February 2002

Water Infrastructure Master Plan

City of Avondale

RBF CONSULTING

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FINAL

Water Infrastructure Master Plan
City of Avondale

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Abbreviations

ac	Acres
AC	Asbestos Cement Pipe
ADEQ	Arizona Department of Environmental Quality
ADWR	Arizona Department of Water Resources
AWS	Assured Water Supply
CAGRD	Central Arizona Groundwater Replenishment District
CAP	Central Arizona Project
CIP	Capital Improvement Program
COA	City of Avondale
COA-ESI	City of Avondale/ESI Corporation
CU	Color Units
dB	Decibel
DES	Department of Economic Security
DIP	Ductile Iron Pipe
DU	Dwelling Units
EDS	City of Avondale Engineering Design Standards
FY	City of Avondale Fiscal Year
gal	Gallons
gpcd	Gallons Per Capita Per Day
gpm	Gallons Per Minute
HMI	Human Machine Interface
IESWTR	Interim Enhanced Surface Water Treatment Rule
IFC	International Fire Code
LPSCO	Litchfield Park Service Company
MAG	Maricopa Association of Governments
MCL	Maximum Contaminant Levels
MDS	Microwave Data Systems (manufacturer)
MG	Million Gallons
MGD	Million Gallons Per Day
MPA	Municipal Planning Area
MTU	Master Telemetry Unit
PLC	Programmable Logic Controller
psi	Pounds Per Square Inch

PVC	Polyvinyl Chloride Pipe
RS-232	A Standard Serial Communication Protocol
RSL	Received Signal Strength
RTU	Remote Telemetry Unit
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
SRP	Salt River Project
SRP	Salt River Project
SWTR	Surface Water Treatment Rule
TAZ	Traffic Analysis Zones
TDS	Total Dissolved Solids
TFM	Total Fade Margin
TON	Threshold Odor Number
UFC	Uniform Fire Code
UPS	Uninterruptable Power Supply
USEPA	U. S. Environmental Protection Agency
VFD	Variable Frequency Drive
WSMP	City of Avondale Water System Master Plan

Executive Summary

E.1 Authorization

The City of Avondale, Arizona authorized RBF Consulting to complete a Water Infrastructure Master Plan and recommend a Capital Improvement Program (CIP) for both immediate and future water infrastructure improvements.

E.2 Purpose of Plan

Avondale is experiencing rapid growth throughout the City. This growth is placing an increased demand on the City's water system. In order to meet this demand and plan for the future, the City of Avondale has commissioned this water infrastructure master plan. The purpose of this plan is to analyze the existing water system within the City, and to identify areas that need immediate improvement. This plan also provides a recommended schedule of improvements needed to meet the projected demands in the City over the next 40 years. As part of the Water Infrastructure Master Plan, a proposed CIP has been developed, including cost estimates for proposed water infrastructure projects.

Another purpose of the Water Infrastructure Master Plan is to establish updated water system design criteria. Based on the analysis performed as part of this study, the City's water design criteria have been updated to reflect the current usage trends. Fire flow requirements have been evaluated, based on the requirements from the 2001 International Fire Code.

E.3 Study Area

The Municipal Planning Area, which was established in the City's *2001 General Plan Update,* was utilized for this study. The purpose of the Municipal Planning Area is to outline the current city boundaries, as well as those areas that the City intends to annex in the future. According to the *2001 General Plan Update*, the City's Municipal Planning Area starts at Indian School Road in the North, and runs to the Patterson Road alignment in the South. For the purposes of this study however, the portion of the planning area from Indian School Road to the Estrella Mountains was studied. Land use, population growth, and development patterns were analyzed for this study, in order to predict the water demand patterns.

E.4 Population and Demand Projections

In order to effectively plan the development of the City's water system, population growth and distribution patterns were studied. Current and historical population growth trends, along with the historical development patterns were analyzed. Results from this analysis were used to project the future population for each of the next five years, as well as for each of the planning periods. The population projections, which were computed for the study, are shown below in Table E.1,

City of Avondale Population Projections (2001-2006), and Table E.2, City of Avondale Planning Period Population Projections.

Table E.1 City of Avondale Population Projections (2001-2006)

Year	2001	2002	2003	2004	2005	2006
Population	40,350*	44,649	48,948	53,247	57,546	61,845

*Current City Estimate

Table E.2 City of Avondale Planning Period Population Projections

Year	2001	2006	2011	2016	2021	2026	2040
Population	40,350*	61,845	83,506	105,167	126,828	148,489	167,665

*Current City Estimate

With the cumulative population projections established, historical growth trends within the City of Avondale were studied. Growth trend predictions produced by Maricopa County were also analyzed. Based on the analysis of the growth trend predictions, population distribution was predicted throughout the City. The projected population distribution is found in Section 3.4 of the 2001 *Water Infrastructure Master Plan*.

Current and future water demands for the City of Avondale were also computed. Various types of historical water usage trends were evaluated (i.e. residential, commercial, construction, etc) in order to establish these water demands. Using the population projections and the water usage information, the anticipated citywide water demands were projected. Table E.3, Water Demand Conditions for 2001, 2006, 2011, 2026, and 2040, shows the average day, maximum day and the peak hour projected water demands in gallons per minute for each of the planning periods.

Table E.3 Water Demand Conditions for 2001, 2006, 2011, 2026, 2040

Year	Population	Avg Day (gpm)	Max Day (gpm)	Peak Hour (gpm)
2001	40,350	4,944	7,494	12,739
2006	61,845	8,590	17,179	29,205
2011	83,506	11,598	23,196	39,433
2026	148,489	20,623	41,247	70,120
2040	167,665	23,287	46,574	79,175

E.5 Infrastructure Analysis

In the City of Avondale 1996 *Water Master Plan*, a series of seven reliability tests were designed to aid the City in planning their water infrastructure. These reliability tests evaluate the capacity and reliability of the storage, supply, and booster pumps in the water distribution system. The existing system (2001) and

future water systems (2006, 2011, 2026, and 2040) were evaluated according to revised reliability test criteria. These evaluations were used to determine additional storage, supply, and booster pump improvements that would need to be constructed during each planning period.

The reliability tests themselves were first evaluated to determine how applicable they were to Avondale's current situation. Two of the original reliability tests were modified to provide a more applicable test of the City's current water system.

The City of Avondale's existing water system was evaluated against the reliability tests to determine if sufficient well supply, storage, and booster pump capacity existed in the current system. The reliability tests indicate that, based on the City's current demands, the existing infrastructure satisfies the reliability tests.

The reliability tests were evaluated for the demand conditions for each of the planning periods identified above. An inventory of the improvements that must be constructed during each of the planning periods is provided in Table E.4, Planning Period Storage and Supply Improvements. The improvements that must be completed during each planning period are listed in this table. For example, 9,700 gpm of supply must be added to the system by 2006, and an additional 8,000 gpm of supply must be added to the system by 2011. Therefore, a total supply of 17,700 gpm must be added to the system over these two planning periods.

Table E.4 Planning Period Storage and Supply Improvements

Improvements	2001	2006	2011	2026	2040	Total
Additional Supply (gpm)	0	9,700	8,000	24,000	7,100	48,800
Additional Storage (gal)	0	3,100,000	3,000,000	9,000,000	5,800,000	20,900,000
Additional Booster Pumps (gpm)	0	3,900	10,200	30,700	9,000	53,800

E.6 Water Model

A computer hydraulic model representing the City's existing water distribution system was developed. This model contains the main distribution lines within the City of Avondale. The existing system hydraulic model was analyzed under various operating scenarios in order to verify its accuracy with actual performance data.

The City's established operating criteria requires a minimum pressure of 40 psi for the maximum day and peak hour demand scenarios, and a minimum residual pressure of 20 psi for maximum day with fire flow conditions. These criteria were used as constraints in the existing water system model. Modeling results were evaluated to determine water system deficiencies.

Proposed water systems improvements were modeled and analyzed for each planning period. Various system improvements and design scenarios were studied in order to develop an efficient water system that meets the City's pressure criteria. These system improvements, together with the results of the reliability tests, formed the basis for developing the CIP.

E.7 Recommendations

Based on the hydraulic analysis performed for this study, the water system improvements, which are required to provide efficient and reliable service to the City of Avondale, have been identified. General recommendations for the City's water system are provided in this section. Specific infrastructure improvements are provided in the 5-year CIP, which is also included. The general recommendations for improving the City's water system fall into the following categories:

- SCADA System
- Water Master Plan Updates
- Capital Improvement Plan

Each category is summarized below.

E.7.1 SCADA System

One system improvement that will provide a significant benefit to the City's water system is the development of a system wide Supervisory Control and Data Acquisition (SCADA) system. A SCADA system will allow the City's critical water infrastructure such as wells, booster pumps, and tanks to communicate with operators at a control center.

Through the implementation of a SCADA system, the City will be able to automate many of their water operations. Automation of the City's water system will provide a two-fold benefit. First, the operation of the City's water system will be more efficient and effective. Second, the SCADA system is able to record key operating parameters, which will allow the City to better analyze what is happening within the system. This information may be used in future system planning and design. The research performed for this study, suggests that Cold Water Springs should act as the control center for the City's SCADA system.

E.7.2 Water Master Plan Updates

Over the next 5 years, many changes will take place within the City of Avondale. The City will continue to grow and expand as residential and commercial growth continues to occur. In order to ensure that the plans presented in this Water Infrastructure Master Plan continue to provide effective guidance to the City of Avondale, it is recommended that this Plan be revised and updated every five years.

As discussed earlier, only the portion of the Avondale Municipal Planning Area north of the Estrella Mountains was studied. Growth in the portion of the Municipal Planning Area south of the Estrella Mountain is not currently anticipated, however, it is recommended that a separate study be performed on this area, at the time development necessitates it.

Based on the water system hydraulic analysis, population growth and associated demands, the reliability test recommendations, and discussions with City staff, a Capital Improvement Plan (CIP) has been developed. This CIP provides an inventory of the projects recommended for construction in order to meet the water system demand and pressure requirements. The CIP also provides a schedule of recommended improvements and corresponding cost estimates. Figure E.1, CIP Progress, shows the proposed increase in storage and supply along with the projected demands. In order to have a sufficient water supply, the supply must always be greater than the average demands as shown in the figure below. Both the average day demands and maximum day demands are shown on the graph. Supply exceeds the max day demands in order to have back up redundancy in the system, and to meet the peak hour demands.

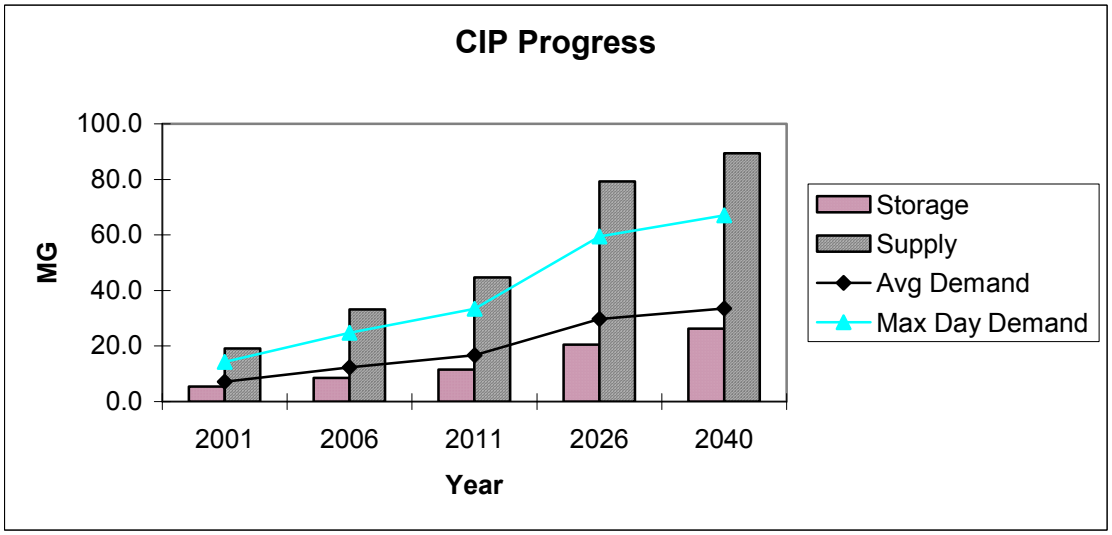


Figure E.1 CIP Progress

In the development of the CIP, one of the key issues that had to be analyzed was new reliable sources of water. Both centralized surface water treatment and wellhead treatment were evaluated. Based on the analysis performed as part of this study, it is recommended that the City increase their well supply in order to recover the water that they have recharged through the recharge basins. Section 6 of the *2001 Water Resources Master Plan*, provides a more detailed discussion of methods for acquiring more wells within the City's boundaries.

One of the greatest costs in this CIP is associated with treating water to meet current and future Federal water quality standards. Avondale has a history of wells with nitrate and arsenic problems. This is further complicated by the fact that the Federal government has mandated a reduction in the amount of arsenic allowed in drinking water from 50 μg/l to 10 μg/l by January 22, 2006. Due to these facts, an estimate of wellhead treatment costs is provided for each well listed in the CIP. A detailed breakdown of the treatment costs is provided in Section 6.3.3 of the 2001 *Water Resources Master Plan*.

It has been estimated that approximately 60% of the wells within the City will not comply with the future arsenic or current nitrate standards. Information available for this study was not sufficient to determine which proposed wells would need treatment. Therefore, when determining the costs for the CIP, it was estimated that the future wells would follow this ratio. In order to equitably represent the costs of treatment, each proposed well listed in the CIP contains 60% of the wellhead treatment costs. These costs may or may not be incurred depending on various characteristics of the well, i.e. volume, water quality, etc. It is also important to note that the cost estimate for each well also includes a half-mile of transmission mains.

The 5-year CIP is shown in Table E.5, 5-Year CIP. It is important to note that the hydraulic model that was used to develop this CIP only analyzed significant water mains. A 10-year and 20-year CIP were also developed as part of this study, and are included in Section 9.3 and Section 9.4 of the 2001 *Water Infrastructure Master Plan.* More detailed information regarding cost estimating procedures, as well as a breakdown of individual items included in the cost estimates is provided in Section 9.1 of the 2001 *Water Infrastructure Master Plan*.

Table E.5 5-Year CIP

Waterline						
Waterline	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Van Buren from Agua Fria River to 115th Ave – 36" Waterline	2,000,000					2,000,000
Indian School from El Mirage to 119th Ave – 20" Waterline	950,000					950,000
99th Ave from Well #8 to Thomas Rd - 16" Waterline	500,000					500,000
Thomas Rd from 107th Ave to 99th Ave – 16" Waterline		900,000				900,000
Lower Buckeye Rd from west bank of Agua Fria River to El Mirage – 16" Waterline			750,000			750,000
Encanto from 103rd Ave to 99th Ave - 12" Waterline			250,000			250,000
107th Ave from Buckeye Rd to Van Buren– 16" Waterline				800,000		800,000
Brinker Dr. alignment from 115th Ave to 107th Ave – 12" Waterline					375,000	375,000
Buckeye Rd from 119th Ave to 115th Ave – 12" Waterline					200,000	200,000
Dysart Rd from Buckeye to Lower Buckeye – 12" Waterline					1,500,000	1,500,000
Waterline Replacement and Upgrade – Old Town and East Avondale*	50,000	50,000	50,000	50,000	50,000	250,000
Subtotal	3,500,000	950,000	1,050,000	850,000	2,125,000	8,475,000

Table E.5 5-Year CIP (Continued)

Storage and Booster Stations						
Storage and Booster Stations	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Garden Lakes Reservoir and Booster Station – 2.5 MG Storage and 3,000 gpm Booster Pumps		1,500,000				1,500,000
Coldwater Springs Reservoir and Booster Station – Phase 2 – 2.5 MG Storage and 4,000 gpm Booster Pumps			1,500,000			1,500,000
Well 8 Reservoir – Phase 2 – 1.0 MG Storage				750,000		750,000
Lower Buckeye Reservoir and Booster Station – 119th Ave and Lower Buckeye Rd – 1MG Storage and 2,500 gpm Booster Pumps					1,000,000	1,000,000
Subtotal	-	1,500,000	1,500,000	750,000	1,000,000	4,750,000
Wells						
Wells	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Coldwater Springs Well – 2,000 gpm	600,000	400,000	4,000,000			5,000,000
Garden Lakes Well – 4,000 gpm		600,000	400,000	8,000,000		9,000,000
Additional Wells – 2,000 gpm		600,000	400,000	4,000,000		5,000,000
PIR Well - 1,000 gpm			600,000	400,000	2,000,000	3,000,000
Annual Hydrogeological Testing	250,000	250,000	250,000	250,000	250,000	1,250,000
Subtotal	850,000	1,850,000	5,650,000	12,650,000	2,250,000	23,250,000

Table E.5 5-Year CIP (Continued)

Miscellaneous						
Miscellaneous	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Well No. 6 Wellhead Treatment - upgrade existing well to comply with arsenic rule	1,500,000					1,500,000
Well No. 7 Wellhead Treatment - upgrade existing well to comply with arsenic rule		1,500,000				1,500,000
Existing Infrastructure SCADA system upgrade		201,000	200,000			401,000
Well No. 14 Wellhead Treatment - upgrade existing well to comply with arsenic rule			800,000			800,000
Well No. 15 Wellhead Treatment - upgrade existing well to comply with arsenic rule				750,000		750,000
Well Head Treatment – existing wells - upgrade existing wells for nitrate removal				2,000,000	2,000,000	4,000,000
Water Master Plans Update					400,000	400,000
Subtotal	1,500,000	1,701,000	1,000,000	2,750,000	2,400,000	9,351,000
Total	**$5,850,000**	**$6,001,000**	**$9,200,000**	**$17,000,000**	**$7,775,000**	**$45,826,000**

E.7.3 Development Fees

In order to help fund water system expansion and improvements, the City of Avondale charges a water development fee (for infrastructure improvements such as well pumps, reservoirs, booster stations and distribution systems) and a water resources development fee (for development of water supplies). For a residential development, the City currently charges $750 each per unit for the water resources and water infrastructure fees, yielding a total development charge of $1,500 for water improvements for each residential unit assessed. A comparison of current water related development fees for surrounding communities is provided in Table E.6, Water Development Fees.

Table E.6 Water Development Fees

City	Water Resources	Water Infrastructure	Total
Avondale	750	750	$ 1,500
Goodyear	3,000	1,000	$ 4,000
Peoria	4,022		$ 4,022
Surprise*	824	1,770	$ 2,594

*The City of Surprise also requires the developer to construct 3,000 gpm of well supply, a 1,000,000 reservoir, and any associated transmission mains for each square mile of development. The City of Surprise will then reimburse the developer for these improvements up to the amount of impact fees paid by the developer.

Based on the significant growth occurring in the City as well as a comparison to surrounding communities, the City of Avondale is planning to increase their water development fees in order to better offset the increased costs associated with these new developments. It is important to note that the development fees listed above do not reflect the costs associated with arsenic treatment. It is likely that surrounding communities will increase their water development fees in order to comply with the arsenic rule requirements that become effective in the year 2006. The City of Avondale performed a development fee analysis to determine the maximum supportable development fees within their City.

The development fee analysis performed by the City of Avondale determined that the maximum supportable development fee for a single-family home is $3,289 (water and water resources improvements only). It is recommended that the City increase their water development fee to this amount. Table E.7, Development Fee Revenue Analysis, provides a comparison of the amount of revenue that will be generated over each of the planning periods for both the current and proposed water development fees. Table E.8, Total CIP Project Cost, shows a breakdown of the total costs for each of the CIP's. It should be noted that even by increasing the water development fees, there is a projected shortfall in the funding for the recommended capital improvements. The City will need to utilize other methods to pay for the remaining projects listed in the CIP.

Table E.7 Development Fee Revenue Analysis

Fee Schedule	2006	2011	2026	2040
Current ($1,500)	$10,800,000	$10,012,500	$30,465,000	$8,256,000
Proposed ($3,289)	$23,680,800	$21,954,075	$66,799,590	$18,102,656

Table E.8 Total CIP Project Cost

5 Year	10 Year	20 Year
$45,826,000	$37,095,000	$64,450,000

In addition to development fees, the City of Avondale should utilize other revenue sources to fund the capital improvement projects. Some of these other sources

may include the General Fund, a portion of the City's sales tax, grants, and other methods deemed appropriate by the City.

1.0 Introduction

1.1 Authorization

The City of Avondale, Arizona authorized RBF Consulting to complete a Water Infrastructure Master Plan and recommend a Capital Improvement Program (CIP) for both immediate and future water infrastructure improvements.

1.2 Purpose of Plan

Water infrastructure is critical to delivering a safe and reliable supply of water to residents in the City of Avondale. This Water Infrastructure Master Plan provides the City with a comprehensive planning tool to serve as a guide for their capital improvement program. This Plan will provide guidance for the orderly expansion of the City's water system including both the production and distribution facilities. The goals of the Water Infrastructure Master Plan are to determine the appropriate improvements for the three primary planning periods defined by the City. The three planning periods are 2001 to 2006, 2007 to 2011, and 2012 to 2026. Recommended improvements to the City's water infrastructure are defined, and a suggested schedule of implementation is presented.

1.3 Previous Master Plans

Avondale has been a rapidly growing and dynamically changing City. Currently Avondale is one of the fastest growing cities in the west valley. In order to grow and develop responsibly, as well as ensure that adequate water service is provided, the City of Avondale has commissioned the development of a water master plan. In 1996 RUST Environmental and Infrastructure Inc. developed a water system master plan for the City. This plan called for updates every five years. It has served as a valuable tool to aid in the effective development of the City of Avondale.

1.4 Scope of Work

The scope of work involved in developing the Water Infrastructure Master Plan included an evaluation of the existing system and current population demands. The future population and growth patterns for the City were projected. These populations and demands provide the foundation for the development of the City's future water infrastructure. A water system model was created in order to determine the most effective capital improvement projects. In addition, this water model was used to determine the appropriate sizing of water transmission mains, and the optimal location of both storage and booster pump facilities.

The Water Infrastructure Master Plan will act as a working tool to guide the development of the Avondale water system, including water resource development. Demand factors for both planning and design will be presented. In addition the appropriate fire flow factors for various development types are

outlined in the plan. For each of the planning periods, the recommended water infrastructure improvements and water resource development projects are identified. Specific recommendations for improvements to the water system for the first planning period 2001 to 2006 are presented in this plan. Recommendations for other planning periods are conceptual, and should be re-evaluated at the end of each period.

2.0 Land Use

2.1 Existing Land Use

The land use within the City of Avondale has changed significantly over the last few decades. Historically Avondale has been a farming community, with the majority of its land use dedicated primarily to agricultural uses. As time has passed a significant portion of the agricultural land in Avondale has been converted to residential subdivisions. A portion of Avondale has also been converted to commercial use. Avondale's municipal planning area is shown in Figure 2.1, Avondale Land Use. However, for the purposes of this study only that portion of the municipal planning area for Avondale which lies north of the Estrella Mountains was analyzed, as can be seen in Figure 2.2, Project Study Area. It is recommended that a separate study be performed on the area south of the mountains, at a time when development necessitates it.

