%	6.26%	8,148	7.51%	11,753	8.76%	16,477
2100	134,742	0.68%	163,350	0.85%	198,031	1.02%	5.95%	8,014	7.25%	11,842	8.55%	16,928

TABLE 5
Population Projection for Unincorporated Areas, Polk County, Oregon: Observed Population from 1970 to 2000; Projected Population from 2000 to 2010

	Polk County						Unincorporated Areas					
	Low		Medium		High		Low		Medium		High	
Year	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	Pop.No.	Pop.Gr.	%Unincorporated Areas of Polk	Pop.No.	%Unincorporated Areas of Polk	Pop.No.	%Unincorporate d Areas of Polk	Pop.No.
1970	35,349	---	35,349	---	35,349	---		14,414	40.78%	14,414		14,414
1975	39,700	2.32%	39,700	2.32%	39,700	2.32%		15,848	39.92%	15,848		15,848
1980	45,203	2.60%	45,203	2.60%	45,203	2.60%		15,149	33.51%	15,149		15,149
1985	45,231	0.01%	45,231	0.01%	45,231	0.01%		15,219	33.65%	15,219		15,219
1990	50,088	2.04%	50,088	2.04%	50,088	2.04%		15,231	30.41%	15,231		15,231
1995	56,450	2.39%	56,450	2.39%	56,450	2.39%		16,714	29.61%	16,714		16,714
2000	62,380	2.00%	62,380	2.00%	62,380	2.00%		16,610	26.63%	16,610		16,610
2005	65,907	1.10%	66,819	1.37%	67,744	1.65%	25.56%	16,844	25.91%	17,311	26.26%	17,788
2010	69,420	1.04%	71,301	1.30%	73,232	1.56%	24.79%	17,207	25.19%	17,959	25.59%	18,738
2015	73,028	1.01%	75,963	1.27%	79,015	1.52%	24.02%	17,539	24.47%	18,586	24.92%	19,688
2020	76,611	0.96%	80,649	1.20%	84,901	1.44%	23.25%	17,810	23.75%	19,152	24.25%	20,586
2025	80,100	0.89%	85,266	1.11%	90,766	1.34%	22.48%	18,004	23.03%	19,634	23.58%	21,400
2030	83,411	0.81%	89,695	1.01%	96,453	1.22%	21.71%	18,106	22.31%	20,008	22.91%	22,095
2035	86,576	0.74%	93,969	0.93%	101,994	1.12%	20.94%	18,127	21.59%	20,285	22.24%	22,680
2040	89,601	0.69%	98,091	0.86%	107,385	1.03%	20.17%	18,070	20.87%	20,469	21.57%	23,160
2045	92,700	0.68%	102,350	0.85%	113,004	1.02%	19.40%	17,981	20.15%	20,621	20.90%	23,615
2050	95,906	0.68%	106,793	0.85%	118,917	1.02%	18.63%	17,864	19.43%	20,747	20.23%	24,053
2055	99,222	0.68%	111,430	0.85%	125,139	1.02%	17.86%	17,718	18.71%	20,845	19.56%	24,474
2060	102,654	0.68%	116,268	0.85%	131,687	1.02%	17.09%	17,541	17.99%	20,913	18.89%	24,872
2065	106,204	0.68%	121,315	0.85%	138,577	1.02%	16.32%	17,329	17.27%	20,948	18.22%	25,245
2070	109,877	0.68%	126,583	0.85%	145,827	1.02%	15.55%	17,083	16.55%	20,946	17.55%	25,589
2075	113,677	0.68%	132,078	0.85%	153,458	1.02%	14.78%	16,798	15.83%	20,904	16.88%	25,899
2080	117,609	0.68%	137,813	0.85%	161,487	1.02%	14.01%	16,474	15.11%	20,820	16.21%	26,172
2085	121,676	0.68%	143,796	0.85%	169,936	1.02%	13.24%	16,106	14.39%	20,688	15.54%	26,403
2090	125,884	0.68%	150,039	0.85%	178,828	1.02%	12.47%	15,694	13.67%	20,506	14.87%	26,587
2095	130,238	0.68%	156,553	0.85%	188,185	1.02%	11.70%	15,234	12.95%	20,269	14.20%	26,717
2100	134,742	0.68%	163,350	0.85%	198,031	1.02%	10.93%	14,723	12.23%	19,973	13.53%	26,788

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– IMPORTANT NOTICE –

Preliminary 2008 Population Estimate

November 15, 2008

To: **Dallas city**

Listed below is the preliminary population estimate for July 1, 2008. Also included are the certified 2007 estimate and 2000 Census figure. The July 1, 2008 estimate will be certified by December 31, 2008.