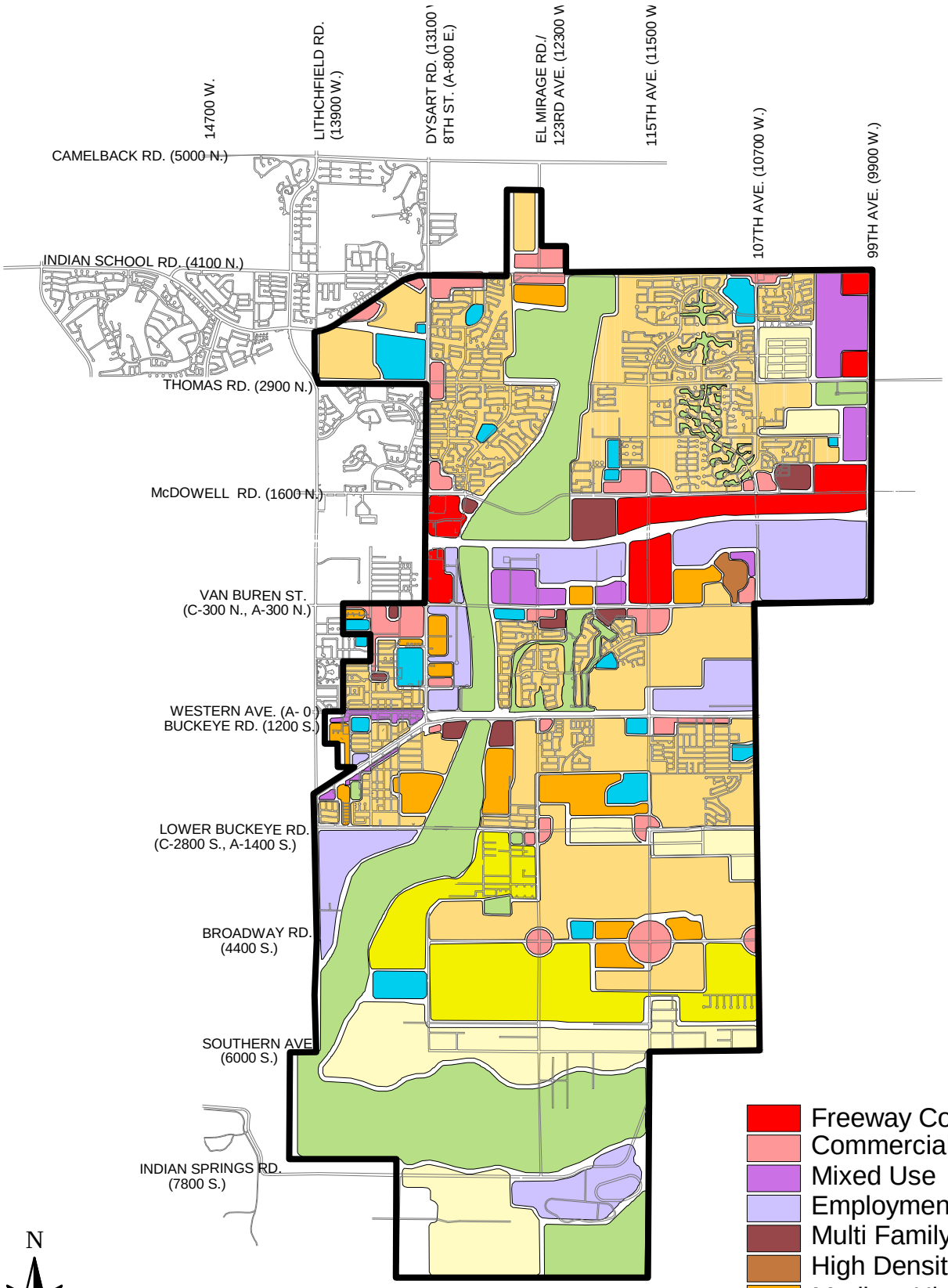
The current land use was evaluated to better understand the potential for population growth as well as the water needs within the City of Avondale. The existing land use was evaluated based on the City of Avondale Residential Developments Progress Map, which was provided by the Avondale Development Services Department, Planning Division. This map is shown in Figure 3.2, Residential Development Progress Map.

2.2 Proposed Land Use

In order to determine the future water needs and to develop an effective water distribution system, the proposed land use map from the City's 2001 *General Plan Update* was evaluated. As shown in the City's proposed land use map (Figure 2.1), Avondale is anticipating a significant amount of residential development. The City is also planning for a large amount of commercial development.

An analysis of the proposed land use was performed to ensure that the development of the water distribution system will meet the requirements for the existing and future needs within the City of Avondale.

According to the 2001 General Plan Land Use Map, 115th Avenue will serve as a main commercial corridor. It is important that a higher than average fire flow be provided along this corridor to satisfy the fire demands for large commercial developments. This was a significant consideration when determining the location of the future distribution mains in the City.



Freeway Commercial
Commercial
Mixed Use
Employment
Multi Family
High Density
Medium High Density
Medium Density
Low Density
Rural Low Density
Open Space
Public Facilities
Streets
Avondale Boundary

N
W
E
S

RBF CONSULTING

Figure 2.1
Avondale Land Use

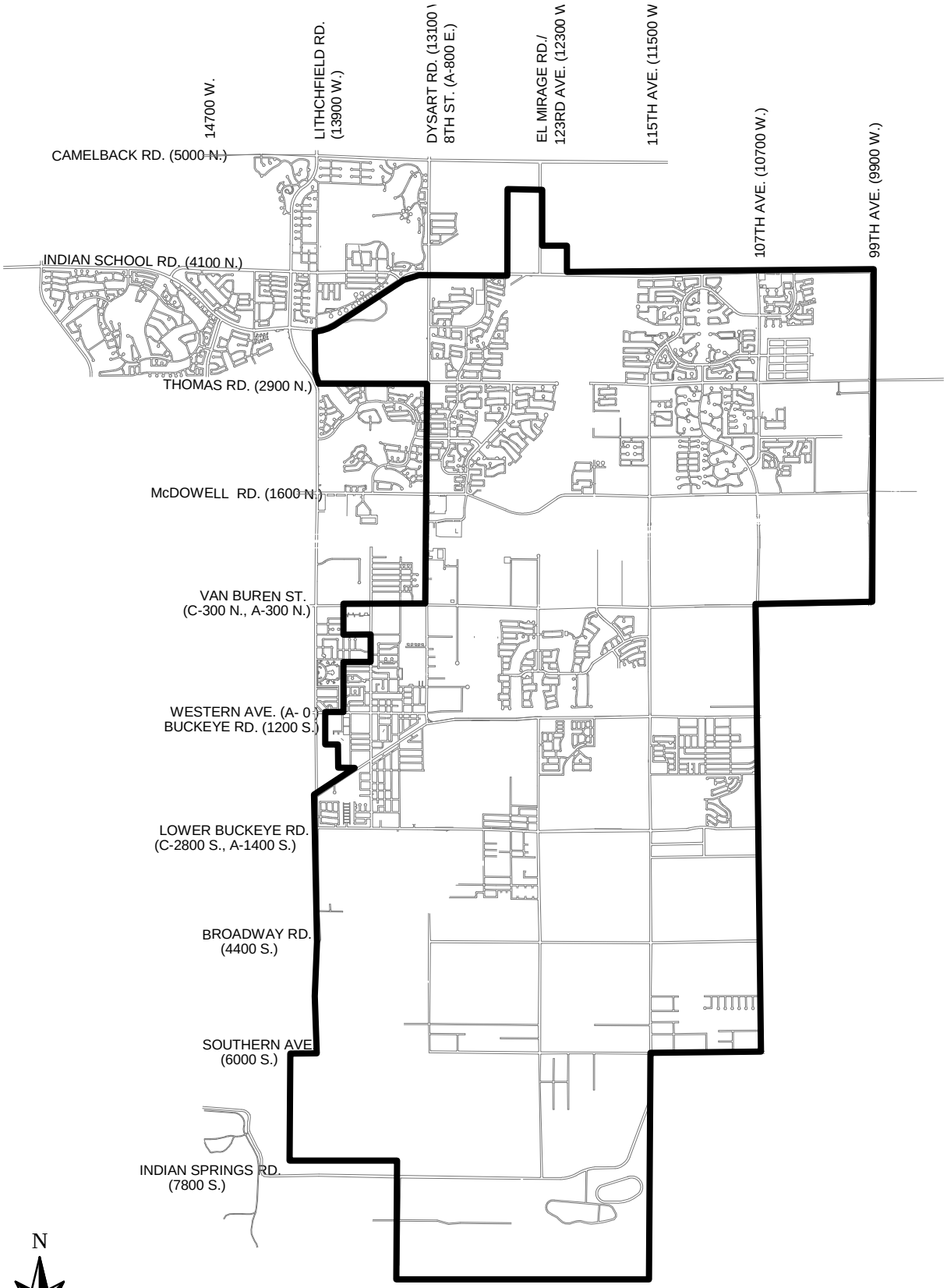


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Figure 2.2
Project Study Area

Avondale Boundary

Streets

3.0 Population and Demand Projections

The population estimates and population projections discussed in this report are defined within a modified Avondale Municipal Planning Area (MPA). The Study Area, which can be seen in Figure 2.2, is that portion of the Avondale municipal planning area located north of the Estrella Mountains.

3.1 DES and U.S. Census Estimates

U.S. Census population estimates for sub-county populations during any particular year are based primarily on the housing unit method. This method accounts for residential building permits distributed by a local government for any given year. The Department of Economic Security (DES) also uses a similar method described as the Housing Unit Method, which determines the number of housing units and multiplies this value by the average number of persons per household to estimate the total household population. Appendix A-1, Summary of Population Estimates for City of Avondale, shows U.S. census and DES population estimates for the City of Avondale from 1990 to 2000.

3.2 Current Population

The City of Avondale is one of the fastest growing communities in the state of Arizona. In order to project future City of Avondale populations, various population studies, completed by both governmental agencies and private companies, were analyzed. Based on their recent growth rate and the 2000 U.S. census report, the City of Avondale estimates that their 2001 population is 40,350 people.

3.3 Population Projections

In order to project the future population for the City of Avondale, three methods of population projections were studied. The methods which were studied include the Department of Economic Security (DES) Housing Unit Method, Maricopa Association of Government's (MAG) 1993 Traffic Analysis Zones (TAZ) studies, and the City of Avondale/ESI Corporation (COA-ESI) study which published three population growth scenarios for Avondale's General Plan Update (March 26, 2001).

3.3.1 DES Projections

One method of projecting population that was studied was the Department of Economic Security Housing Unit Method. The DES determines population projections for any future population based on the *State of Arizona Demographic Cohort – Survival Projections* Model. This model has four main components of population change, which consist of births, deaths, in-migration, and out-migration. See Appendix A-2, Summary of Population Projections for the City of Avondale MPA, for DES projections for the Avondale MPA (Study Area) from

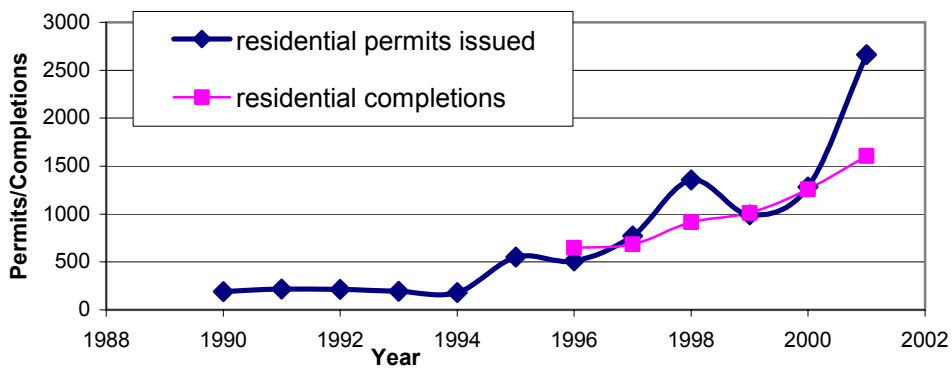
2001 through 2040. See Appendix A-3, Population Estimates and Projections, for a visual display of the DES projections.

3.3.2 TAZ Studies

The 1996 *Water System Master Plan* used the Maricopa Association of Government's (MAG) 1993 Traffic Analysis Zones (TAZ) to estimate future populations. This study utilizes zones based on USGS sections. The population projections were made according to land use percentages within each TAZ zone. Intermediate year populations for the summation of all TAZ zones were calculated by linear interpolation. The most recent TAZ count was prepared in 1995 and the updated TAZ count will not be finalized until June of 2002. Appendix A-2, Summary of Population Projections for the City of Avondale MPA, contains TAZ projections for the Avondale MPA from 2005 through 2040. See Appendix A-1, Population Estimates and Projections, for a visual display of the TAZ projections.

3.3.3 City of Avondale/ESI Study

The City of Avondale and ESI Corporation (COA-ESI) have published three different population projection scenarios for Avondale's *General Plan Update* dated March 26, 2001. These population projections were made for three different growth scenarios, "conservative", "moderate", and "aggressive". The "conservative" projection uses the growth rate associated with the DES projection for the Avondale MPA along with DES population estimate for the City in 2000. The "moderate" projection is based on the number of residential permits per year for the years 1996-2000 and the DES estimate for the City in 2000. Figure 3.1, Residential Building Permit Growth Trend, displays the trend in residential permits from 1990 to 2001 and residential completions from 1996 to 2001.



Note: total number of residential permits issued and residential completions shown for 2001 is estimated through Dec. 2001 based on the number of residential permits issued and residential completions recorded thru July of 2001.

Figure 3.1 Residential Building Permit Growth Trend

The "aggressive" projection uses the growth rate associated with the City of Avondale Fees Report, along with the DES estimate for the City in 2000. See Appendix A-2, Summary of Population Projections for the City of Avondale MPA, for COA-ESI projections through 2020. See Appendix A-3, Population Estimates and Projections, for a visual display of the COA & ESI projections for conservative, moderate, and aggressive growth.

3.4 Population Recommendations and Conclusions

The basis for the population projection that will be used for this study is the moderate COA & ESI projection (Appendix A-2) included in the General Plan Update for the City of Avondale dated March 26, 2001. This projection was computed by the City's Development Services Department, Planning Division in conjunction with ESI and is based on the growth of residential permits from 1996 to 2000 and the types of residential permits that were issued.

3.4.1 Population Projections

Table 3.1, City of Avondale Population Projections, lists the population projections which will be used for this study through the year 2040.

Table 3.1 City of Avondale Population Projections

Year	2001	2006	2011	2016	2021	2026	2040
Population	40,350*	61,845	83,506	105,167	126,828	148,489	167,665

*Current Population

Estimated population projections for the 5-year period between 2001 and 2006 have been projected, and are shown in Table 3.2, City of Avondale Population Projections (2001-2006).

Table 3.2 City of Avondale Population Projections (2001-2006)

Year	2001	2002	2003	2004	2005	2006
Population	40,350*	44,649	48,948	53,247	57,546	61,845

*Actual Population

3.4.2 Ultimate Build-out Population

The ultimate build-out population of the Avondale MPA is estimated to reach 167,665 by the year 2040 as seen in Table 3.1. The estimated ultimate build-out population figure is based on updated GIS land use data for the Avondale MPA, and assumes the maximum densities allowed per land use. Appendix A-4, Population Buildout Summary, presents backup for maximum build-out population calculations. This build-out population is comparable to the population studies completed by the Maricopa County Association of Governments (MAG). MAG determined the ultimate build-out population to be

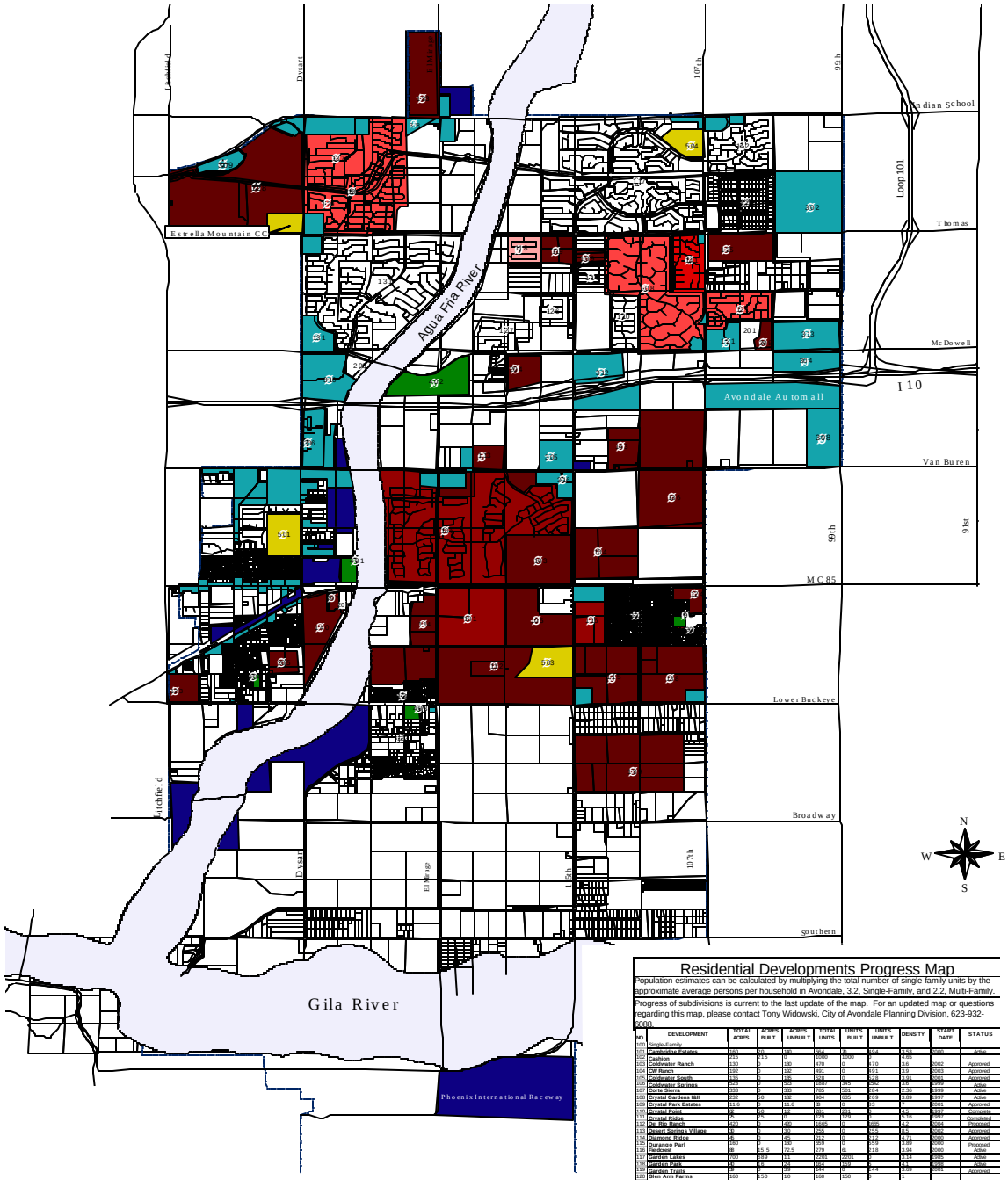
approximately 156,140 for the Avondale MPA in the *2000 MAG Regional Transportation Plan-Alternative Growth Scenarios Report*.

3.5 Future Growth Patterns

After establishing the total populations projected for each of the planning periods, the pattern in which the City will grow was projected. The City of Avondale Development Services Department, Planning Division assisted in the determination of anticipated growth patterns. In general, growth is occurring in the City from north to south. This general trend was followed, however more specific growth information was considered, to provide a better estimate.

The City of Avondale Development Services Department, Planning Division has compiled a *Residential Development Progress Map* which is shown in Figure 3.2. This map lists the current active, approved, and planned subdivisions for the City of Avondale as well as the anticipated number of lots for each subdivision. Based on information provided by the City's Development Services Department, it was assumed that a typical subdivision is constructed in a 5-year time frame. The 1993 TAZ boundaries were used to delineate specific areas where the growth would occur. Figure 3.3, 1993 Traffic Analysis Zones, illustrates these growth boundaries. Anticipated growth patterns were then projected using the 1993 TAZ boundaries.

The population distribution throughout the City of Avondale was determined for the year 2001 by using the number of completed lots as found in the City's *Residential Developments Progress Map*. The number of lots, for existing subdivisions not listed on the map, was estimated. For the year 2006 and 2011, the population was distributed through the existing, planned, and approved subdivisions. A higher priority was given to completing subdivisions that are currently under construction. All developments listed on the *Residential Developments Progress Map* are anticipated to be completed by the year 2011. The remaining development in the years 2011, 2026, and 2040 were then estimated and projected using the 1993 TAZ boundaries. Table 3.3, Population Distribution Projected on the 1993 TAZ, lists the population distribution projected on the TAZ boundaries for the planning periods.



1 0 1 2 Miles

Figure 3.2

Legend

Development Progress

- No Homes Built Yet
- 1% - 24%
- 25% - 49%
- 50% - 74%
- 75% - 99%
- 100% - Complete
- Commercial Zones
- Industrial Zones
- High school/ Higher Education
- City Parks

Residential Developments Progress Map

Population estimates can be calculated by multiplying the total number of single-family units by the approximate average persons per household in Avondale, 3.2, Single-Family, and 2.2, Multi-Family.

Progress of subdivisions is current to the last update of the map. For an updated map or questions regarding this map, please contact Tony Widowski, City of Avondale Planning Division, 623-932-6088.

NO.	DEVELOPMENT	TOTAL ACRES	ACRES BUILT	ACRES UNBUILT	TOTAL UNITS	UNITS BUILT	UNITS UNBUILT	DENSITY	START DATE	STATUS
100	Single-Family									
101	Cambridge Estates	160	15	145	364	0	364	4.63	2000	Active
102	Cashion	235	15	220	1000	1000	0	4.65		
103	Coldwater Ranch	130	0	130	470	0	470	3.6	2002	Approved
104	CW Ranch	192	0	192	491	0	491	3.9	2003	Approved
105	Coldwater South	235	0	235	918	0	918	3.91	2003	Approved
106	Coldwater Springs	553	0	553	1887	945	942	3.9	1999	Active
107	Corte Sierra	333	0	333	785	501	284	2.36	1999	Active
108	Crystal Gardens I&II	232	50	182	904	635	269	3.89	1997	Active
109	Crystal Park Estates	11.6	0	11.6	38	0	38	3.3	2001	Approved
110	Crystal Point	96	50	47	211	211	0	2.2	1997	Complete
111	Crystal Ridge	35	35	0	619	619	0	17.6	1997	Completed
112	Cyrtle Palms	420	0	420	1565	0	1565	3.7	2004	Proposed
113	Desert Springs Village	81	0	81	255	0	255	3.1	2002	Approved
114	Diamond Ridge	75	0	75	225	0	225	3.0	2002	Approved
115	Durango Park	160	0	160	509	0	509	3.18	2000	Proposed
116	Fieldcrest	38	15.5	22.5	279	0	279	7.34	2000	Active
117	Garden Lakes	700	189	511	2201	2201	0	3.14	1995	Active
118	Garden Park	6	1.6	4.4	19	19	0	3.1	1998	Active
119	Garden Trails	97	39	58	154	0	154	1.59	2002	Approved
120	Glen Arm Farms	160	150	10	160	150	0	1		
121	Harbor Shores	98	0	98	470	271	199	4.8	1999	Planned
122	Las Ligas									
123	Litchfield Mountain View	25	0	25	100	0	100	4	2003	Approved
124	Lithias Manor	35	0	35	209	0	209	5.97	1999	Planned
125	Los Arbolitos	80.5	0	80.5	306	0	306	3.8	2002	Proposed
126	PalmGardens	38	15	23	142	142	0	3.68	1996	Completed
127	PalmMeadows	61	0	61	183	0	183	3.0	2002	Active
128	PalmValley-Avondale	375	0	375	2088	0	2088	5.57	2000	Planned
129	PalmValley/Las Palmaras	42	0	42	281	187	94	6.43	1999	Planned
130	Pecan Groves	32	17	15	98	98	0	3.81	1998	Active
131	Rancho Santa Fe I	365	159	206	1202	1202	0	3.29	1995	Completed
132	Rancho Santa Fe II	188	113	75	597	595	2	3.42	1997	Active
133	Rio Cocopah	159	0	159	470	0	470	2.9	2000	Proposed
134	Rio Vista	70	70	0	300	300	0	4.3		
135	Riverfarm	28	0	28	116	0	116	4.08	2002	Planned
136	Roosevelt Park	207	0	207	1400	0	1400	6.8	2003	Proposed
137	Sage Creek	98	0	98	464	0	464	4.74	1999	Planned
138	Shadow Canyon	160	0	160	600	0	600	3.75	2001	Proposed
139	Shadowridge	232	0	232	934	0	934	4	2002	Planned
140	Tres Rios Landing	80	0	80	400	0	400	5	2000	Proposed
141	Upland Park	87	0	87	531	531	0	6.09	1999	Active
142	Westwind	132	132	0	538	538	0	4.07	1995	Completed
	TOTALS	6487.1	1749.5	4737.6	23581	9664	14307			

200	MULTI-FAMILY	UNITS	DENSITY	NO. 300	COMMERCIAL
201	Aventura Apts.	400		301 Alameda Crossing	
202	Avondale Senior Village	96	27.6	302 Algodon Business Center	
203	Coldwater Springs Apts.	300		303 Avondale Automall	
204	Crystal Springs Apts.	200	25.3	304 Avondale Park Plaza	
205	Rose Terrace	180	6.9	305 Avondale Town Centre	
206	Rio Santa Fe Apts.	360	19.9	306 Home Depot/ RIOR Walgreens	
207	Siesta Pointe Apts	104	5.08	307 Flowing Well	
208	Sonoma Apts.	128	7.2	308 Interstate Commerce Center	
209	Village at Avondale	80	5.08	309 Palm Valley Commercial	
	TOTALS	1921		310 Wal-Mart Super Center/ Office Max/ In N' Out Burger	
500	HIGH SCHOOL/ HIGHER EDUCATION			311 Phoenix Int'l Raceway	
501	Agua Fria High School			312 Tait Commercial	
502	Estrella Mountain Community College			313 Costco/ Harkins Theater	
503	Tolleson Union High School			314 CWP Sprint Commercial	
504	Westview High School			315 Indian School Crossing	
				316 Flowing Well Center	
				317 Westwind Plaza	

NO. 400	PARKS
401	Coldwater Park
402	Community Park II
403	Las Ligas Park
404	Interstate Valley Park
405	Dorothy Hale Park

Total Population By 2000 U.S. Census:

35,883

* The above information is approximate and is provided without warranty or guarantee for its accuracy. The information is submitted subject to errors and can be changed without notice.

Last Update: July 12, 2001

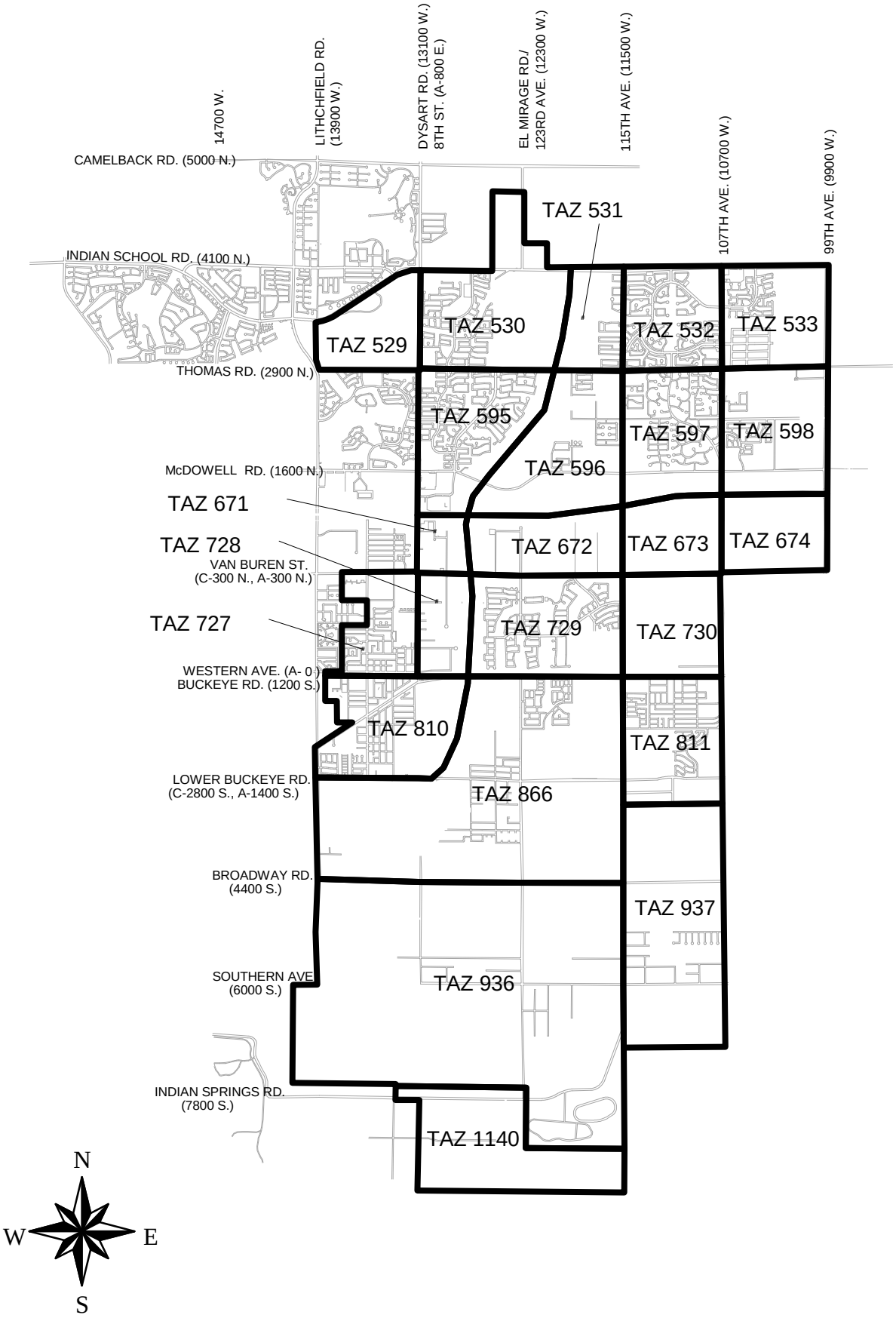


Figure 3.3
1993 Traffic Analysis Zones

TAZ
Streets

Table 3.3 Population Distribution Projected on the1993 TAZ

TAZ	2001	2006	2011	2026	2040
529	0	960	5,120	6,682	6,682
530	3,094	5,571	6,275	7,235	7,235
531	640	640	640	1,920	2,400
532	6,450	6,450	6,450	6,450	6,450
533	2,447	2,447	2,447	5,007	8,079
595	6,390	6,390	6,390	6,390	6,390
596	1,589	2,626	2,642	4,562	7,634
597	3,557	5,164	5,164	6,764	6,764
598	2,032	5,874	6,034	7,922	7,922
671	790	790	790	1,430	1,590
672	296	1,112	1,112	5,912	5,912
673	0	0	0	4,160	4,160
674	0	0	0	3,200	3,360
727	2,541	2,541	2,541	3,101	3,101
728	467	467	467	2,467	2,467
729	1,154	3,446	6,598	8,444	8,604
730	0	1,920	4,131	9,571	9,571
810	2,563	3,675	4,982	5,462	5,519
811	3,875	5,738	8,270	8,270	9,840
866	2,464	5,554	10,464	21,361	29,022
936	0	0	0	12,790	13,600
937	0	480	2,989	9,389	9,414
1140	0	0	0	0	1,949
TOTAL	40,350	61,845	83,506	148,489	167,665

3.6 Existing Water Demands

3.6.1 Current Water Demands Based on Well Pumping Records

Water pumping records from week ending January 9, 1998 to December 31, 2001 (see Appendix B-1, Water Pumping Records 1998 - 2001) and water consumption use by category records from 1998 to 2000 (see Appendix B-3, Water Consumption Use By Category for 1998, 1999 and 2000) were used to determine current water demand. The current water demand will be used to project future water demands.

The total average daily flow per capita was determined by dividing the average daily flow in gallons by the population served for that year. It is important to note that the City of Avondale water company does not distribute water to all Avondale residents. Some residents are served water by private water companies; Litchfield Park Service Company (LPSCO), Wilhoit Water Company (Wilhoit), and Rigby Water Company (Rigby). The private water companies are further addressed in Section 5.8. In 2000, ADWR determined the population served by

the City of Avondale Water Company was approximately 94% of the City's actual population. The Average Daily Flow and the Maximum Daily Flow for any given year were calculated from Appendix B-1, Water Pumping Records 1998 – 2001 and Appendix B-2, Daily Water Pumping Records for Peak Season. The maximum daily flow per capita was determined by dividing the maximum daily flow in any year by the corresponding service area population for that year. The maximum daily demand factor is computed by calculating the ratio of maximum daily flow (gallons) to average daily flow (gallons) for any particular year. Table 3.4, City of Avondale Water Consumption 1998-2001, shows the average daily flow per capita compared with the maximum daily flow per capita for the years 1998-2001. The flow values shown in Table 3.4 are for combined residential and nonresidential water use per capita.

Table 3.4 City of Avondale Water Consumption 1998-2001

	Average Daily Flow	Maximum Daily Flow	Population	Service Area Population	Total Avg Daily Flow per capita l	% Increase	Maximum Daily Flow per capita	Max Daily Demand Factor
Year	(gallons)	(gallons)	(DES estimate)[1]		(gpcd)		(gpcd)	ratio
1998	4,469,532	8,074,600	28,650	26,931	166		300	1.81
1999	5,304,869	8,586,100	32,270	30,334	175	5.4%	283	1.62
2000	6,192,434	10,564,000	35,850	33,699	184	5.1%	313	1.71
2001	7,118,921	10,791,000	40,350	37,929	188	2.2%	284	1.52

[1] Service Area Population estimates are established in July of each year by ADWR.

As seen in Table 3.4, the total average daily flow per capita has been increasing over the past four years. The per capita flows for residential use only are shown in Table 3.5, Residential Flows Per Capita. The average daily flow per capita for residential use was determined by dividing the total gallons pumped by the population served for that year divided by 365 days per year (Appendix B-3, Water Consumption Use By Category for 1998, 1999, and 2000).

Table 3.5 Residential Flows Per Capita

	Total Gallons Pumped – Residential	Service Area Population	Average Daily Flow per capita - Residential
Year	(gallons)	Served (1)	(gpcd)
1998	1,134,641,675	26,931	115
1999	1,309,880,000	30,334	118
2000	1,653,460,000	33,699	134
2001	2,598,406,000	37,929	188

Figure 3.4, Trends in Average Daily Flow Per Capita, displays the trend of the average daily flows from 1998-2001. Demands displayed in this figure include both residential and commercial demand.

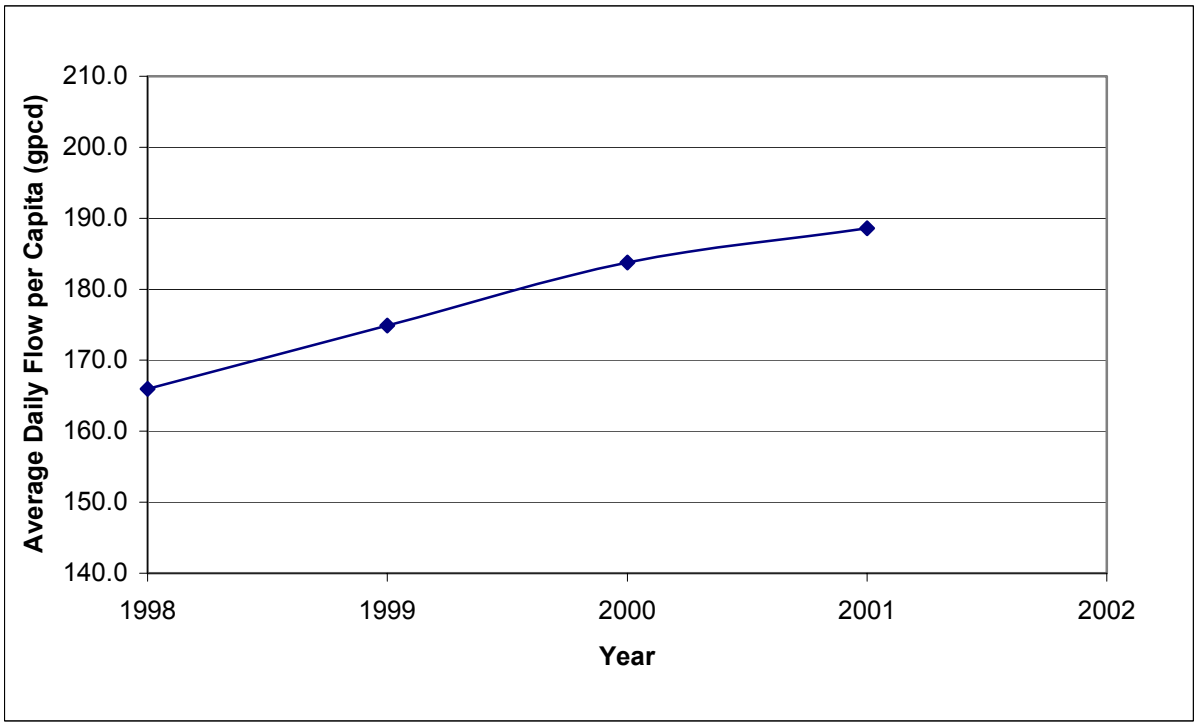


Figure 3.4 Trends in Average Daily Flow per Capita

It is important to note that, although average daily demand per capita has been increasing in recent years, this rate of increase appears to be slowing.

3.6.2 Current Water Demands Based on Use Type

In addition to evaluating the water demands based on the population, the demands were evaluated according to the land use type. Table 3.6 Recorded Water Consumption & Demand, presents current consumption data based from the City's meter book areas and calculated yearly average demands for the areas identified within the City of Avondale.

Table 3.6 Recorded Water Consumption & Demand

	Calculated Yearly Average Demand[1]			
	Total[1] (gpcd)	Residential [1] (gpcd)	Commercial [2] (gal/ac/day)	Residential [2] (gal/ac/day)
Demand (gpcd)	**151**	**146**	**257**	**1,807**

Notes: [1] See table 2 in Appendix B-4
[2] Commercial - See Table 1 in Appendix B-4

The calculated unit demand for commercial in gallons per acre per day, based on the meter data, does not accurately represent the actual commercial demand for commercial areas since these areas are not fully built out. This caused an abnormally low water usage for commercial developments as seen in Table 3.6.

The water demand based on land use records differs from the water demand based on the well production records due to the fact that the land use demand is based solely on meter records for five different areas within the Avondale MPA. The meter-based data does not account for water loss, fire flows, well bypass water and other types of unaccounted for water uses. These demands were determined from an accumulation of actual individual monthly meter readings for the month of September 2001 and grouped by land use and density. September 2001 was the only meter usage data provided by the City. Water usage was then divided into two categories, commercial and residential. Commercial demands were determined directly from the meter data provided from the City and residential demands were calculated by subtracting the commercial use from the total water usage. The calculated water demands based on land use type are shown in Table 3.7, Land Use-Based Water Demand Data. The water demands for the different densities were calculated based on the projected water use per dwelling unit for the different residential densities.

Table 3.7 Land Use-Based Water Demand Data

Land Use	Demand (gpcd)	Consumption (gal/ac/day)
Residential	141	
Multi family Density		5584
High Density		5414
Medium High Density		3610
Medium Density		1805
Low Density		1128
Rural Low Density		451
Industrial		See Table 5 Appendix B-4
Open Space		4,000
Commercial		2,000
Non-Residential		
School		See table 5 Appendix B-4
Church		n/a

It is recommended that an overall average commercial demand of 2,000 gallons per acre per day be used. This demand however, does not include high-rise commercial developments. This figure is also used by the City of Phoenix and Tempe as an average commercial demand and fluctuates depending on type of commercial development.

3.6.3 Current Water Demands in Other Municipalities

In order to compare the water rate trends for the City of Avondale, the average daily flows for various other communities in the Phoenix metropolitan area were researched. Table 3.8, Water Consumption Trends, lists the water consumption trends for these municipalities.

Table 3.8 Water Consumption Trends

City	Average Daily Flow - Total (gpcd)	Comments
Phoenix	226	2000-2001 Fiscal Report
Gilbert	222	Gilbert Master Water Plan 1996
Tempe	316 156	City of Tempe Residential (City of Tempe)
Mesa	200	City of Mesa

3.7 Water Demand Projections

In determining the water demand design value for this water master plan, the past water usage trends were analyzed. The average daily flow has been

increasing at a rate of 5% per year, as shown in Table 3.4, City of Avondale Water Consumption 1998-2001. As Avondale's median household income and economic development increases, this trend will continue and should level off somewhere between 190 - 200 gpcd. For the purposes of this report, and for design considerations, it is recommended that an average daily water demand of 200 gpcd be used. It is important to note, that although the per capita demand rate is increasing and currently at 188 gpcd, the future demand will ultimately depend on the implementation and success of the water conservation efforts.

4.0 Engineering and Planning Criteria

4.1 Demand Factors

4.1.1 Current Water Demands

Water pumping records (see Appendix B-1, Water Pumping Records 1998-2001, and Appendix B-2, Daily Water Pumping Records for Peak Season) from January 9, 1998 to December 31, 2001 and water consumption by category records (see Appendix B-3, Water Consumption Use by Category) from 1998 to 2000 were used to determine current water demand and will help establish future water demands.

The total average daily flow was determined by dividing the average daily flow in gallons by the population served for that year. The current population served by the City of Avondale water system is estimated to be 94% of the estimated population. The water system service population is updated by ADWR on an annual basis based on City of Avondale water service area maps. The maximum daily demand was determined by dividing the maximum water usage (see Appendix B-2, Daily Water Pumping Records for Peak Season) in any year by the corresponding population served for that year. The maximum daily demand factor is computed by determining the ratio of maximum daily flow per capita (gpcd) to the average daily flow per capita (gpcd) for any particular year. See Appendix B-3, Water Consumption Use by Category for water consumption per month for the years 1998-2000 and Table 4.1, City of Avondale Water Consumption 1998-2001 for average daily demand and maximum daily demand for the years 1998-2001.

Table 4.1 City of Avondale Water Consumption 1998-2001

	Average Daily Flow	Maximum Daily Flow	Population	Population	Average Daily Flow - Total	% Increase	Maximum Daily Flow per capita	Max Daily Demand Factor
Year	(gallons)	(gallons)	(DES estimate)	Served[1]	(gpcd)		(gpcd)	ratio
1998	4,469,532	8,074,600	28,650	26,931	166		300	1.81
1999	5,304,869	8,586,100	32,270	30,334	175	5.4%	283	1.62
2000	6,192,434	10,564,000	35,850	33,699	184	5.1%	313	1.71
2001	7,118,921	10,791,000	40,350	37,929	188	2.2%	284	1.52

Notes: [1] Population Served = 94% of actual population as recommended by ADWR

The average daily flow for the City of Avondale was analyzed for the years 1998-2001. In general, the average daily flow has been increasing at a rate of

approximately 5.0% per year. It is estimated that this trend should continue for the next few years, and the demand should eventually level off at approximately 200 gpcd. For the purposes of design, and for master planning purposes, an average daily flow of 200 gpcd will be used for this report. Table 4.2, City of Avondale Design Flows (gpcd), illustrates the demands that are recommended for design purposes, and are used throughout this water master plan. The actual average day demand rate will ultimately depend on the implementation and success of the City's proposed water conservation efforts. See Section 6.4 Water Conservation Measures in the 2002 *Water Resources Master Plan* for more information. A more detailed analysis of the development of the City's water demands can be found in Section 3.7 Water Demand Projections.

Table 4.2 City of Avondale Design Flows (gpcd)

Average Day	Maximum Day	Peak Hour
200	400	680

4.2 Peaking Factors

In order to plan for the growth and development in the City of Avondale, peaking factors were established for both daily and monthly intervals. The daily peaking factors were established by comparing the maximum day demand during the peak month of the year, with the computed average day demand. The maximum daily demand factor for the years 1998 to 2000 ranged from 1.52 to 1.81, as can be seen in Table 4.1, City of Avondale Water Consumption 1998-2001. For the purposes of this study, and for future design projects, it is recommended that a maximum daily peaking factor of 2.0 be used.

The City production and meter data was analyzed in order to recommend a peak hour peaking factor. While the City of Avondale currently has many flow meters that measure water production on a daily basis, they do not have any meters which record water production or consumption on an hourly basis. Due to the lack of this type of information, a peak hour factor was not able to be determined from field gathered data. Therefore, the peak hour factors for various surrounding communities were analyzed. It was determined that a peaking factor of 1.7 times the maximum day demand is the most applicable peaking factor for the City. Based on the usage patterns of surrounding communities, and the previous recommendation in the *1996 Water Master Plan* a peak hour factor of 1.7 times the maximum day demand has been used. Table 4.3, Recommended Peaking Factors, lists the peaking factors for the City of Avondale.

Table 4.3 Recommended Peaking Factors

Maximum Day	Peak Hour
2.0	1.7

Monthly peaking factors were also determined from water consumption data provided by the City of Avondale for the years 1998, 1999, and 2000. The monthly peaking factor was also determined (Table 4.4, Monthly Peak Factors for Avondale 1998, 1999, and 2000). This information is also presented in Figure 4.1, Monthly Water Use for Avondale 1998, 1999, and 2000.

Table 4.4 Monthly Peak Factors for Avondale 1998, 1999, and 2000

Month	1998	1998 Peak Factor	1999	1999 Peak Factor	2000	2000 Peak Factor	Average Peak Factor
Jan	83,768,561	0.68	104,215,000	0.69	154,794,000	0.77	0.71
Feb	72,398,204	0.58	92,131,000	0.61	118,777,000	0.59	0.60
Mar	81,949,453	0.66	95,012,000	0.63	124,361,000	0.62	0.64
Apr	93,566,866	0.75	135,058,000	0.90	158,718,000	0.79	0.82
May	146,372,134	1.18	158,443,000	1.05	160,603,000	0.80	1.01
Jun	164,756,307	1.33	219,089,000	1.46	256,041,000	1.28	1.35
Jul	154,638,563	1.25	169,336,000	1.13	228,244,000	1.14	1.17
Aug	169,015,935	1.36	202,073,000	1.34	296,389,000	1.48	1.40
Sep	187,523,480	1.51	174,718,000	1.16	286,601,000	1.43	1.37
Oct	113,011,694	0.91	157,190,000	1.05	180,955,000	0.90	0.95
Nov	119,165,710	0.96	176,884,000	1.18	271,968,000	1.36	1.17
Dec	101,330,885	0.82	119,534,000	0.80	167,138,000	0.83	0.82
Total	1,487,497,792	-	1,803,683,000	-	2,404,589,000	-	-
Average	123,958,149	-	150,306,917	-	200,382,417	-	-

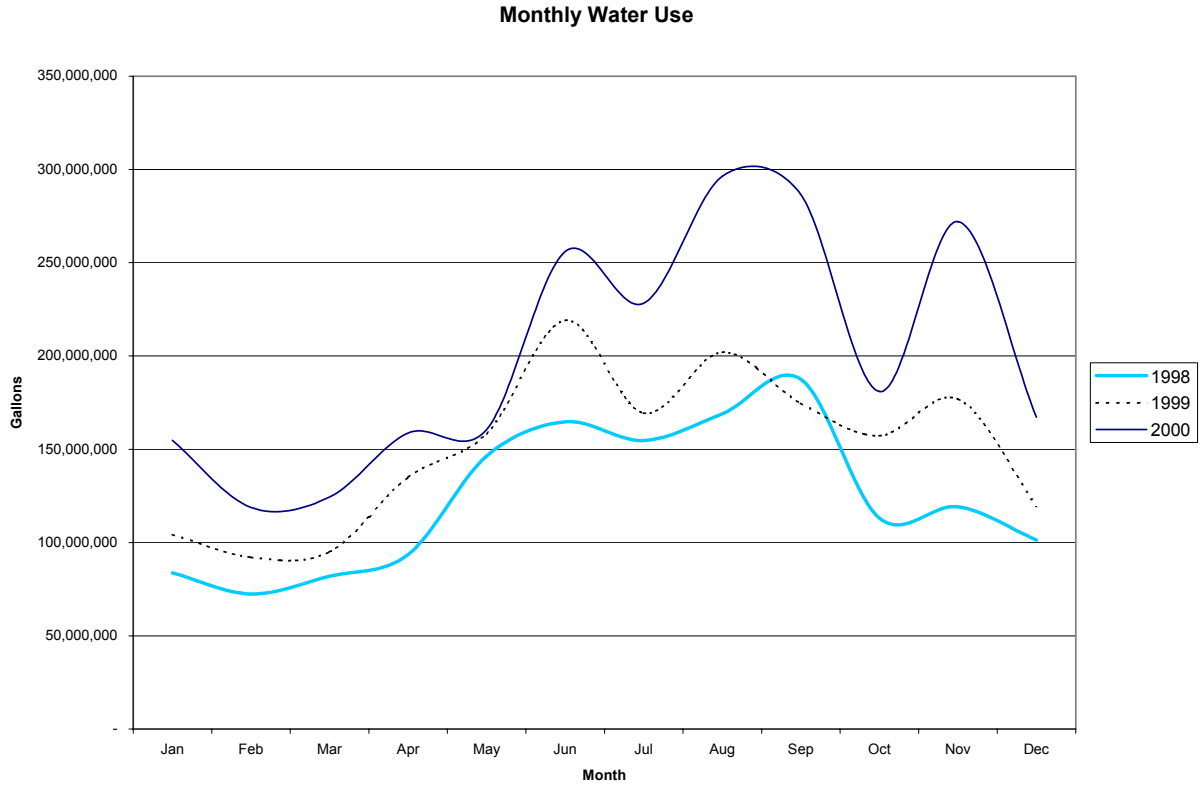


Figure 4.1 Monthly Water Use for Avondale 1998, 1999, and 2000

4.3 Fire Flow Protection Requirements

4.3.1 Existing Fire Flow Protection Requirements

One of the fundamental purposes of a municipal water system is to provide sufficient flow and pressure to fight fires. Currently the City of Avondale utilizes the 1997 Uniform Fire Code (UFC) to establish their fire flow and residual pressure requirements. The UFC has defined minimum flow and pressure requirements based on the type of construction of the building, as well as the building use. For example, the typical residential fire demand is generally less than a typical commercial fire flow demand. Table A-III-A-1 in Appendix III-A in the UFC lists the fire flow requirements for the various types of construction. The UFC also allows for a reduction in the required fire flow (as determined by the City's fire chief) if an approved automatic sprinkler system has been installed.