PRELIMINARY POPULATION ESTIMATE:

JULY 1, 2008: **15375**

CERTIFIED POPULATION ESTIMATE:

JULY 1, 2007: **15065**

CERTIFIED CENSUS FIGURE:

APRIL 1, 2000: **12459**

If you have any questions, please contact:

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Certified Population Estimates for Oregon's Cities and Towns

Incorporated City/Town	Certified Estimate July 1, 2008	Incorporated City/Town	Certified Estimate July 1, 2008	Incorporated City/Town	Certified Estimate July 1, 2008
Adair Village	930	Dayville	175	Imbler	295
Adams	335	Depoe Bay	1,405	Independence	8,030
Adrian	185	Detroit	265	Ione	350
Albany	48,770	Donald	1,025	Irrigon	1,865
Amity	1,480	Drain	1,080	Island City	995
Antelope	60	Dufur	655	Jacksonville	2,655
Arlington	610	Dundee	3,050	Jefferson	2,655
Ashland	21,485	Dunes City	1,360	John Day	1,845
Astoria	10,080	Durham	1,395	Johnson City	675
Athena	1,270	Eagle Point	8,730	Jordan Valley	240
Aumsville	3,535	Echo	715	Joseph	1,105
Aurora	970	Elgin	1,705	Junction City	5,300
Baker City	10,140	Elkton	250	Keizer	36,150
Bandon	3,300	Enterprise	1,975	King City	2,775
Banks	1,435	Estacada	2,820	Klamath Falls	21,305
Barlow	140	Eugene	154,620	La Grande	12,935
Bay City	1,265	Fairview	9,735	La Pine	1,610
Beaverton	86,205	Falls City	965	Lafayette	3,925
Bend	80,995	Florence	9,410	Lake Oswego	36,590
Boardman	3,330	Forest Grove	21,465	Lakeside	1,560
Bonanza	435	Fossil	465	Lakeview	2,750
Brookings	6,465	Garibaldi	895	Lebanon	15,185
Brownsville	1,775	Gaston	660	Lexington	285
Burns	3,025	Gates	505	Lincoln City	7,875
Butte Falls	445	Gearhart	1,220	Lonerock	20
Canby	15,165	Gervais	2,260	Long Creek	220
Cannon Beach	1,690	Gladstone	12,215	Lostine	250
Canyon City	675	Glendale	955	Lowell	1,015
Canyonville	1,730	Gold Beach	2,155	Lyons	1,130
Carlton	1,755	Gold Hill	1,080	Madras	6,640
Cascade Locks	1,050	Granite	30	Malin	810
Cave Junction	1,730	Grants Pass	32,260	Manzanita	725
Central Point	17,160	Grass Valley	170	Maupin	490
Chiloquin	720	Greenhorn	2	Maywood Park	750
Clatskanie	1,740	Gresham	100,655	McMinnville	32,400
Coburg	1,075	Haines	435	Medford	76,850
Columbia City	1,975	Halfway	355	Merrill	915
Condon	780	Halsey	840	Metolius	880
Coos Bay	16,670	Happy Valley	11,455	Mill City	1,640
Coquille	4,165	Harrisburg	3,435	Millersburg	1,135
Cornelius	10,955	Helix	230	Milton-Freewater	6,580
Corvallis	54,880	Heppner	1,425	Milwaukie	20,915
Cottage Grove	9,445	Hermiston	16,080	Mitchell	175
Cove	640	Hillsboro	89,285	Molalla	7,590
Creswell	4,710	Hines	1,870	Monmouth	9,565
Culver	1,325	Hood River	6,850	Monroe	690
Dallas	15,360	Hubbard	3,125	Monument	135
Damascus	9,975	Huntington	560	Moro	385
Dayton	2,500	Idanha	230	Mosier	470

Incorporated City/Town	Certified Estimate July 1, 2008	Incorporated City/Town	Certified Estimate July 1, 2008
Mt. Angel	3,785	Silverton	9,540
Mt. Vernon	600	Sisters	1,875
Myrtle Creek	3,665	Sodaville	290
Myrtle Point	2,550	Spray	160
Nehalem	240	Springfield	58,005
Newberg	22,645	St. Helens	12,325
Newport	10,580	St. Paul	415
North Bend	9,855	Stanfield	2,215
North Plains	1,905	Stayton	7,815
North Powder	500	Sublimity	2,285
Nyssa	3,210	Summerville	120
Oakland	945	Sumpter	170
Oakridge	3,745	Sutherlin	7,795
Ontario	11,435	Sweet Home	9,045
Oregon City	30,405	Talent	6,635
Paisley	250	Tangent	985
Pendleton	17,295	The Dalles	13,170
Philomath	4,610	Tigard	47,150
Phoenix	4,855	Tillamook	4,700
Pilot Rock	1,560	Toledo	3,610
Port Orford	1,275	Troutdale	15,465
Portland	575,930	Tualatin	26,040
Powers	730	Turner	1,730
Prairie City	1,110	Ukiah	260
Prescott	60	Umatilla	6,495
Prineville	10,370	Union	1,960
Rainier	1,810	Unity	115
Redmond	25,445	Vale	2,085
Reedsport	4,305	Veneta	4,840
Richland	150	Vernonia	2,365
Riddle	1,045	Waldport	2,145
Rivergrove	350	Wallowa	890
Rockaway Beach	1,375	Warrenton	4,650
Rogue River	2,090	Wasco	420
Roseburg	21,235	Waterloo	215
Rufus	275	West Linn	24,400
Salem	154,510	Westfir	340
Sandy	8,005	Weston	745
Scappoose	6,580	Wheeler	455
Scio	775	Willamina	1,885
Scotts Mills	300	Wilsonville	17,940
Seaside	6,445	Winston	5,890
Seneca	230	Wood Village	3,100
Shady Cove	2,850	Woodburn	23,355
Shaniko	40	Yachats	780
Sheridan	6,020	Yamhill	855
Sherwood	16,420	Yoncalla	1,115
Siletz	1,190		

FOR THE BOARD OF HIGHER EDUCATION
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12/15/2008