According to the 1997 Uniform Fire Code, the minimum required fire flow for a single family residential development is 1,000 gpm. Commercial developments on the other hand may require a fire flow ranging from 1,500 gpm to 8,000 gpm depending on the size of the building, its type of construction, and whether or not an approved fire sprinkler system is installed.

4.3.2 Recommended Fire Flow Protection Requirements

In the Spring of 2002, the City of Avondale anticipates adopting the 2000 International Fire Code (IFC). Similar to the UFC, the IFC contains fire flow requirements which are set based on the size, type of construction, and proposed usage of the buildings. The fire flows set forth in the IFC are based on maintaining a residual pressure of 20 psi, and are determined from the building's fire area. As defined by the IFC: "The fire area is the total floor area of all levels within the exterior walls, and under the horizontal projections of the roof of a building used to protect storage or use areas…"

The fire flows that should be provided for the different types of construction are described in Table B105.1 in Appendix B of the IFC. Similar to the UFC, the IFC allows for a reduction in the required fire flow (as approved by the fire chief) if an approved automatic sprinkler system has been installed. Should the City proceed with the adoption of the IFC, the basic fire flow requirements will be as follows:

- Minimum fire flow for one and two-family dwellings (less than 3,600 square-feet) is 1,000 gpm.
- Fire flow for commercial developments can range from 1,500 gpm to 8,000 gpm, but shall not be lower than 1,500 gpm with the fire flow reduction.

4.4 System Design Standards

4.4.1 Existing Design Standards

The City of Avondale has established engineering design requirements to help ensure that the most effective water system is designed for the City. The design standards for water mains are outlined in the *City of Avondale Engineering Design Standards, June 1997*. The City of Avondale minimum water system design requirements are shown in Table 4.5, Existing System Design Requirements.

Table 4.5 Existing System Design Requirements

Requirements	
Velocity	< 5 fps
Headloss	< 10 feet /1000 feet
Pressure	40 psi
Pipe Sizing	
Grid Arterials	16 in
Mid-section line	12 in
Others	8 in
Demands	
Max Day	440 gpcd
Max Day	2.3 × avg day
Peak Hour	1.7 × max day

The City of Avondale has established pressure requirements that the water system must maintain during the Average Day, Max Day, and Peak Hour demand scenarios. A minimum pressure of 40 psi must be maintained in the system during the Max Day and Peak Hour demand scenarios. A minimum pressure of 60 psi should be maintained throughout the water system on an Average Day demand scenario.

In order to plan for future system expansion, minimum size requirements for distribution mains have been developed. The City of Avondale requires that all water mains along grid arterials have a minimum diameter of 16-inches. Water mains on mid-section lines should have a minimum diameter of 12-inches, and all other water mains should have a minimum diameter of 8-inches. These water line sizes are to provide guidance on the minimum allowable diameter for a water line in the above locations. The actual diameter of the water line may need to be larger depending on the specific situation.

4.4.2 Proposed Design Standards

The existing design standards which are in use by the City of Avondale were analyzed based on their applicability to Avondale's current situation. Based on the research and analysis performed for this water master plan, it is proposed

that the maximum day water demand used for design be changed from 440 gpcd to 400 gpcd. This new demand provides an up to date design value based on the current water usage trends within the City (see Section 4.1). A 400 gpcd maximum day design value was computed based on the average and maximum day demand trends over the last four years. Table 4.1,City of Avondale Water Consumption 1998-2001, shows the actual average day and maximum day demands within the City of Avondale for the last four years. These values were extrapolated and anticipated to level off at 200 gpcd and 400 gpcd for the average day demand and maximum day demand respectively.

It is also recommended that the peaking factors which are used for design be changed from 2.3 to 2.0 times average day demand for the maximum day scenario, and remain at 1.7 times max day demand for the peak hour scenario. A maximum day factor of 2.0 provides a better representation of the maximum day use in the City. This also corresponds with current research, that as the population in a City increases, the peak demand on the water system becomes more level.

Proposed water distribution design standards are summarized in Table 4.6, Proposed Design Standards, below.

Table 4.6 Proposed Design Standards

	Requirements
Velocity	< 5 fps
Headloss	< 10 feet /1000 feet
Pressure	40 psi
Pipe Sizing	
Grid Arterials	16 in
Mid-section line	12 in
Others	8 in
Demands	
Max Day	400 gpcd
Max Day	2.0 × avg day
Peak Hour	1.7 × max day

5.0 Existing Water Supply and Distribution Facilities

The City of Avondale's water system consists of wells, tanks, booster pumps, distribution lines, and a wetlands water treatment and recharge facility. In 2001 the City's water system delivered an average daily flow of 7.2 MGD (see Table 4.1, City of Avondale Water Consumption 1998-2001) through a distribution network of pipes ranging in diameter from 4-inches to 24-inches.

The supply to the City's system is provided through both surface water and groundwater sources. The City receives surface water in the form of its CAP allotments and SRP entitlements. The surface water is processed through the wetlands treatment system, recharged within the recharge facilities, and then pumped into the reservoirs through the recovery wells. The reservoirs within the distribution system are currently on a level control system. A level control system works by filling the reservoir, when the water level drops below a certain level.

The majority of the groundwater wells in the system are currently pumped directly into the storage reservoirs, and then distributed through the system as needed. Since the majority of the storage reservoirs are filled directly from groundwater wells, there is minimal impact to the system pressure when the reservoirs are filled. Well #14 is the only recovery well in the system which pumps directly into the distribution system and not into a storage tank. In order to provide sufficient pressure to the system, each reservoir site is equipped with a booster pumping station. A detailed description and inventory is provided below for the various components in the City's water system. A conceptual layout of the City's existing water supply system can be seen in Figure 5.1, Existing Water Supply System. A layout of the City's existing waterlines can be seen in Figure 7.1, 2001 System. A photographic inventory of the City's well, tanks, and booster station sites is provided in Section 5.9.

5.1 Groundwater Wells

The primary source of water for the City of Avondale's distribution system is through the pumping of groundwater. Currently the City of Avondale has 14 wells with a design capacity of 17,330 gpm. Due to new water quality regulations, currently only 10 of these wells are in use. The pumping capacity, which is currently available for each well, was estimated by the City of Avondale Utilities Department. The sum of the estimated available well pump capacities is 13,311 gpm for the City's existing wells. Table 5.1, Avondale Municipal Well Inventory provides an inventory of the City of Avondale's wells, their actual production rates, as well as various other characteristics.

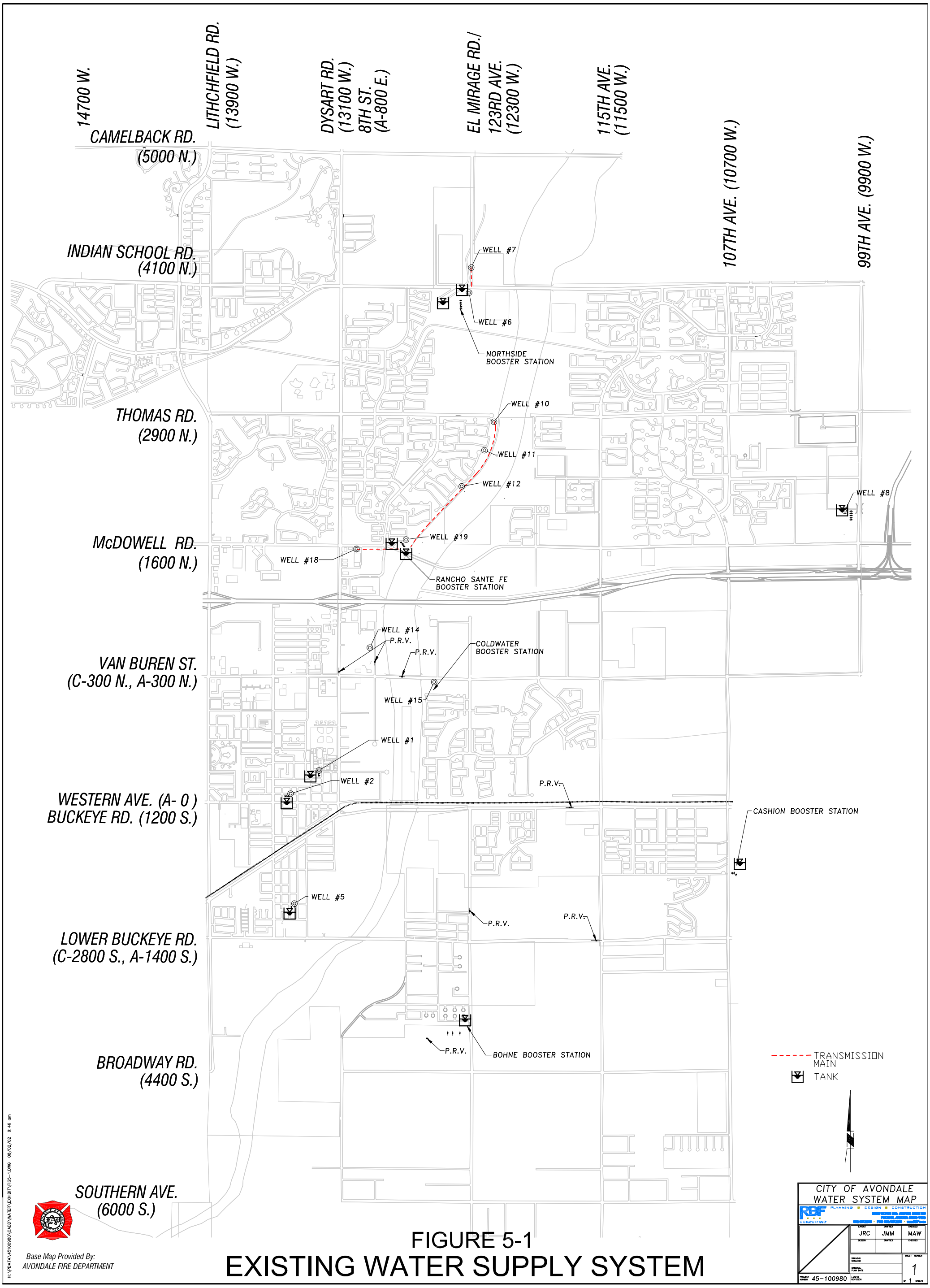


FIGURE 5-1

EXISTING WATER SUPPLY SYSTEM

Table 5.1 Avondale Municipal Well Inventory

COA well Number	Well Registration Number	Location of Well (All located within GSRB&M)	Available Pump Capacity (GPM)	Design Pump Capacity (GPM)	Well Depth (Feet)	Casing Diameter (Inches)	Maximum Annual recovery (Acre Feet)
1	55-608731	NW¼ SE¼ SE¼ Sec.10 T1N, R1W	1,068	1,200	456	16	1,000
2	55-608732	SW¼ SW¼ SE¼ Sec 10, T1N, R1W	0	1,200	460	20	500
4	55-608729	NW¼ NW¼ NW¼ Sec.15 T1N, R1W	0	880	400	14	500
5	55-608733	NE¼ SW¼ SE¼ Sec.15 T1N, R1W	0	1,200	550	16	500
6	55-501247	NE¼ NE¼ NE¼ Sec.26 T2N, R1W	1,448	1,650	608	18	1,500
7	55-501288	SE¼ SE¼ SE¼ Sec.23 T2N, R1W	1,358	1,600	530	18	1,500
8	55-520499	NE¼ SE¼ SE¼ Sec.32 T2N, R1E	616	1,750	660	16	500
10	55-608792	NE¼ NW¼ NW¼ Sec.36 T2N, R1W	2,071	1,500	866	20	1,500
11	55-608791	NE¼ NW¼ SW¼ Sec.36 T2N, R1W	1,638	1,500	618	20	1,500
12	55-608793	NE¼ NE¼ SE¼ Sec.35 T2N, R1W	1,761	3,000	458	20	1,500
14	55-583017	SE¼ NW¼ SW¼ Sec.2 T1N, R1W	450	450	800	20	970
15	55-578749	NW¼ NW¼ NE¼ Sec.11 T1N, R1W	600	600	800	20	1,500
18	55-607157	NW¼ NW¼ NW¼ Sec.2 T1N, R1W	2,301	1,500	650	20	1,500
Cashion	55-626592	NW¼ SW¼ NE¼ Sec.18 T1N, R1E	0	500	538	12	500
Total Available Pump Capacity			13,311				

In order to meet the increasing water demands, the City is actively striving to construct additional groundwater wells. By July 2002, the City anticipates to have an additional well online. Well 19 will provide an estimated production capacity of 1,500 gpm. With the addition of Well 19, the total well production for the City of Avondale will be 14,811 gpm.

5.2 Reservoirs

The City's water system is comprised of 10 storage reservoirs ranging in size from 100,000 gallons to 1.75 million gallons. Currently, the City has a total storage capacity of 5.4 million gallons. All of the City's reservoirs are above ground steel reservoirs. Table 5.2, Avondale Reservoir Inventory provides an inventory of the existing reservoirs within the system. Each of the reservoirs is connected to a series of booster pumps, which pump the water from the reservoir into the distribution system.

Table 5.2 Avondale Reservoir Inventory

Reservoir Description	Cadastral Location	Tank Vol (gal)	Base Elev (ft)	Min Elev (ft)	Max Elev (ft)	Diam (ft)
1	B(1-1) 10 ddb	200,000	975	978	1006	33
2	B(1-1) 10 dcc	300,000	972	975	1002	41.25
5	B(1-1) 15 dcd	300,000	965	968	997	40
6a	B(2-1) 26 aaa	600,000	1012.3	1015.3	1028.3	81
6b	B(2-1) 26 aaa	600,000	1012.3	1015.3	1028.3	81
8	A(2-1) 32 dda	400,000	1024.85	1027.85	1056.85	46.5
Cashion	A(1-1) 18 add	100,000	984	987	1007	28.5
Bohne	B(1-1) 23 add	150,000	960	963	984	33
Rancho Santa Fe I	B(1-1) 2 bbb	1,000,000	981.63	984.63	998	104
Rancho Santa Fe II	B(1-1) 2 bbb	1,750,000	981.63	984.63	998	137
Total Available Storage		5,400,000				

By March 2002, an additional reservoir and booster site will be online at Coldwater Springs. Coldwater Springs will add an additional 2,500,000 gallons of storage to the distribution system. This will bring the total storage within the City of Avondale to 7.91 million gallons.

5.3 Booster Pumps

The water system within the City of Avondale utilizes booster pump stations to introduce flow into the distribution system and to provide pressure throughout the system. Booster pump stations are located at each of the reservoir sites. Each booster pump station has between two and five booster pumps. Table 5.3, Booster Pump Inventory, provides an inventory of the existing booster pumps within the City, and their corresponding production settings. The City currently has a total design pumping capacity of 27,590 gpm.

Table 5.3 Booster Pump Inventory

Well	Booster Pump	Production (gpm)
6	1	1,200
6	2	1,200
6	3	1,200
6	4	1,200
6	VFD	1,800
8	1	400
8	2	600
8	3	850
8	4	850
8	5	1,200
1	1	1,000
1	2	1,000
1	Diesel Fire	1,500
5	1	500
5	2	500
2	1	100
2	2	1,500
Cashion	2	450
Cashion	3	250
Bohne	1	90
Bohne	2	900
Bohne	3	500
Rancho SF	VFD-1	2,200
Rancho SF	VFD-2	2,200
Rancho SF	VFD-3	2,200
Rancho SF	VFD-4	2,200
Total		27,590

By March of 2002 the City anticipates to have its Coldwater Springs booster station online. This site will have two booster pumps providing 2,000 gpm and 4,000 gpm of flow respectively. These booster pumps are both designed to provide a head of 184-feet (79 psi) to the system. With the addition of the two proposed booster pumps, the total pumping capacity for the City of Avondale will be 33,590 gpm. Ultimately, it is anticipated that there will be four booster pumps located at this site each providing 4,000gpm of flow.

5.4 Water Distribution System

The water distribution system within the City of Avondale consists of waterlines ranging in size from 2-inch to 24-inch. The pipelines consist of various materials such as cast iron pipe (CIP), ductile iron pipe (DIP), asbestos cement (AC), galvanized steel and Polyvinyl chloride (PVC).

The adequacy and condition of the water distribution system was analyzed in the 1996 *Water Master Plan*. It was determined in this master plan that the majority

of the smaller and older water mains are located in the Old Town area. Numerous connections between 2, 3, 4, and 6-inch pipes are located in the Old Town area. Due to the age and condition of the majority of these pipes, the Old Town area is one of the main areas of the city that requires immediate attention. The Cashion and Bohne neighborhoods also tend to have smaller, older pipes, and consequently lower flow and pressures. By replacing the main distribution pipes in these sections of the City, flow and pressure in the areas will be increased.

The City of Avondale has updated their water design standards, such that the minimum diameter pipe that will be constructed within the City is an 8-inch pipe.

The ground elevations within the City of Avondale generally tend to slope from a high in the northeast to a low in the southwest. The pressures within the water distribution system also tend to increase in the southern portions of the distribution system.

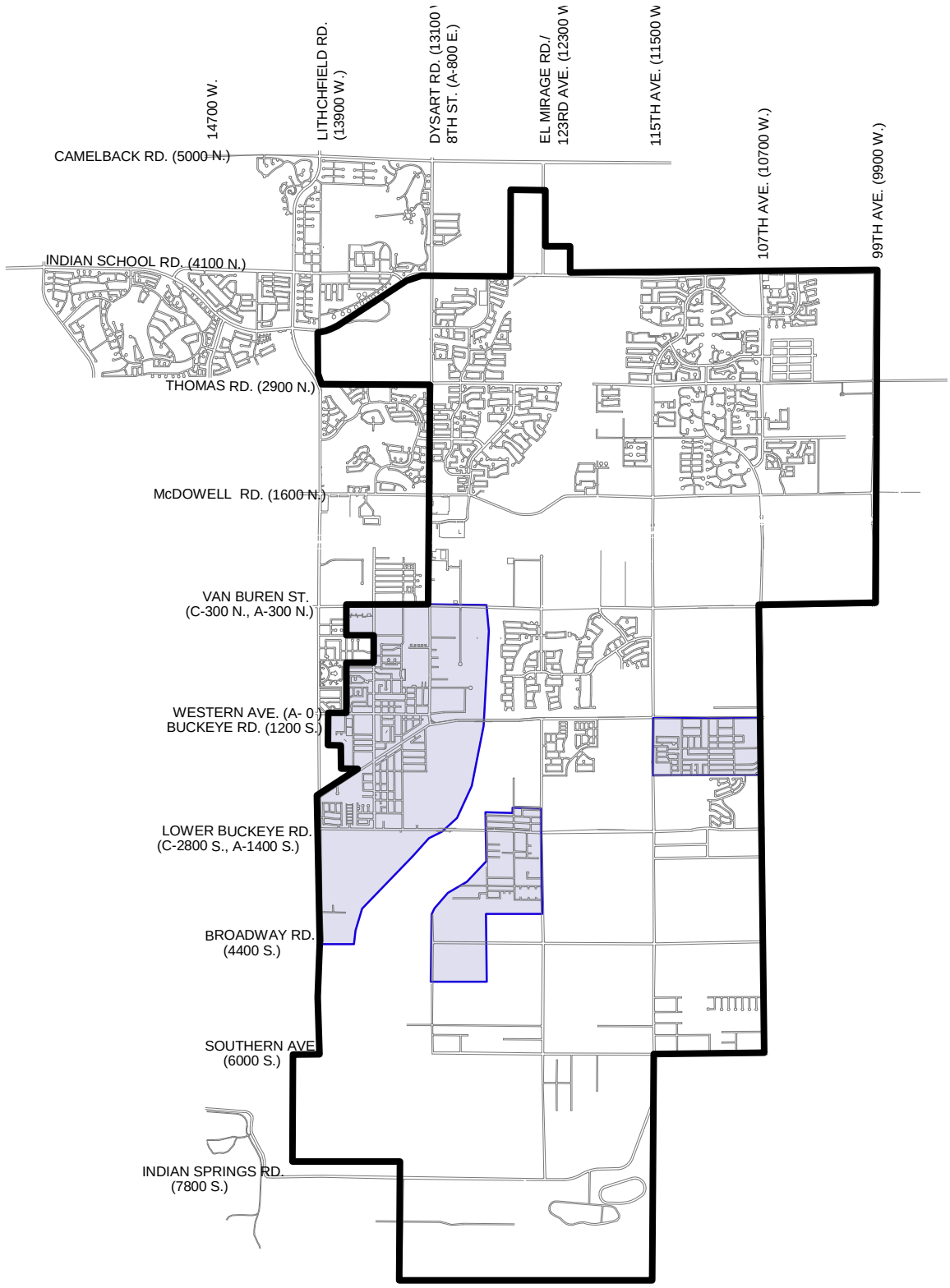
5.5 Pressure Zones

The City of Avondale is divided into two separate pressure zones, a high pressure zone and low pressure zone. The pressure zones within the City have been established to protect the pipes in the older areas of the City from damage due to increased pressures. Three areas of the City have been designated as low pressure zones. These areas include Bohne, Cashion, and Old Town Avondale. Figure 5.2, Low Pressure Zones, illustrates the locations of the lower pressure zones within the City of Avondale. All other areas of the City have been designated as a high pressure zone.

Pressure is controlled to the three low pressure zones through a network of Pressure Reducing Valves. A pressure reducing valve acts by reducing the pressure on the downstream side of the valve to a preset value. If the pressure on the upstream side of the valve is less than the valve's pressure setting, flow through the valve is unrestricted. Pressure is limited to the Old town area of Avondale through three pressure reducing valves located in Van Buren Street at the Agua Fria River, in Dysart at Van Buren, and in Eliseo Felix Way at Van Buren. These valves have a pressure setting of 45 psi.

The pressure is limited to the Cashion Area through pressure reducing valves located in Buckeye Road and Lower Buckeye Road. The pressure in these valves is set at 55 psi.

The third area where pressures are limited in the system is in the Bohne area. The pressure reducing valves in the Bohne area are located in El Mirage Road north of Lower Buckeye, and just west of the Bohne booster pump station. The pressure reducing valves in the Bohne area are currently set at 55 psi.



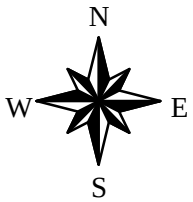


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Figure 5.2
Low Pressure Zones

Avondale Boundary
Low Pressure Zones
Streets

The three areas which are designated as the City of Avondale's low pressure zone are the three oldest areas in the distribution system. Consequently, due to the aging pipes in these areas, the pressures have been severely reduced in order to protect against pipe bursts.

5.6 Wetlands of Avondale

The City of Avondale has taken an innovative approach to effectively eliminate their dependence on groundwater to meet their domestic needs. In 1998, the City of Avondale constructed a wetlands treatment system as well as a recharge basin to help treat and recharge Avondale's Salt River Project (SRP) entitlements and Central Arizona Project (CAP) water allotments. The Wetlands of Avondale project receives high nitrate as well as high TDS surface water. The water passes through one of three parallel series of treatment cells. The first cell for each series removes solids and the remaining cells reduce the nitrogen concentration. The treated water is then recharged into the aquifer to provide storage credits. The Wetlands of Avondale treatment system can be seen in Figure 5.3. The Wetlands of Avondale project is permitted to treat and recharge up to 10,000 ac-ft of surface water per year.

Figure 5.3 Wetland of Avondale Treatment System

As the population in Avondale continues to grow, and the water demands increase accordingly, it is anticipated that the Wetlands of Avondale will play an even more important role in the City's water infrastructure.

5.7 Supervisory Control and Data Acquisition (SCADA) Controls

5.7.1 Current SCADA System

The City of Avondale's existing SCADA system consists of the sites described in Table 5.4, Existing Well and Reservoir Site Table. The location of these sites is shown in Figure 5.1, Existing Water Supply System. The Rancho Santa Fe site communicates with and controls Wells 6, 10, 11, and 12 via Microwave Data Systems (MDS) spread spectrum radios. Additionally, Well 18 is designed to

communicate with Rancho Santa Fe. The Modicon PLC at Rancho Santa Fe is programmed to act as a Modbus Master, and the remote sites are configured as Modbus Slaves.

Table 5.4 Existing Well And Reservoir Site Table

Well and Reservoir Sites	Information
Coldwater Springs	The Coldwater Springs Booster Pump Station is currently under construction. The design includes an MDS-9820 master radio, a Modicon Quantum Programmable Logic Controller (PLC) with hot backup, and an Omni directional antenna to be mounted 6 feet above the existing tank.
Well 15	Located on the Coldwater Springs site. Well 15 can be hardwired to the Coldwater Springs system.
Well 14	Well 14 is currently under construction. The design includes an MDS 9810 radio, Modicon 984 PLC, and a Yagi directional antenna.
Rancho Santa Fe	The Rancho Santa Fe site is equipped with a Modicon 984-E241 PLC, MDS 9310 radio and an Omni antenna mounted 10 feet above a 16-foot tank for an effective antenna height of 26 feet. The Rancho Santa Fe radio currently communicates with Well 10, Well 11, Well 12, and Reservoir 6.
Well 19	The Well 19 site is located approximately 400 feet from the Rancho Santa Fe telemetry unit. When this well is constructed, it can be hardwired to the Rancho Santa Fe telemetry unit.
Wells 10, 11 and 12	Each are equipped with a Modicon 984 Micro PLC, an MDS 9310 radio and a Yagi directional antenna mounted on a 20 foot mast.
Reservoir 6	The Reservoir 6 site (Northside) is equipped with an MDS 9310 radio and a Yagi directional antenna mounted 4 feet above a 16-foot tank. Well 7 is hardwired to Reservoir 6.
Reservoir 8 and Bohne Reservoir	Reservoir 8 and Bohne Reservoir are each equipped with a Modicon 984 Compact PLC, an MDS 9310 radio and a Yagi directional antenna. The antennas are mounted on 20-foot masts. These radios are not currently communicating.
Well 18	Well 18 was designed to include an MDS 9810 radio, Modicon 984 PLC, and a Yagi directional antenna mounted 3 feet above the block wall.
Reservoir 1, Reservoir 2 (Hill St. Booster) and Reservoir 5	Reservoirs 1, 2 and 5 do not have a PLC or radio. These sites currently operate under local control.
Cashion Reservoir	The Cashion Reservoir currently operates under local control. The upgrade design includes a Modicon 984 Micro PLC, an MDS 9810 radio, and a Yagi directional antenna mounted on a 20 foot mast.

Modbus RTU is the communication protocol used by the PLCs to communicate with each other. The radio-modems are the communication medium. Each radio-modem connects to a PLC using an RS-232 serial cable. The port configuration is 8 data bits, no parity, one stop bit, and the baud rate is 9600. The MDS radios are said to be "transparent" to the communication protocol, meaning that the radios faithfully transmit and receive the Modbus messages between PLCs without distorting the messages. However, if the radio signals are weakened by distance, atmospheric conditions, geographical or structural obstacles, the messages can become distorted or corrupted. Even poorly installed cables, connectors, and antennas can cause attenuation of the radio signal. Therefore, the Modbus RTU protocol at each PLC checks the messages, and if they have been corrupted the protocol rejects the bad message and requests a new message from the sender. Hence, the Modbus RTU protocol protects the PLCs from acting on bad data that could cause control system problems.

The City of Avondale has standardized on the Modbus RTU protocol and is currently utilizing Modicon PLCs. However, other PLC manufacturers such as Allen-Bradley and Siemens produce PLCs capable of communicating via the Modbus RTU protocol. Therefore, the City is not "locked in" to one PLC manufacturer, because the Modbus RTU protocol is considered an "open" protocol.

5.7.2 System Requirements

The City of Avondale requires the ability to monitor and control operations at remote well and reservoir sites. This monitoring can be achieved by installing a Remote Telemetry Unit (RTU) at each remote site not currently equipped with an RTU. Each RTU will contain a Programmable Logic Controller (PLC) and radio equipment. The PLC will monitor and control the facility. The PLC will receive command and setpoint instructions from a Master Telemetry Unit (MTU) via the radio-modem. The PLC will then control the facility based on the command setpoints received from the MTU. The PLC will pass the status of local operating parameters to the radio, which will transmit this information back to the MTU.

5.7.3 Radio Survey

A field survey of 17 sites was conducted on December 27th, 2001. The longitude and latitude coordinates of each site were recorded using a global positioning system receiver. A table of these sites and coordinates is shown on Table 5.5, Site Coordinates Table. A preliminary study of input/output requirements was conducted at sites that currently do not have PLC's. The purpose of this study was to determine the size of the PLC needed for each site. The size of the PLC was used in calculating the opinion of probable cost. The exact location of the Garden Lakes Reservoir, Well 16, Well 17, and Phoenix International Raceway were not known at the time of the field survey and have not been included in the radio path study.

Table 5.5 Site Coordinates Table

Site Name	Latitude	Longitude
Reservoir Bohne	33 24 53.6 N	112 19 31.0 W
Cashion Reservoir & Well	33 25 44.2 N	112 17 23.4 W
Coldwater Springs Booster Pump Station	33 26 56.9 N	112 19 47.8 W
Reservoir Rancho Santa Fe I & II	33 27 50.4 N	112 20 00.4 W
Reservoir No. 1 & Well No. 1	33 26 19.6 N	112 20 35.8 W
Well No. 10	33 28 42.7 N	112 19 15.9 W
Well No. 11	33 28 30.5 N	112 19 18.5 W
Well No. 12	33 28 16.8 N	112 19 27.3 W
Well No. 14	33 27 11.3 N	112 20 12.6 W
Well No. 18	33 27 50.2 N	112 20 20.9 W
Well No. 19	33 27 50.8 N	112 19 56.8 W
Reservoir No. 2	33 26 10.5 N	112 20 53.0 W
Reservoir No. 5 & Well No. 5	33 25 27.6 N	112 20 49.0 W
Reservoirs 6, 6A, & 6B	33 29 31.3 N	112 19 25.7 W
Well No. 7	33 29 41.9 N	112 19 26.1 W
Reservoir No. 8 & Well No. 8	33 28 06.0 N	112 16 19.3 W
Well No. 9 (Abandoned)	33 25 54.6 N	112 17 51.4 W

Software generated Path Profiles are included for all 13 paths considered for the non-licensed, 928 MHz band, Multiple Address System (MAS) radio network. The path profiles were generated using a US Geological Survey (USGS) 30-meter point elevation digitized terrain database. The path profiles were displayed and then plotted. The path profile provides a visual means for determining if line-of-sight is practical for a given path. The path profiles provide a convenient means for determining if the viability of a particular path warrants further analysis of that path.

Microwave Budget Link Analysis provides the capability for studying the complete radio system of each path including the transmitter power output, the passive elements of the transmitter antenna feed system, the transmit antenna, free-space path, propagation loss, receive antenna, the passive elements in the receive antenna feed system, and the receiver sensitivity so that a reliability prediction can be calculated. The free-space path loss is calculated using a selected prediction model (e.g. –Lenkurt or Vigants) and climate and terrain factors. The propagation loss is calculated using the Longly-Rice prediction model. The Microwave Budget Link Analysis can be applied to reduce the total amount of actual on-the-air testing required.

A summary of Received Signal Strength (RSL) and total fade margin (TFM) for each path can be found in Table 5.6, Signal Strength and Total Fade Margin. Appendix D contains additional information from the radio survey. Plots of the Path Profiles can be found in Appendix D, Figures 1 through 13. Propagation Loss can be found in Appendix D, Tables 1a through 13a. Microwave Budget Link Analysis can be found in Appendix D, Tables 1b through 13b.

Table 5.6 Signal Strength and Total Fade Margin

Radio Path Endpoints		Dist.	Propagation Loss	RSL	TFM
From	To	Miles	dB	dBm	dB
Coldwater Springs	Rancho Santa Fe	1.04	2.85	-55.78	54.22
Coldwater Springs	Well No. 14	0.48	0.24	-42.44	67.56
Rancho Santa Fe	Reservoir Bohne	3.42	5.50	-64.76	45.24
Rancho Santa Fe	Cashion Reservoir & Well	3.49	5.84	-65.17	44.83
Rancho Santa Fe	Reservoir Nos.6, 6A & 6B	2.01	6.43	-60.74	49.26
Rancho Santa Fe	Reservoir No. 1 & Well No. 1	1.83	3.51	-57.95	52.05
Rancho Santa Fe	Well No. 10	1.23	7.00	-56.72	53.28
Rancho Santa Fe	Well No. 11	1.02	5.68	-53.77	56.23
Rancho Santa Fe	Well No. 12	0.73	17.46	-62.65	47.35
Rancho Santa Fe	Well No. 18	0.33	0.00	-38.29	71.71
Rancho Santa Fe	Reservoir No. 2	2.09	3.79	-58.77	51.23
Rancho Santa Fe	Reservoir No. 5 & Well No. 5	2.84	6.22	-63.76	46.24
Rancho Santa Fe	Reservoir No. 8 & Well No. 8	3.56	7.89	-67.17	42.83

A summary of the findings is listed in Table 5.7, Path Study Results. Its important to note that 40 foot towers would require a Special Use Permit.

Table 5.7 Path Study Results

Path Definition	Study Results
Coldwater Springs to Well 14	This path has line of sight and a total fade margin of 67.56 dBm.
Coldwater Springs to Rancho Santa Fe	This path has line of sight and a total fade margin of 54.22 dBm. Rancho Santa Fe will require a 40-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Well 10	This path has line of sight and a total fade margin of 53.28 dBm.
Rancho Santa Fe to Well 11	This path has line of sight and a total fade margin of 56.23 dBm.
Rancho Santa Fe to Well 12	Although this path has been in operation for approximately six years, it does not have line of site, and it has a 17.46 dB loss due to terrain interference. The successful operation of this path is probably due to the relatively short path length.
Rancho Santa Fe to Reservoir 1 & Well 1	This path has line of sight and a total fade margin of 52.05 dBm. Reservoir 1 will require a 40-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Reservoir 2 (Hill St. Booster Station)	This path has line of sight and a total fade margin of 51.23 dBm. Reservoir 2 will require a 35-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Well 18	This path has line of sight and a total fade margin of 71.71.
Rancho Santa Fe to Reservoir 5 (Mountain View)	This path has line of sight and a total fade margin of 46.24dBm. Reservoir 5 will require a 30-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Reservoir 8	This path has line of sight and a total fade margin of 42.83 dBm. Reservoir 8 will require a 30-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Cashion Reservoir	This path has line of sight and a total fade margin of 44.83 dBm. Cashion Reservoir will require a 30-foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Bohne Reservoir	This path has line of sight and a total fade margin of 45.24 dBm. The Bohne Reservoir will require a 35 foot antenna height to achieve line of sight for this path.
Rancho Santa Fe to Reservoir 6 (Northside)	This path has line of sight and a total fade margin of 49.26 dBm.

Recommendations for developing a comprehensive SCADA system for all of the City of Avondale sites is provided in Section 8.1.

5.8 Private Water Companies

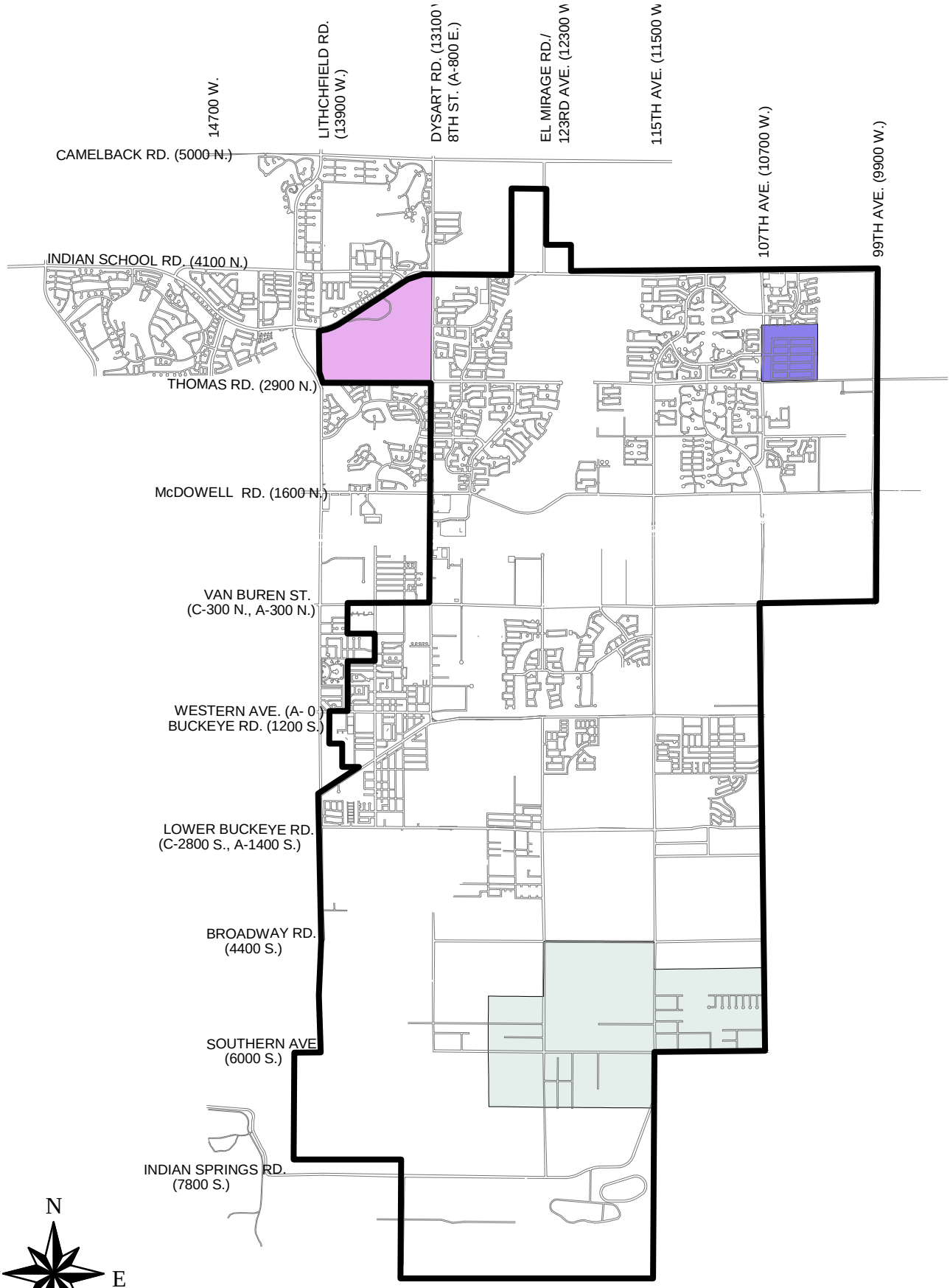
There are three private water companies located within the City of Avondale. These private water companies are the Litchfield Park Service Company (LPSCO), Wilhoit Water Company (Wilhoit), and Rigby Water Company (Rigby). The service area locations for these water companies are shown in Figure 5.4, Private Water Companies.

According to the Arizona Corporation Commission, LPSCO provides water service to portions of Avondale, Goodyear and Litchfield Park. LPSCO provides water service to the portion of Avondale that falls within Section 27, Township T2N, Range R1W (west of Dysart Road between Thomas Road and Indian School Road). LPSCO currently does not serve residential customers within Avondale but serves Estrella Mountain Community College, a commerce park, and several churches. LPSCO has a 6 million gallon storage tank located on the southeast corner of Indian School Road and Dysart Road. The water provided to its service area in Avondale comes from two active LPSCO wells and is distributed through 12-inch waterlines. LPSCO would be willing to discuss the possible acquisition by the City of Avondale but LPSCO feels they are better suited to serve the area.

The Wilhoit Water Company encompasses a quarter section of land located on the northeast corner of 107th Avenue and Thomas Road (SW1/4, Section 29, Township T2N, Range R1E. Wilhoit has approximately 140 water service connections within the Glenn Arm Farms community. Wilhoit's water supply is provided through a single active well. The City of Avondale has a fire line along Earll Rd. to provide fire service to Glenn Arm Farms. However, this fire line is not connected to the Wilhoit water system. The Wilhoit Water Company would be willing to discuss the purchase of their system by the City.

The Rigby Water Company is generally located between Broadway Road and the Gila River, east of 127th Avenue alignment. Rigby has 320 water service connections and is served by four Rigby wells. The distribution system consists primarily of 4-inch and 6-inch lines. However, it does include some 8-inch lines. The property management company for the Rigby Water Company has expressed interest in discussing the purchase of the system by the City of Avondale.

If Avondale should purchase any of the private water companies, the City should consider assessing a monthly charge to the water users within the private water company area for repayment of the purchase price for the water company. At such time as Avondale may consider acquisition of private water companies, the City should seek council from a bonding attorney for identifying procedures in establishing repayment charges.



Avondale Boundary
LPSCO
Wilhoit Water Company
Rigby Water Company
Streets

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Figure 5.4
Private Water Companies

5.9 Photographic Inventory

In order to provide a better understanding of each of the City's well, tank, and booster station sites, a photographic inventory is included here which provides a photograph of the well, tank, and booster pumps at each of the City's sites (see Figure 5.5 to Figure 5.24). It is important to note that while some of these sites are referenced by a well site number, the well at the site may have been abandoned, and therefore no well is shown.

5.9.1 Well Site #1

Figure 5.5 Well Site #1 Well

Figure 5.6 Well Site #1 Tank

Figure 5.7 Well Site #1 Booster Pumps

5.9.2 Hill Street Booster Pump Station (Well Site#2)

Figure 5.8 Hill Street Tank

Figure 5.9 Hill Street Booster Pumps

5.9.3 Mountain View Booster Station (Well Site #5)

Figure 5.10 Mountain View Tank

Figure 5.11 Mt View Booster Pump

5.9.4 North Side Station

Figure 5.12 Well Site #6

Figure 5.13 North Side Tank

Figure 5.14 North Side Pumps

Figure 5.15 Well Site #7

5.9.5 Well Site #8

Figure 5.16 Well Site #8 Well

Figure 5.17 Well Site #8 Tank

Figure 5.18 Well Site #8 Pumps

5.9.6 Well Site #10

Figure 5.19 Well Site #10

5.9.7 Well Site #11

Figure 5.20 Well Site #11

5.9.8 Well Site #12

Figure 5.21 Well Site #12

5.9.9 Well Site #18

Figure 5.22 Well Site #18

5.9.10 Rancho Santa Fe

Figure 5.23 Rancho Santa Fe Tanks

Figure 5.24 Rancho Santa Fe Pumps

6.0 Supply and Storage Analysis

6.1 Supply and Storage Analysis

In the 1996 *Water Master Plan*, a series of seven reliability tests were designed to aid the City in the development of their water infrastructure. These reliability tests evaluate the capacity of the storage, supply, and booster pumps in the water distribution system, to see if the system is capable of providing adequate pressure, flow, and fire flow. The existing system (2001) and future water systems (2006, 2011, 2026, and 2040) were evaluated according to these reliability test criteria. These evaluations were used to determine additional amounts of storage, supply, and booster pump capacities that would need to be constructed each planning period to satisfy the testing criteria.

In order to evaluate the water system, the average day, maximum day, and peak hour flow rates for each of the design periods were computed. Table 6.1, Demand Conditions for 2001, 2006, 2011, 2026, 2040, lists the projected demand conditions for the design periods. In computing the future flow rates required for each of the planning periods, design values established in Section 4 were used. These values include an average day flow of 200 gpcd and peaking factors of 2.0 times average day and 1.7 times max day for maximum day and peak hour flows respectively. However, the actual average day flow for the existing system (2001) was used since this information was available from pumping records. The maximum day and peak hour peaking factors were utilized for the 2001 system demands.

Table 6.1 Demand Conditions for 2001, 2006, 2011, 2026, 2040

Year	Population	Avg Day (gpm)	Max Day (gpm)	Peak Hour (gpm)
2001	40,350	4,944*	7,494	12,739
2006	61,845	8,590	17,179	29,205
2011	83,506	11,598	23,196	39,433
2026	148,489	20,623	41,247	70,120
2040	167,665	23,287	46,574	79,175

*Actual average day and maximum day consumption from pumping records.

The required city wide fire flow that was computed and used for the reliability tests is listed in Table 6.2, Required Fire Flow below. These fire flows were calculated from an empirical flow equation developed by the National Board of Fire Underwriters to estimate the fire demand required in downtown business districts. The equation used to develop these flows is shown in Figure 6.1, Fire Flow Equation. These fire flows differ from the flows described in Section 4.3, which provided information on fire flows for a specific building or development. Table 6.2 lists the required fire flow for each of the planning periods.

$$Q = 3.86 \times \sqrt{P}\left(1 - 0.01 \times \sqrt{P}\right)$$
$$Q = \text{flow rate } (m^3/\text{min})$$
$$P = \text{population (thousands)}$$
$$1\ m^3 = 264.17\ gal$$

Figure 6.1 Fire Flow Equation

Table 6.2 Required Fire Flow

Year	Population	Fire Flow (gpm)
2001	40,350	6,066
2006	61,845	7,388
2011	83,506	8,467
2026	148,489	10,912
2040	167,665	11,494

6.1.1 Existing Reliability Tests

The seven reliability tests that are currently utilized by the City of Avondale are described below.

- **Ultimate Source-** The system should supply the maximum day demand with the largest well out of service.

- **Reliable Source-** The system should supply the maximum day demand with all wells operating no more than 18 hours.

- **Peak Hour Storage-** The system should have sufficient storage to meet the peak hour demand for four hours without depleting storage more than fifty percent.

- **Fire Flow-** The system should meet the fire flow demand concurrent with the maximum day demand, utilizing supply sources and eighty percent of the total available storage.

- **Operating Storage-** The total storage should supply twenty percent of the maximum day demand.

- **Emergency Supply-** The average day demand should be met using eighty percent of the total available storage without any well supply.

- **Booster Pump Capacity-** Booster pumps should be able to supply the larger of the maximum day plus fire demand or the peak hour demand without the single largest pump.

The City's existing water distribution system was analyzed based on the seven reliability tests listed above. The water system was analyzed for each of the planning periods to determine if the system has sufficient resources to meet the reliability tests, and if not to identify the improvements required.

6.2 Existing System Evaluation

The existing system (2001) was evaluated based on the defined reliability tests. At the present time, the City of Avondale has an actual well supply of 13,311 gpm (see Table 5.1). The total available storage in the City's water system is 5,400,000 gallons (see Table 5.2). Each of the reliability tests was evaluated based on the estimated demands to determine the deficiencies in the system.

6.2.1 Ultimate Source Capacity

In the Ultimate Source Capacity test, the system should supply the maximum day demand with the largest producing well (Well 18) out of service. The 2001 maximum day demand is 9,888 gpm (see Table 6.1). The total well capacity without Well 18 is 11,010 gpm (13,311 gpm – 2,301 gpm). Since the maximum day demand is less than the total well capacity with the largest producing well out of service, the system meets the ultimate source capacity test.

6.2.2 Reliable Source Capacity

To prove reliable source capacity, the system must supply the maximum day demand with all wells operating no more than 18 hours. The maximum day demand for the City of Avondale is 9,888 gpm or 14.24 MGD. The total well capacity for 18 hours is 14.38 MGD (13,311 gpm x 60 min/hr x 18 hr/day). Since the maximum day demand is less than the 18 hour well capacity, the system meets the reliable source capacity test.

6.2.3 Peak Hour Storage

The system must have sufficient storage to meet the peak hour demand for the City of Avondale for 4 hours without depleting storage more than 50%. The peak hour demand for four hours requires 4,034,400 gallons (16,810 gpm × 240 min). 50% of the present day available storage is 2,700,000 gallons (0.50 × 5,400,000 gal). The peak hour demand for 4 hours is greater than 50% of the storage capacity. Therefore, the system does not meet the requirement for peak hour storage and is deficient approximately 2,668,800 gallons.

6.2.4 Fire Flow

The system must meet the fire flow requirement concurrent with the maximum day demand. To satisfy this requirement, the system may utilize supply sources and eighty percent of the total available storage.

The estimated max day demand (from Table 6.1) and Fire Flow demand (from Table 6.2) for the City of Avondale is 15,954 gpm (9,888 gpm + 6,066 gpm). The

volume required to serve the fire flow demand concurrently with the maximum day demand for 5 hours is 4,786,200 gallons (15,954 gpm x 60 min/hr x 5 hrs).

The summation of the supply sources and 80% of the available storage is 8,313,300 gallons [(13,311 gpm x 60 min/hr x 5 hrs) + (0.8 x 5,400,000 gallons)]. The system is able to meet the fire flow test requirements.

6.2.5 Operating Storage

The total storage in the system should supply 20% of the maximum day demand. Twenty percent of the maximum day demand is 2,847,744 gallons (9,888 gpm × 0.20 × 1440 min). The total available storage is 5,400,000 gallons. Therefore, the system is able to meet the operating storage requirements.

6.2.6 Emergency Supply

The average day demand should be met by the system using eighty percent of the total available storage without using any well supply. The average day demand for the City of Avondale in 2001 is 7,118,921 gallons (Appendix B-1). Eighty percent of the total available storage is 4,320,000 gallons (5,400,000 gallons × 0.80). Therefore, the system does not meet the requirement for emergency storage and is deficient approximately 3,498,651 gallons.

6.2.7 Booster Pump Capacity

Without the largest pump operating, booster pumps should be able to supply the larger of the maximum day demand plus the fire flow or the peak hour demand. The maximum day demand plus fire flow is 15,954 gpm (maximum day demand + fire flow = 9,888 gpm + 6,066 gpm). The peak hour demand is 16,810 gpm (from Table 6.1). The peak hour demand is the larger of the two types of demand.

The total existing booster pump capacity for the City of Avondale is 27,590 gpm (see Section 5.3). The largest pump (located at the Rancho Santa Fe tank site) delivers 2,200 gpm. With the largest pump out of service there remains 25,390 gpm of booster pump capacity to provide for the 16,810 gpm which corresponds to the peak hour demand. Therefore, the system is able to meet the booster pump capacity requirements.

6.2.8 Summary

The existing water system was evaluated against the criteria established in the seven reliability tests. The existing system analysis did not include the 2,500,000 gallon storage tank which is being constructed at Coldwater Springs, as it has not been put online yet. The evaluation shows that for the 2001 system, five of the seven reliability tests pass the requirements. Two tests that failed the reliability tests were Peak Hour Storage and Emergency Supply. Table 6.3, Supply/Storage Results – Existing Reliability Tests, provides a summary of the deficiencies found in the existing system.

Table 6.3 2001 Supply/Storage Results - Existing Reliability Tests

Test	Pass/Fail	Deficiency
Ultimate Source Capacity	Pass	
Reliable Source Capacity	Pass	
Peak Hour Storage	Fail	2,668,800 gal
Operating Storage	Pass	
Fire Flow	Pass	
Emergency Supply	Fail	3,498,651 gal
Booster Pump Capacity	Pass	

6.3 Immediate Improvements System Evaluation

The seven reliability tests were re-evaluated addressing deficiencies recognized in the existing system. For purposes of this analysis, supply was added in 100 gpm increments, storage was added in 100,000 gallon increments, and booster pump capacity was added in 1,000 gpm increments until all of the reliability tests were satisfied.

In order to meet the reliability tests for the existing system demands 3,600,000 gallons of storage are required. Once this improvement has been made, the existing system will meet the reliability tests established. As noted earlier, this evaluation does not include the 2.5 MG reservoir at Coldwater Springs since the reservoir is not yet online.

6.4 Future Improvements System Evaluation

The proposed systems were also analyzed based on the seven reliability tests. For each of the planning periods, the reliability tests were analyzed utilizing the projected demand and fire flow data found in Table 6.1, Demand Conditions for 2001, 2006, 2011, 2026, 2040, and Table 6.2, Required Fire Flows. The deficiencies in the system for each of the planning periods were noted, and improvements were added as described in the previous section until the system met the reliability test requirements. Table 6.4, Planning Period Storage and Supply Improvements, illustrates the additional improvements which must be added during each of the planning periods to meet the test requirements.

Table 6.4 Planning Period Storage and Supply Improvements

	2001	2006	2011	2026	2040	Total
Additional Supply (gpm)	0	9,700	8,000	24,000	7,100	48,800
Additional Storage (gal)	3,500,000	6,500,000	5,400,000	16,300,000	4,800,000	36,600,000
Additional Booster Pumps (gpm)	0	4,000	10,100	30,700	9,000	53,800

6.5 Modified Reliability Tests

Due to various changes in the City's water system over the last five years, as well as changes in technology, it is recommended that some of the reliability

tests be updated to better meet the City's current situation. The recommended reliability tests are described in the following.

Based on the analysis of the existing (2001) water distribution system, as well as the proposed recommended water distribution systems for the future planning scenarios (2006, 2011, 2026, 2040), it was seen that the *Peak Hour Storage* requirement and the *Emergency Supply* requirement always control the amount of storage that is required in the system. It is recommended that both of these reliability tests be modified to better represent the system as it currently exists.

The City of Avondale currently has a large wetlands treatment area which treats and recharges surface water to the groundwater aquifer. This groundwater aquifer acts as an underground storage system which allows the City of Avondale to withdraw water at almost any time (within the legal requirements described in the 2001 *Water Resources Master Plan*). It is proposed that the City begin utilizing this underground storage system to meet some of the reliability test criteria. Additionally, when the 1996 *Water Master Plan* was completed relatively few of the wells in Avondale had any type of backup power supply. Today, many of the wells in the City's system have backup power generators. It is also recommended that a backup power supply be provided for all new wells constructed for the City's water system.

With the changes that have occurred to the City's water system, it is recommended that two of the reliability tests be changed. The *Peak Hour Storage* reliability test requires that the Peak Hour demands be satisfied for four consecutive hours without depleting storage more than 50%. It is recommended that the *Peak Hour Storage* requirement be modified as follows:

> The *Peak Hour Storage* test requires that the Peak Hour demand be satisfied for four consecutive hours without depleting storage by more than 50% and utilizing only 50% of the well supply.

The *Emergency Supply* reliability test requires that the Average Day demand should be satisfied using 80% of the total available storage without any well supply. This definition for the reliability test doesn't take into consideration either the underground storage within the system or the backup power supply on the various wells. It is recommended that the *Emergency Supply* requirement be modified as follows:

> The *Emergency Supply* reliability test requires that the Average Day demand should be met using only 80% of the total available storage and 50% of the total well supply, with no wells operating longer than 18 hours.

Modifying the *Peak Hour Storage* and *Emergency Supply* tests as described above will provide for more cost effective system design and expansion by

eliminating excessive water storage reservoirs. These modifications are recommended since many of the wells in Avondale are currently equipped with backup generators. Additionally, APS the power provider for the majority of Avondale was contacted, and it was determined that rarely within the Avondale service area does a power failure exceed more than two hours. The majority of the time, a power failure is corrected in under an hour. Based on the above information, the modified peak hour storage test will allow a more cost effective system to be developed without compromising system performance.

In order to develop an efficient, yet cost effective water system, it is recommended that the City of Avondale use the modified Peak Hour Storage and Emergency Supply tests. **Therefore, the water system storage requirements recommended in this report and described in the CIP are based on the findings incorporating these modified tests.**

The water systems were evaluated for each of the planning periods utilizing the two modified tests in addition to the other five original tests. When the modified tests are used, a significant reduction in the required amount of storage will be realized.

6.6 Immediate Improvements System Evaluation (Modified Reliability Tests)

The existing system (2001) was analyzed again utilizing the two modified reliability tests. By taking into consideration the underground storage within the City as well as the backup power supply at the wells, the existing water system passes all of the reliability tests and no immediate improvements are required. However, while the current system meets these reliability requirements, improvements must still be made to the system as the population in the City grows, and to improve system performance in deficient areas.

6.7 Future Improvements System Evaluation (Modified Reliability Tests)

Table 6.5, Planning Period Storage and Supply (with Modified Reliability Tests) shows the required improvements, which must be constructed for each of the planning periods, based on the modified reliability tests. The improvements listed in the table are the additional improvements that must be completed during each planning period, assuming the improvements for the previous planning periods were completed during that period. For example, 8,000 gpm of supply must be added to the system by 2006, and then an additional 8,000 gpm must be added to the system by 2011. Therefore, 16,000 gpm of supply must be added to the system over these two planning periods.

Table 6.5 Planning Period Storage and Supply (Modified Reliability Tests)

	2001	2006	2011	2026	2040	Total
Additional Supply (gpm)	0	9,700	8,000	24,000	7,100	48,800
Additional Storage (gal)	0	3,100,000	3,000,000	9,000,000	5,800,000	20,900,000
Additional Booster Pumps (gpm)	0	3,900	10,200	30,700	9,000	53,800

As can be seen in a comparison between Table 6.4, Planning Period Storage and Supply Improvements and Table 6.5, Planning Period Storage and Supply (Modified Reliability Tests), a savings can be realized when utilizing the modified Peak Hour Test due to a reduction in required storage (3,500,000 gallons in 2001, and 15,700,000 gallons overall). These savings can be realized without compromising the reliability of the City's water supply system. Therefore, improvements required using the modified reliability tests will be recommended in the development of the CIP.

7.0 Hydraulic Model Analysis

7.1 Hydraulic Model

A computer hydraulic model was developed for the existing water distribution system. The model contained all of the main distribution lines within the City of Avondale. The main lines 8-inches and larger were included in the system. In select locations, various 6-inch lines were also included where these lines are serving as distribution mains. The pipe locations and sizes were identified with the help of City staff, until a final agreed upon layout was established. The pipe layout for the existing system is shown in Figure 7.1, 2001 System Map.

The water modeling system, which was utilized for this model, was the H_2ONet modeling system developed by MW Soft Inc. The water model was used to perform a steady-state analysis on the water system. A “steady-state” analysis provides a look at what the water system is doing at a single instance in time. The water system was evaluated to see how well pressure and flow are provided throughout the system, and to identify the areas that need improvements.

The water distribution system was modeled for the 2001, 2006, 2011, 2026, and Ultimate system. The improvements which were required in order to help the system meet the design requirements were identified and included in the model. The required improvements along with their associated costs are outlined in the capital improvement program which was developed, and is included in Section 9.

For the 2001, 2006, and 2011 scenarios, pressure contour maps of the results are provided. These pressure contours reflect the conditions of the water system for a specific instance in time and a specific condition, and therefore may not necessarily reflect field conditions on a given day.

7.2 Assumptions

The City of Avondale water distribution system was modeled as a steady-state model. This type of model provides a picture of what the model is doing in a single instance of time. By nature of a steady state water model, all flow into the system is provided from the tanks in the model. Filling and draining of the reservoirs is not analyzed in a steady-state model. In order to create a representative model of the system, various assumptions were required. This section outlines the assumptions that were made when developing the model.

The water lines, which were placed in the system, were identified according to discussions with City staff. The size and placement of the waterlines was verified by the City of Avondale Development Services Department, Engineering Division and Utilities Department. These sizes and locations are considered to be the correct sizes and locations and are utilized for this model.

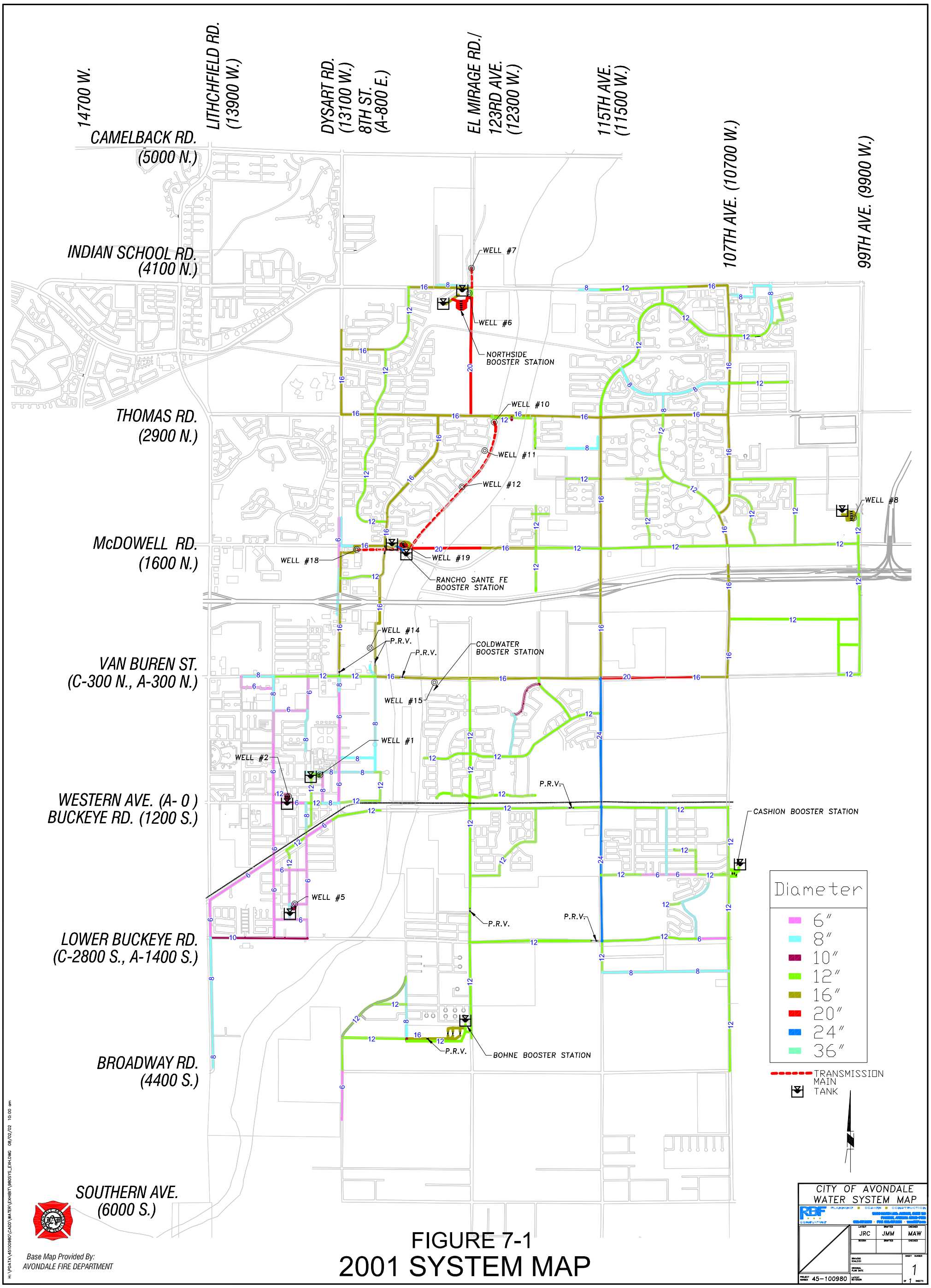

FIGURE 7-1
2001 SYSTEM MAP

One of the key factors in setting up a water model is determining accurate demands. The demands were determined based on the factors established in Section 3 and Section 4 of this water master plan. The demands were distributed throughout the model according to the patterns of residential development. A composite water demand factor was utilized, and it was assumed that for the purposes of this model the commercial demands would grow in accordance with the residential demands. This may vary depending on the ultimate growth pattern and land use within the City of Avondale. The demands for future scenarios were distributed based on the projected populations according to the 1993 TAZs. This is described in greater detail in Section 3 of this report.

The pump curves which were input into the model also have a dramatic effect on the results of the model. Since no pump testing was performed for this study, actual pump performance data was not attained. The pumping curves which were input into the model were based on information received from the City of Avondale Utilities Department and from the pump manufacturers. Therefore the pumping performance as shown in the model may differ from actual pump performance. It is recommended that the City of Avondale perform periodic pump testing, to verify the field performance of their booster pumps and to improve the accuracy of the City's hydraulic model.

Pipe roughness values also have a large effect on the results of a hydraulic model. C-value testing was not performed as part of this study, and therefore the roughness values for the pipe were assumed. A C-value of 130 was typically assumed for the piping in the water model. In the Old Town area, where the pipes are known to be significantly older than the rest of the system, a C-value of 70 was assumed. This was done to take into consideration the effects of corrosion buildup in the pipes, and the reduced efficiency which is found in aging pipes.

7.3 Calibration of Existing System

After the water model was setup, information that was obtained from various fire flow tests was used to calibrate the model to match existing conditions. Fire flow tests were taken throughout the City to determine the actual pressure and flow found in various parts of the system. These fire flow tests were then used to calibrate the water model to match the actual system performance as close as possible.

Fire flow tests were performed throughout the City from November 2 to November 16, 2001. Five fire flow tests were selected in representative portions of the City to be used in the calibration. The fire flow tests that were used in the calibration are shown in Table 7.1, Fire Flow Tests, along with their corresponding results. The results from the fire flow tests are shown in Appendix C-7, Water Flow Test Data.

Table 7.1 Fire Flow Tests

Location	Date	Time	Flow	Static	Residual
Cashion	11/6/01	10:07	924	60	50
Harbor Shores/ Coronado	11/5/01	12:52	1026	52	48
13015 N Rancho Santa Fe	not dated	12:05	1160	60	59
Desert Flower/ 125 Ln	11/2/01	11:20	1119	60	58
4th St/Harrison	11/16/01	10:00	939	42	38

The fire flow tests were input into the model individually, and the model was calibrated to match the existing fire flows as close as possible. The main parameters which were adjusted to meet the fire flow demand were the pump On-Off operations and the pipe C-values. The model was calibrated to provide the best pressure match for each of the fire flow static pressures simultaneously. Therefore, the overall model provides a good representation of the actual behavior of the distribution system, but was not narrowed down on one particular section of the City.

Once the model was calibrated, the water system was analyzed for the five planning periods.

7.4 Existing System

The existing water system was modeled to identify the areas where the most attention needs to be focused. For each model, four scenarios were analyzed. The scenarios analyzed included the average day, maximum day, peak hour, and maximum day with fire flow. For each of the scenarios evaluated, the system should maintain an acceptable system pressure of at least 40 psi, and not greater than 80 psi. The maximum day with fire flow scenario however, should maintain a residual pressure of 20 psi at all points throughout the system.

For each of the scenarios evaluated, the maximum and minimum pressure is provided, as well as the node number. A complete listing of the results for each of the nodes is provided in Appendix C.

The existing system was created to represent the system as it stands today, with no system improvements. The average day, maximum day, and peak hour scenarios were evaluated for this system, and the results can be seen in Table 7.2, Existing System Model Results. The pressure results exclude the values for nodes which are located between the reservoirs and tanks (pump-in nodes). Pressure contour maps of the results are provided in Figure 7.2 – Figure 7.4.

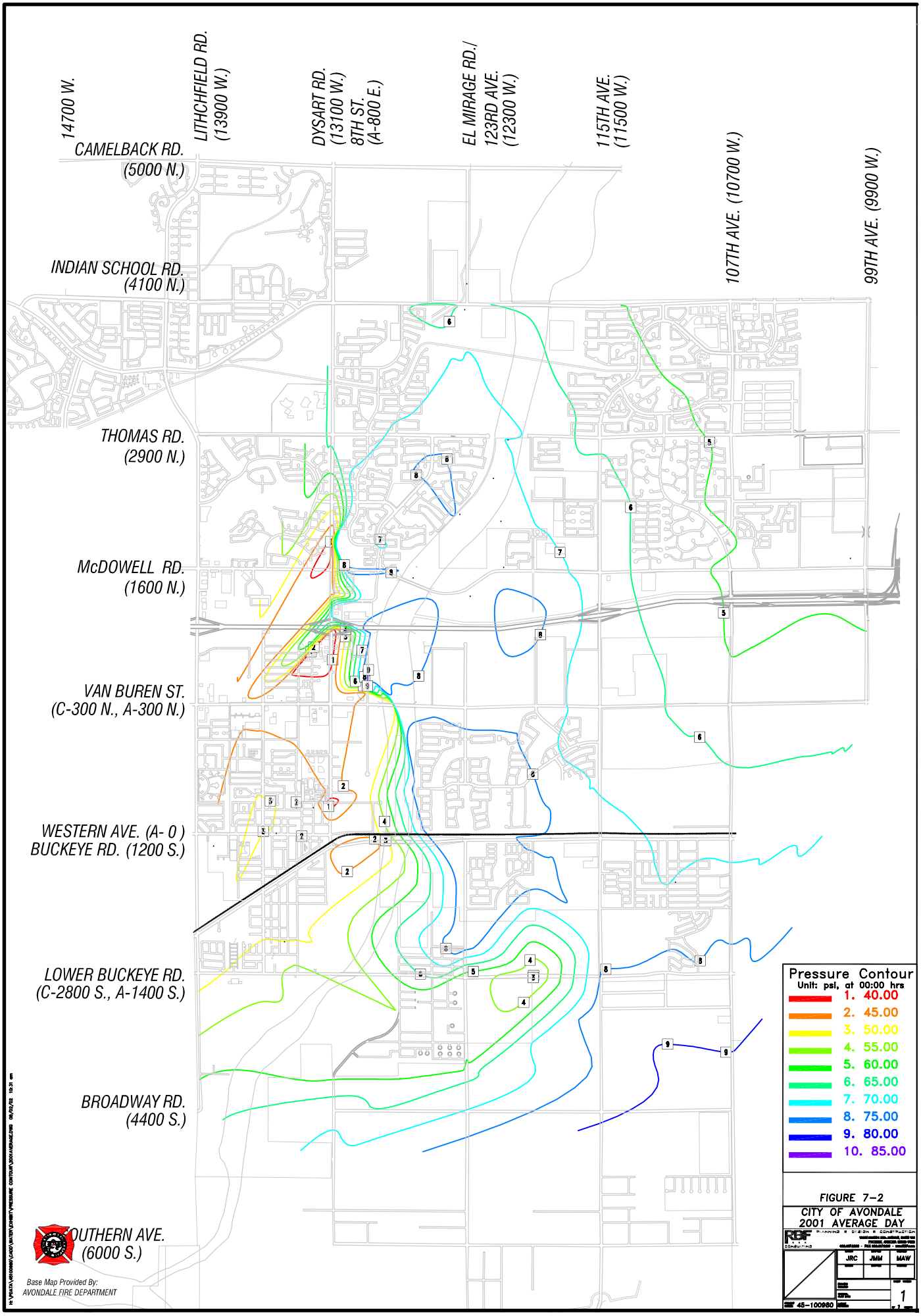

FIGURE 7-2
CITY OF AVONDALE
2001 AVERAGE DAY

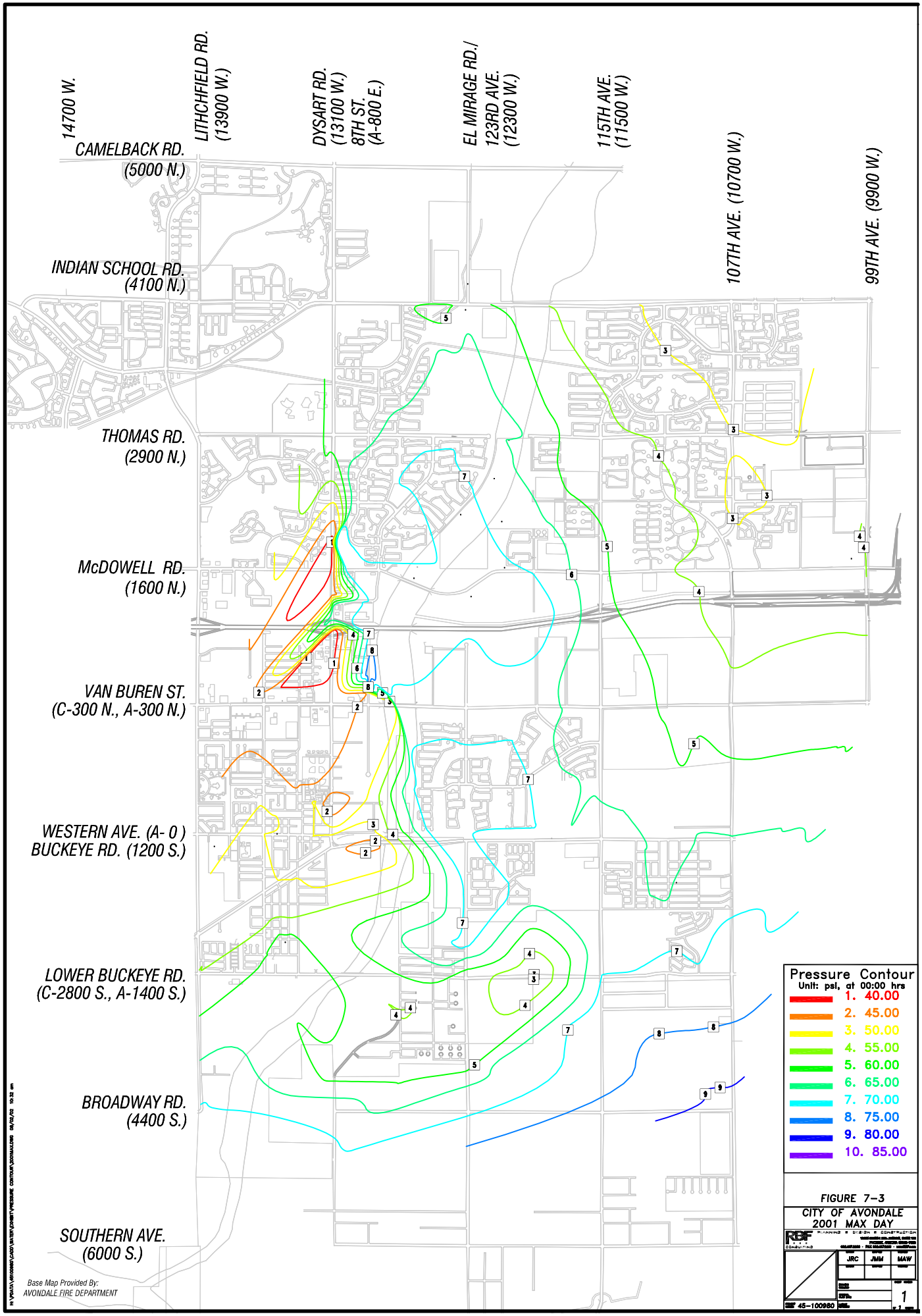

FIGURE 7–3
CITY OF AVONDALE
2001 MAX DAY

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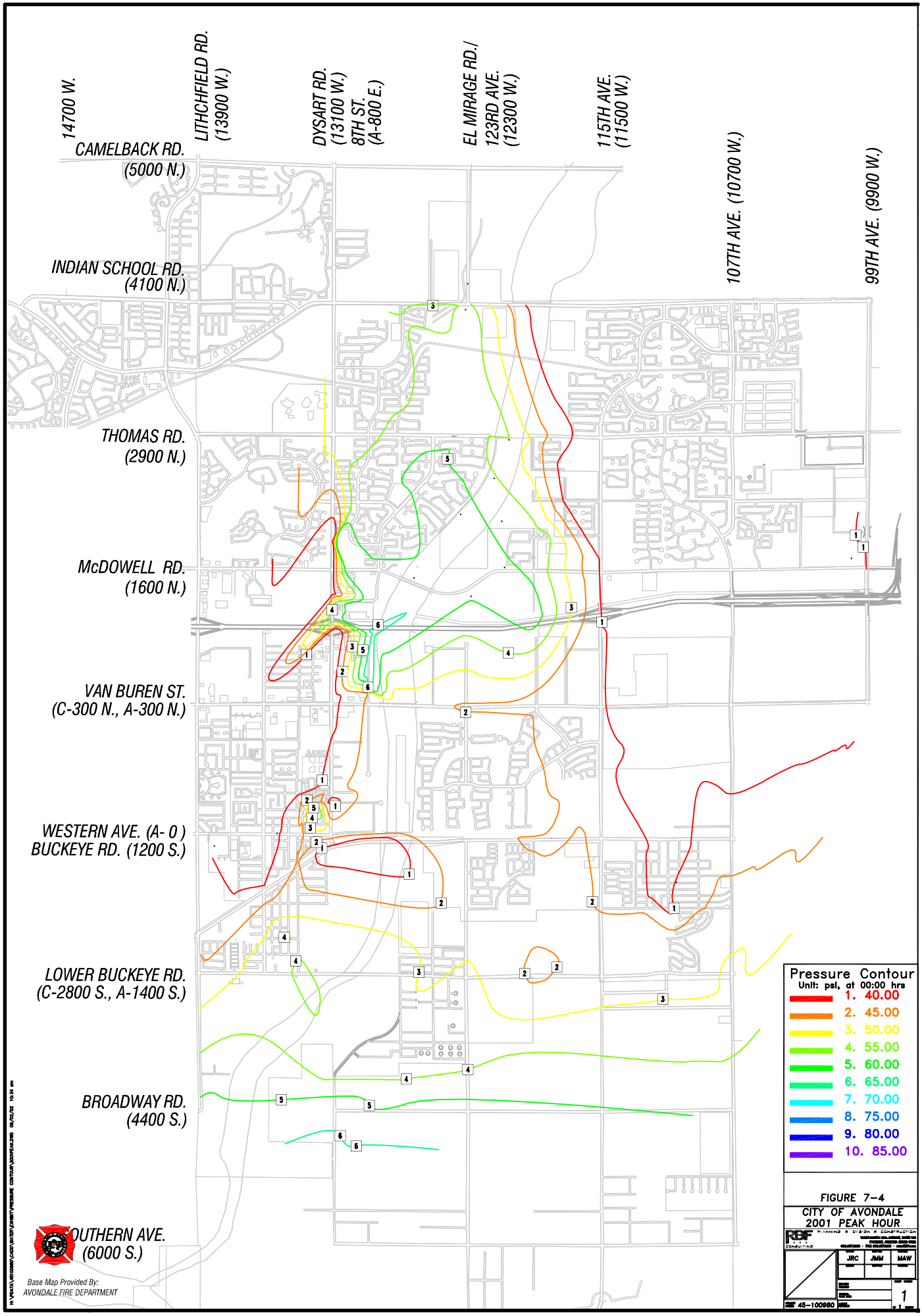

FIGURE 7–4
CITY OF AVONDALE
2001 PEAK HOUR

RBF CONSULTING

JRC	JMM	MAW

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Table 7.2 Existing System Model Results

Scenario	Average Pressure	Minimum Pressure	Minimum Node	Maximum Pressure	Maximum Node
Average Day	62.80	34.94	26	85.02	J1342
Maximum Day	59.83	34.27	26	82.16	J1342
Peak Hour	45.99	20.31	J137	75.2	149

A complete listing of nodes can be found in Appendix C-1.

In order to meet the maximum day and peak hour pressure requirements, various improvements must be made to the system. The improvements listed below are the recommended improvements to bring the City into compliance with its design standards. These improvements include the improvements required by the reliability test analysis. The improvements which are listed throughout this chapter have been designed to meet the needs of the system as well as the reliability test requirements. The quantities of improvements listed below may exceed the requirements set forth by the reliability tests during certain years. This discrepancy is due to the future need for larger improvements in a certain area. Additionally, well supply quantities have been estimated for the new storage reservoir sites. These storage quantities provide guidance only on the optimal supply for that specific location, it does not indicate that the listed quantity of supply can actually be found at that location. The reader is referred to the 2001 *Water Resources Master Plan* for guidance on areas that provide the best groundwater potential.

The following improvements need to be made to the City of Avondale water distribution system as soon as possible.

- 16-inch waterline 107th Avenue from I-10 to McDowell
- 20-inch waterline crossing the Agua Fria River in Indian School Road
- 36-inch waterline from Coldwater Springs Reservoir to 115th Avenue

7.5 2006 System Model

In order to meet the increasing demands for the 2006 system model, a series of improvements were implemented during the years 2001-2006. For modeling purposes, it is assumed that the following improvements have been completed by 2006.

- 2,500,000 gallon reservoir (Garden Lakes)
- 4,000 gpm supply (Garden Lakes)
- 4,000 gpm booster pumps at Garden Lakes site
- 2,500,000 gallon reservoir (Coldwater Springs- the construction of this reservoir is scheduled for completion in Summer 2002)
- 6,000 gpm booster pumps at Coldwater Springs (the construction of these booster pumps should be complete by Summer 2002)
- 3,000 gpm supply (Coldwater Springs)
- 1,000,000 gallon reservoir at Lower Buckeye and 119th Ave

- 2,500 gpm booster pumps at Lower Buckeye and 119th Ave
- 1,000 gpm additional supply to the system
- Replace or refurbish Well #8
- 16-inch waterline in 107th Ave. from Van Buren to Buckeye Rd
- 12-inch waterline in Coldwater Springs Blvd alignment from 115th Ave to 107th Ave.
- 12-inch waterline parallel- to Buckeye Rd (north of railway) from 119th Ave to 115th.
- 12-inch waterline in 119th Ave alignment from Cocopah to Durango St.
- 12-inch waterline Durango St alignment from 115th Ave to El Mirage.
- 16-inch waterline in 99th Ave from Well #8 to Thomas Rd.
- 16-inch waterline in Thomas Rd from 107th Ave to 99th Ave.
- 12-inch waterline in Encanto from 103rd Ave to 99th Ave.
- 12-inch waterline loop in Old Town Avondale.
- 16-inch waterline in Lower Buckeye across the Agua Fria River.
- Old Town Avondale and Cashion waterline replacement and upgrade program.

The 2006 system model was set according to the demands calculated for that year. A map of the 2006 system can be seen in Figure 7.5, 2006 System Improvements. The model was run to evaluate the average day, maximum day, and peak hour scenarios. The results from this analysis can be seen in Table 7.3, 2006 System Model Results. The average day, maximum day and peak hour pressure contours for this analysis can be seen in Figure 7.6 - Figure 7.8. A complete listing of results for the 2006 system is provided in the Appendix C-2.

Table 7.3 2006 System Model Results

Scenario	Average Pressure	Minimum Pressure	Minimum Node	Maximum Pressure	Maximum Node
Average Day	64.45	44.89	J811	86.40	J1342
Maximum Day	63.28	47.39	166	85.03	J1342
Peak Hour	59.48	41.16	J811	81.59	J1342

7.6 2011 System Model

In order to meet the increasing demands for the 2011 system model, a series of improvements were implemented during the years 2006-2011. For modeling purposes, it is assumed that the following improvements have been completed by 2011.

- 2,500,000 gallon reservoir at 115th Avenue and Miami
- 2,500,000 gallon reservoir at 107th Avenue and Van Buren
- 2,000 gpm supply at Lower Buckeye and 119th
- 3,000 gpm supply at 115th Avenue and Miami
- 3,000 gpm supply at 107th Avenue and Van Buren

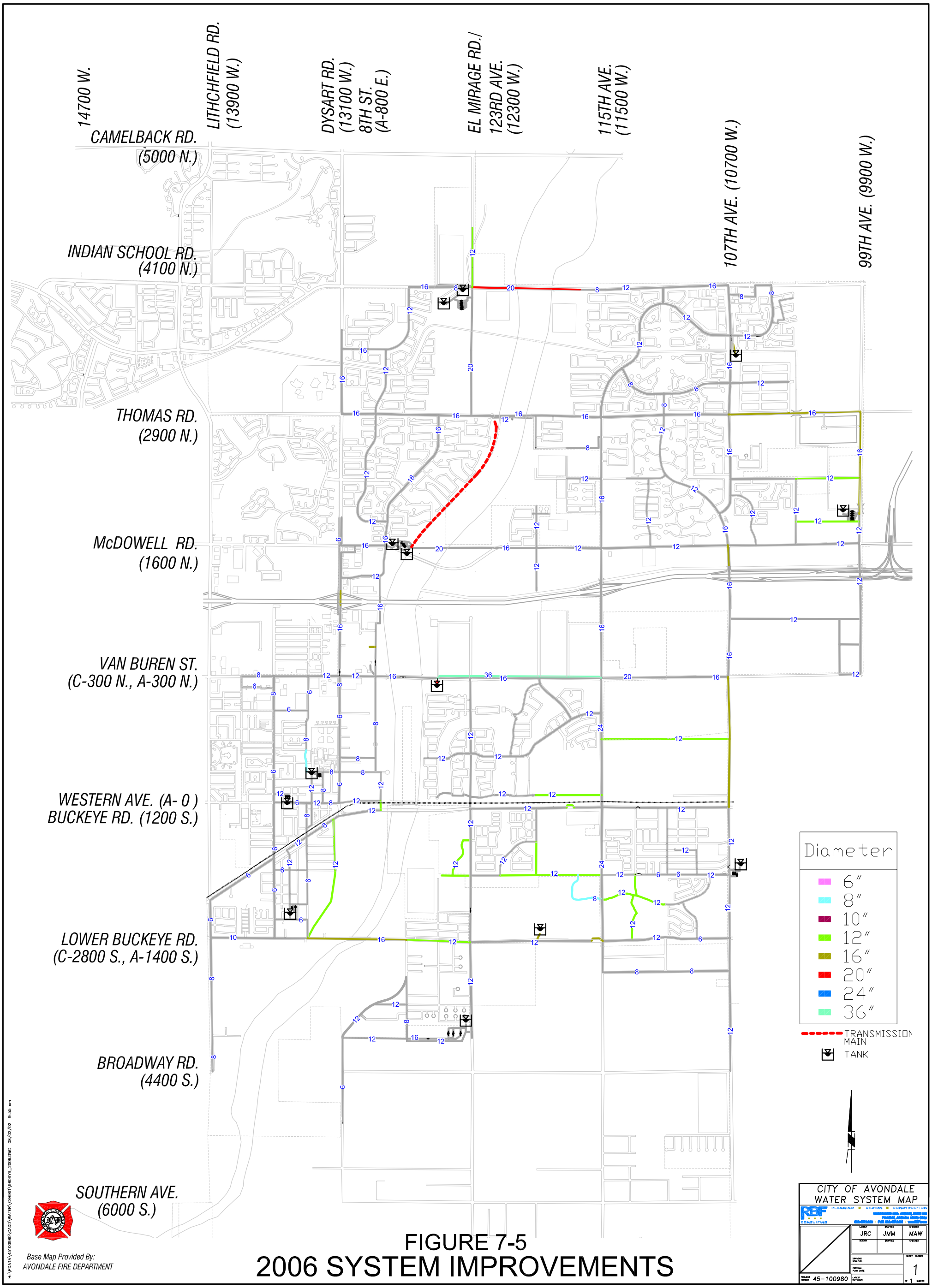

FIGURE 7-5
2006 SYSTEM IMPROVEMENTS

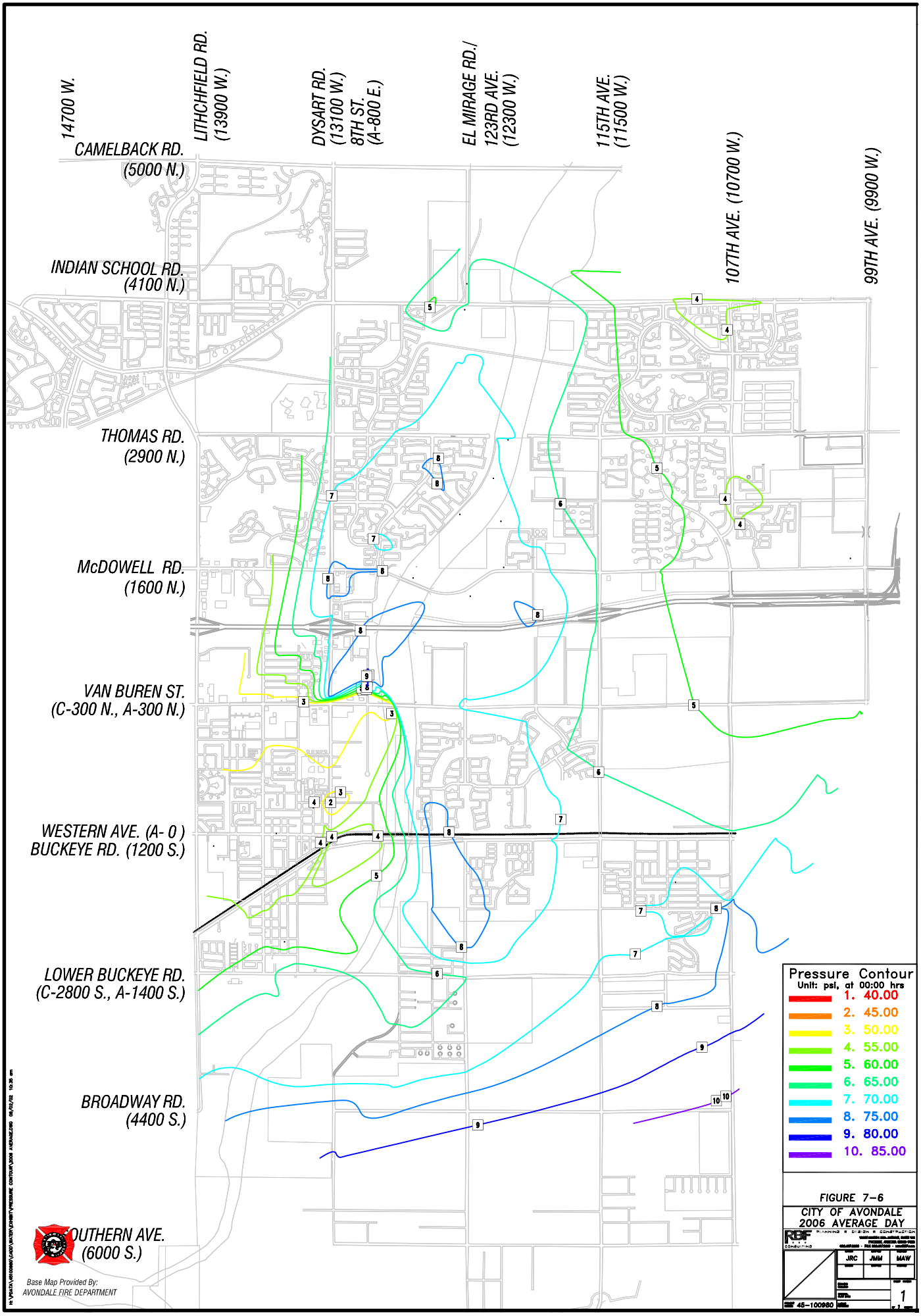

FIGURE 7–6
CITY OF AVONDALE
2006 AVERAGE DAY

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JRC	JMM	MAW

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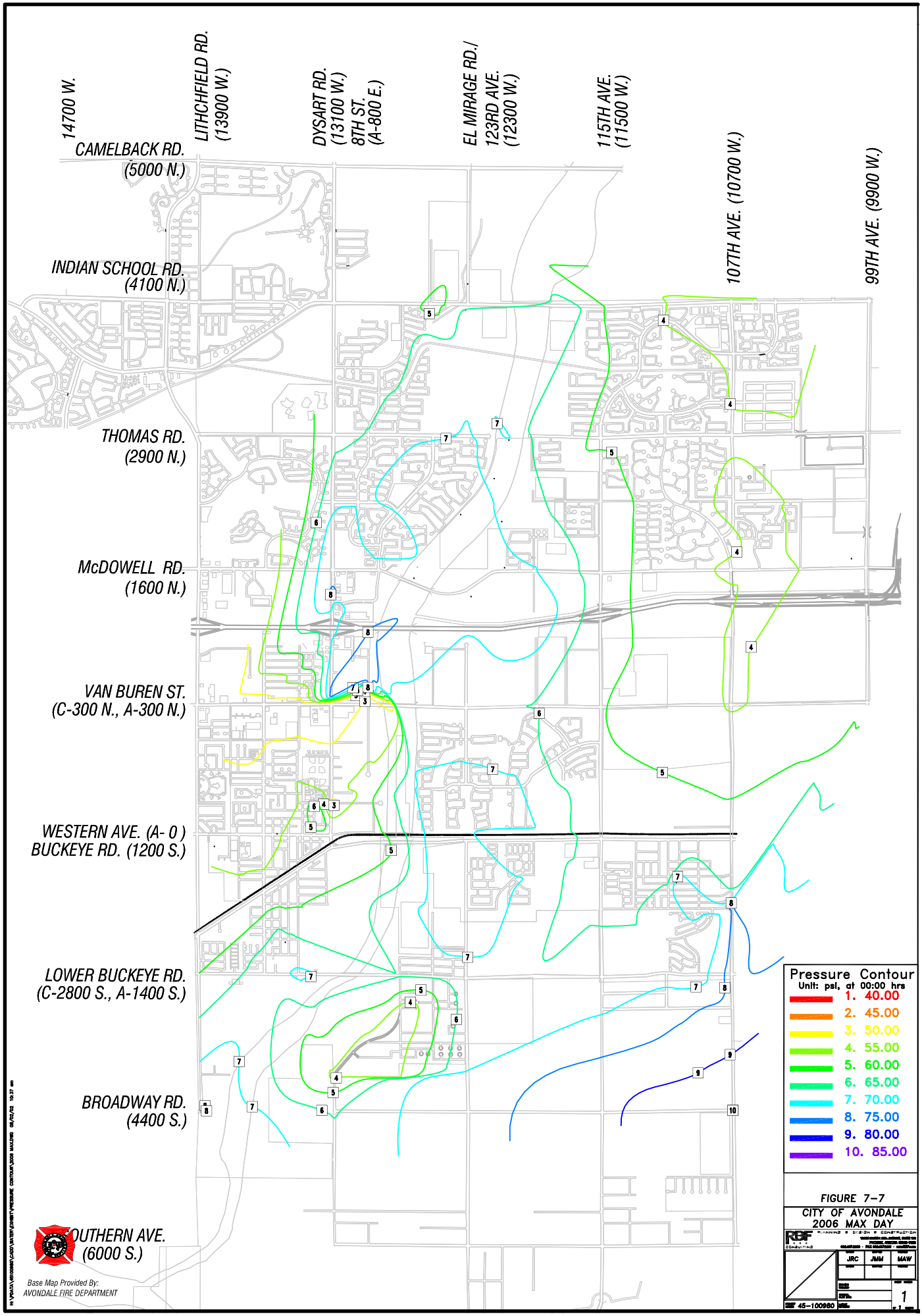

FIGURE 7–7
CITY OF AVONDALE
2006 MAX DAY

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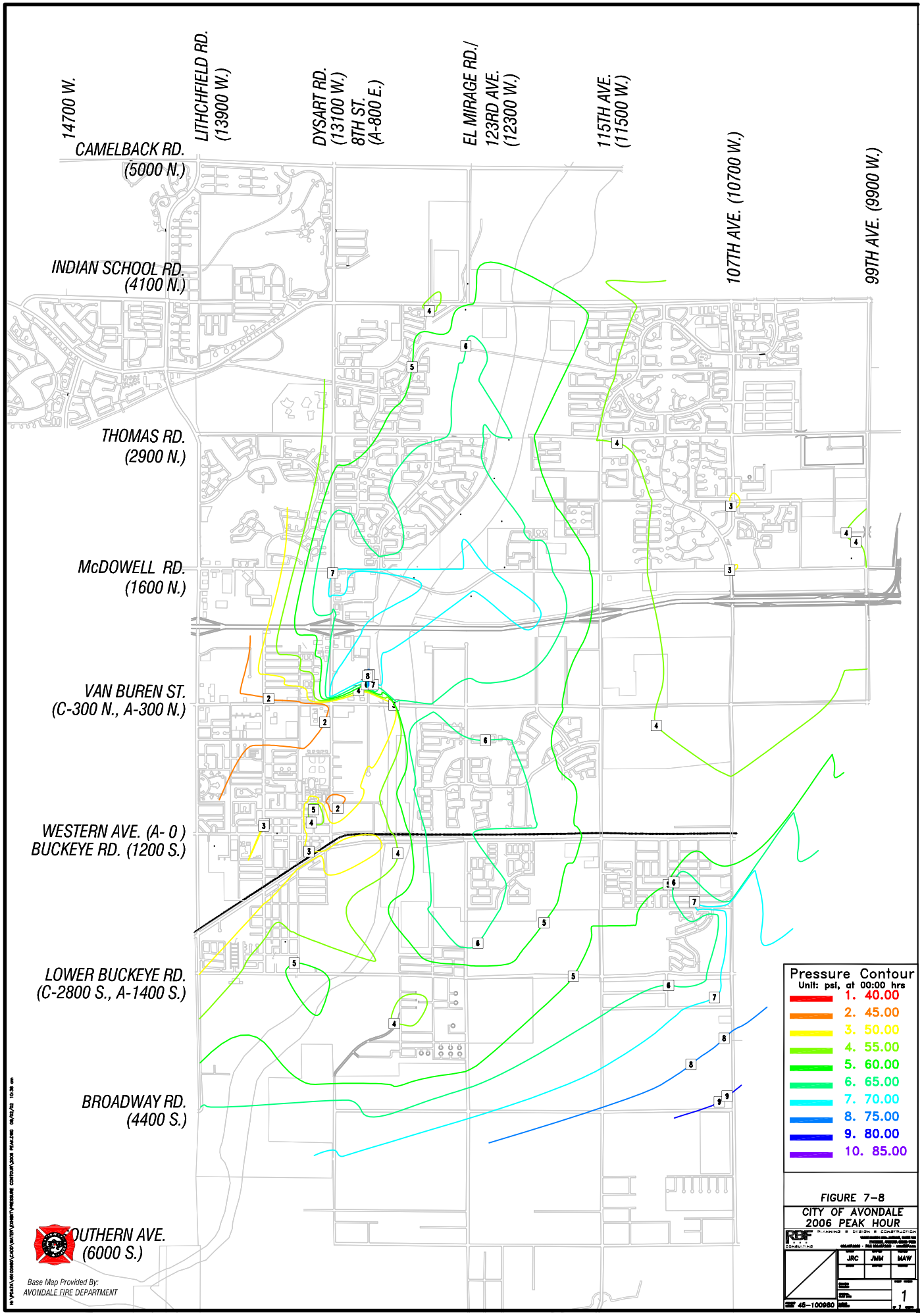

FIGURE 7–8
CITY OF AVONDALE
2006 PEAK HOUR

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- 3,000 gpm booster pumps at 115th Avenue and Miami
- 4,000 gpm booster pumps at 107th Avenue and Van Buren
- 12-inch waterline 111th Avenue alignment from Miami to Elwood
- 12-inch waterline Illini St alignment from 115th Ave to 107th Ave
- 12-inch waterline in 127th Avenue from Lower Buckeye to Pioneer St
- 16-inch waterline in 99th Avenue from Thomas to Indian School
- 16-inch waterline in Indian School from 99th to Westwind Parkway
- 24-inch waterline in 115th Avenue from Southern to Lower Buckeye
- 16-inch waterline in 115th Avenue from Southern to PIR.
- Old Town Avondale and Cashion Waterline replacement and upgrade program.

The 2011 system model was setup according to the demands calculated for that year. The 2011 system can be seen in Figure 7.9, 2011 System Improvements. The model was run to evaluate the average day, maximum day, and peak hour scenarios. The results from this analysis can be seen in Table 7.4, 2011 System Model Results. The average day, maximum day, and peak hour pressure contour maps for this system are provided in Figure 7.10 – Figure 7.12 respectively. A complete listing of results is provided in Appendix C-3.

Table 7.4 2011 System Model Results

Scenario	Average Pressure	Minimum Pressure	Minimum Node	Maximum Pressure	Maximum Node
Average Day	69.91	56.98	J811	92.36	248
Maximum Day	61.33	46.54	J811	80.98	248
Peak Hour	57.00	42.36	J811	77.04	J138

7.7 2026 System Model

In order to meet the increasing demands for the 2026 system model, a series of improvements were implemented during the years 2011-2026. For modeling purposes, it is assumed that the following improvements have been completed by 2026.

- 1,500,000 gallon reservoir at Thomas and 115th Ave.
- 2,500,000 gallon reservoir at Buckeye and 115th Ave.
- 2,500,000 gallon reservoir at Broadway and 115th Ave.
- 2,500,000 gallon reservoir at Coldwater Springs
- 2,000 gpm supply at Thomas and 115th Ave.
- 7,000 gpm supply at Buckeye and 115th Ave.
- 7,000 gpm supply at Broadway and 115th Ave.
- 6,000 gpm supply at Coldwater Springs
- 2,000 gpm supply additional in system

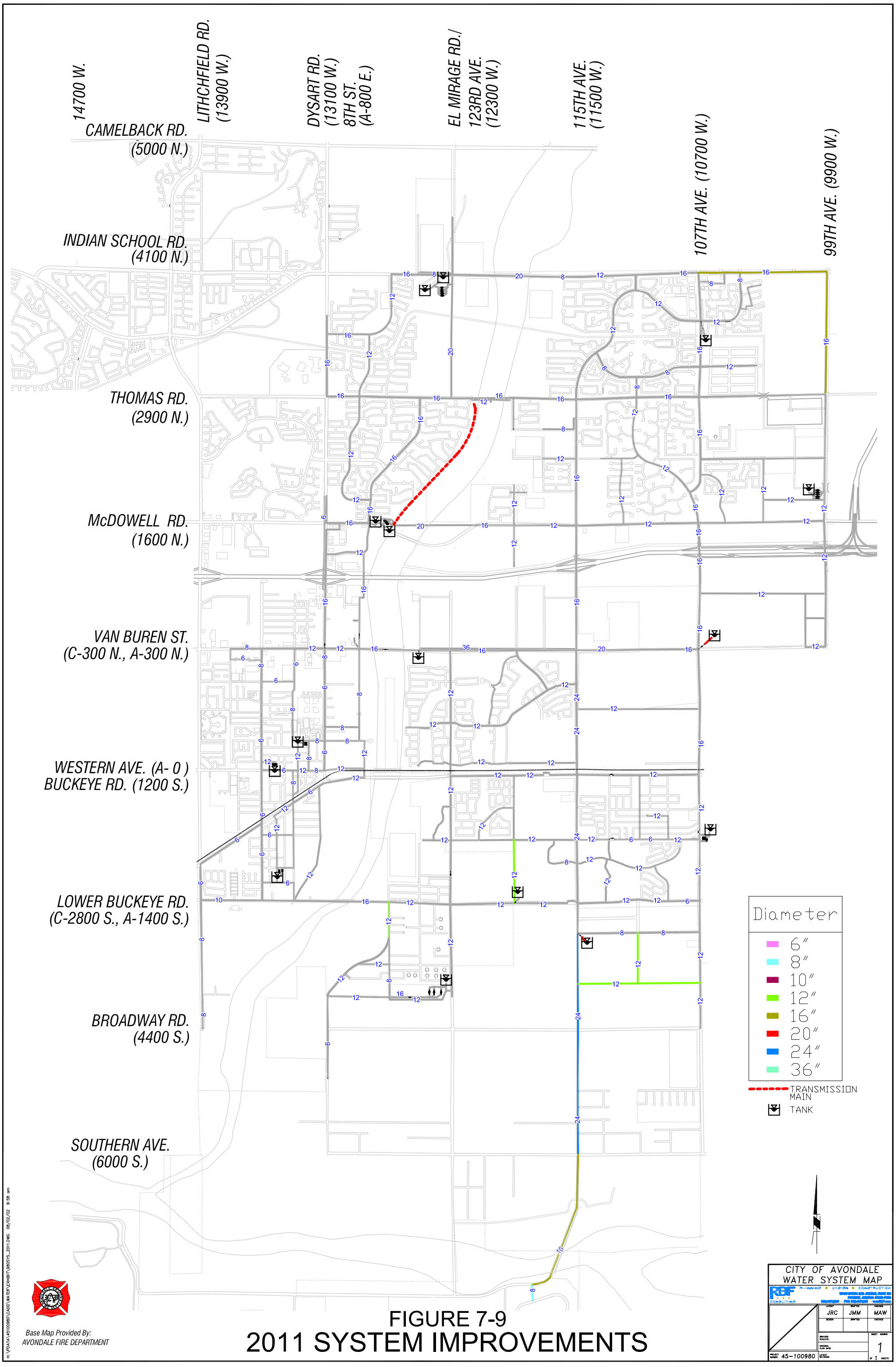

FIGURE 7-9
2011 SYSTEM IMPROVEMENTS

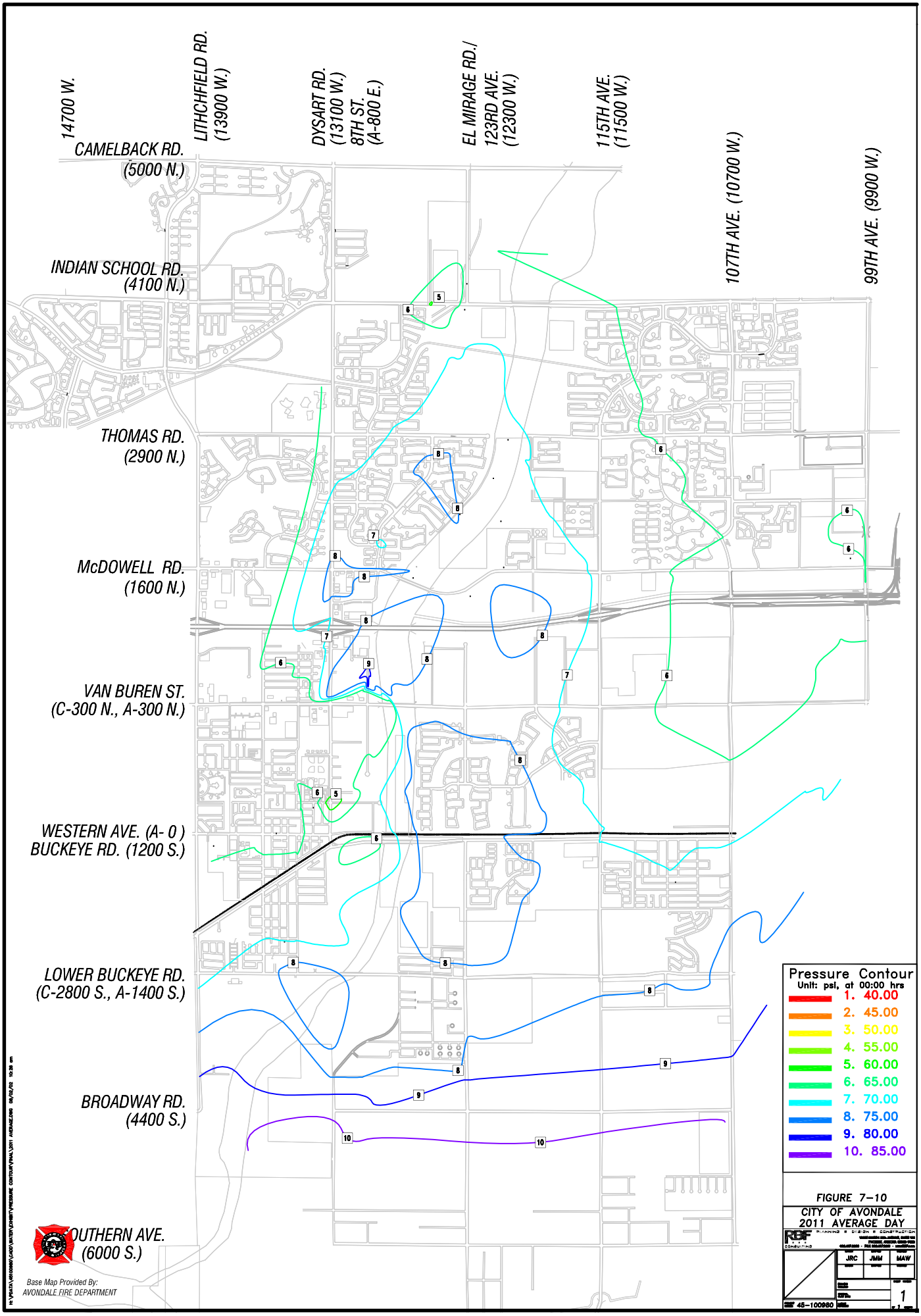

FIGURE 7–10

CITY OF AVONDALE
2011 AVERAGE DAY

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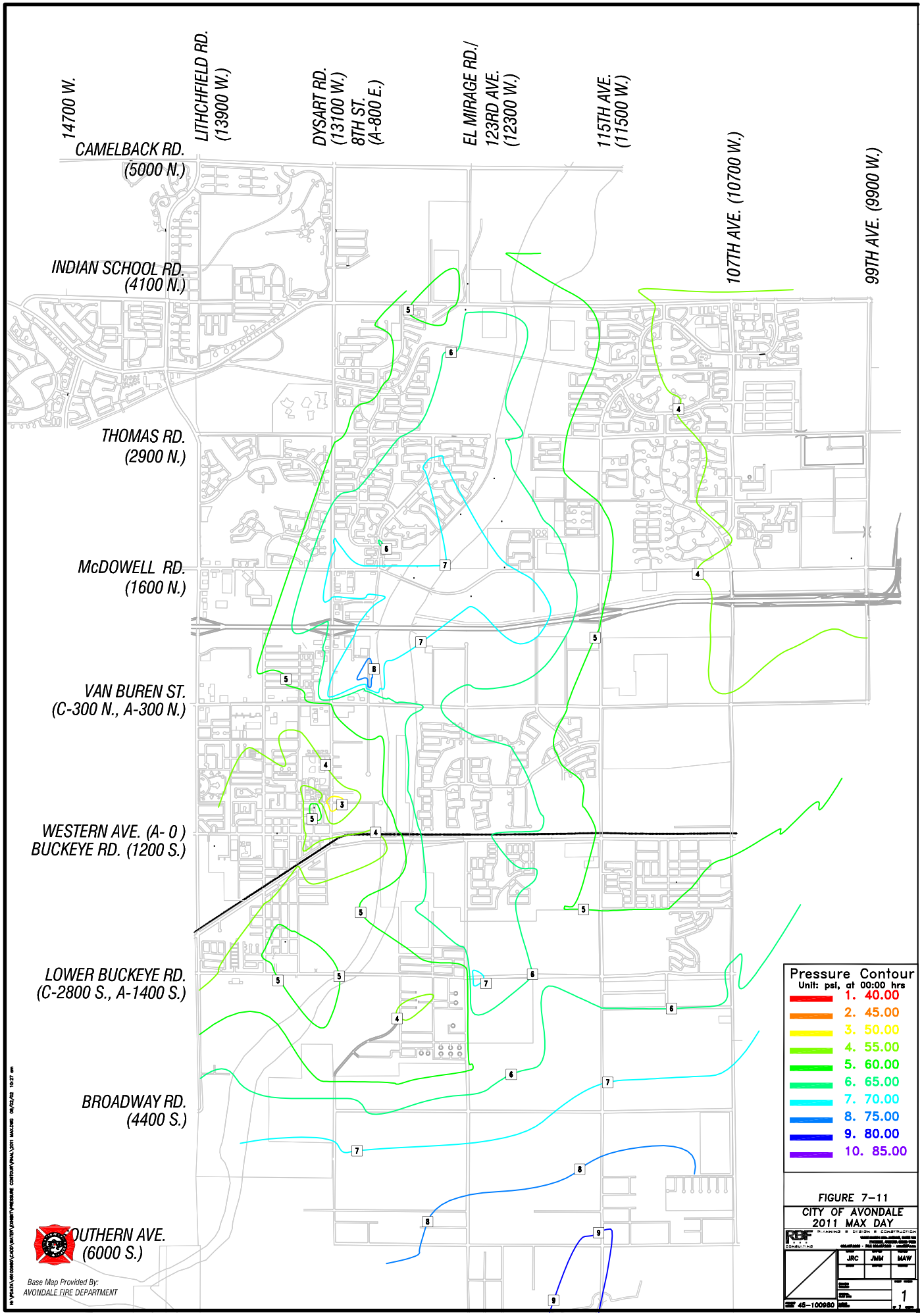

FIGURE 7–11
CITY OF AVONDALE
2011 MAX DAY

RBF CONSULTING

JRC	JMM	MAW

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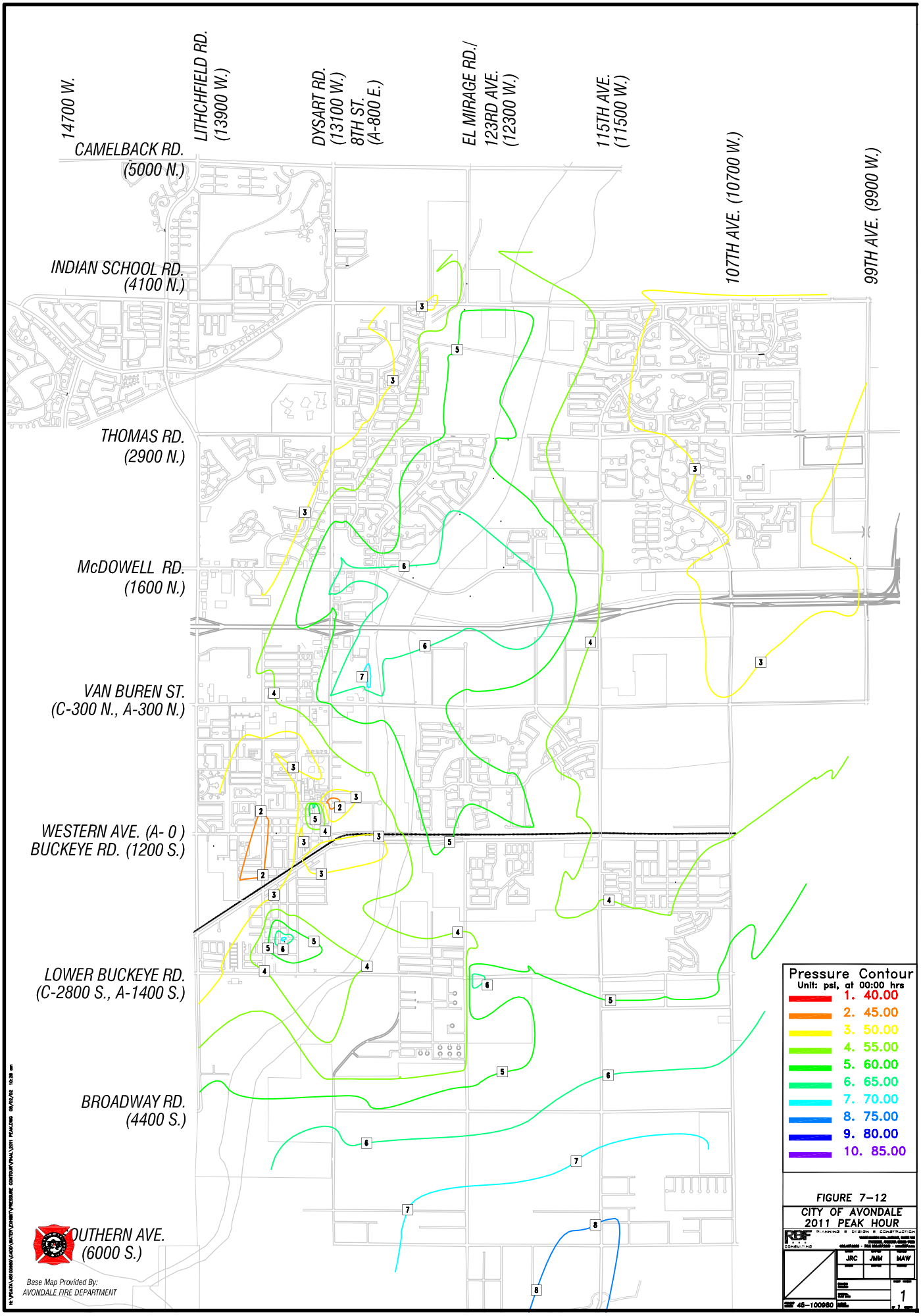


FIGURE 7-12
CITY OF AVONDALE
2011 PEAK HOUR

- 4,500 gpm booster pumps at Thomas and 115th Ave.
- 9,000 gpm booster pumps at Buckeye and 115th Ave.
- 9,000 gpm booster pumps at Broadway and 115th Ave.
- 8,000 gpm booster pumps at Coldwater Springs
- 16-inch waterline in Broadway from Dysart to 107th Ave.
- 16-inch waterline in Southern from Dysart to 107th Ave.
- 16-inch waterline in 107th Ave from Southern to Broadway
- 16-inch waterline in Dysart from Southern to Broadway
- 16-inch waterline in Indian School from 107th to 99th
- 16-inch waterline in 99th Ave from Indian School to Thomas
- 16-inch waterline in Thomas from 107th to 99th
- 16-inch waterline in Van Buren from 107th Ave. to 100th Ave.
- 16-inch waterline in Indian School from Rancho Santa Fe Trail to Dysart
- 12-inch waterline in Encanto from 115th Ave to 112th Dr.
- 16-inch waterline in Buckeye Road across the Agua Fria River
- 8-inch waterline in 4th Street from Riley to St Lawrence
- 12-inch waterline in Broadway alignment from Dysart to Litchfield Road

The 2026 system model was setup according to the demands calculated for that year. A map of the 2026 system is shown in Figure 7.13, 2026 System Improvements. The model was run to evaluate the average day, maximum day, and peak hour scenarios. The results from this analysis can be seen in Table 7.5, 2026 System Model Results. A complete listing of results for the 2026 system is provided in Appendix C-4.

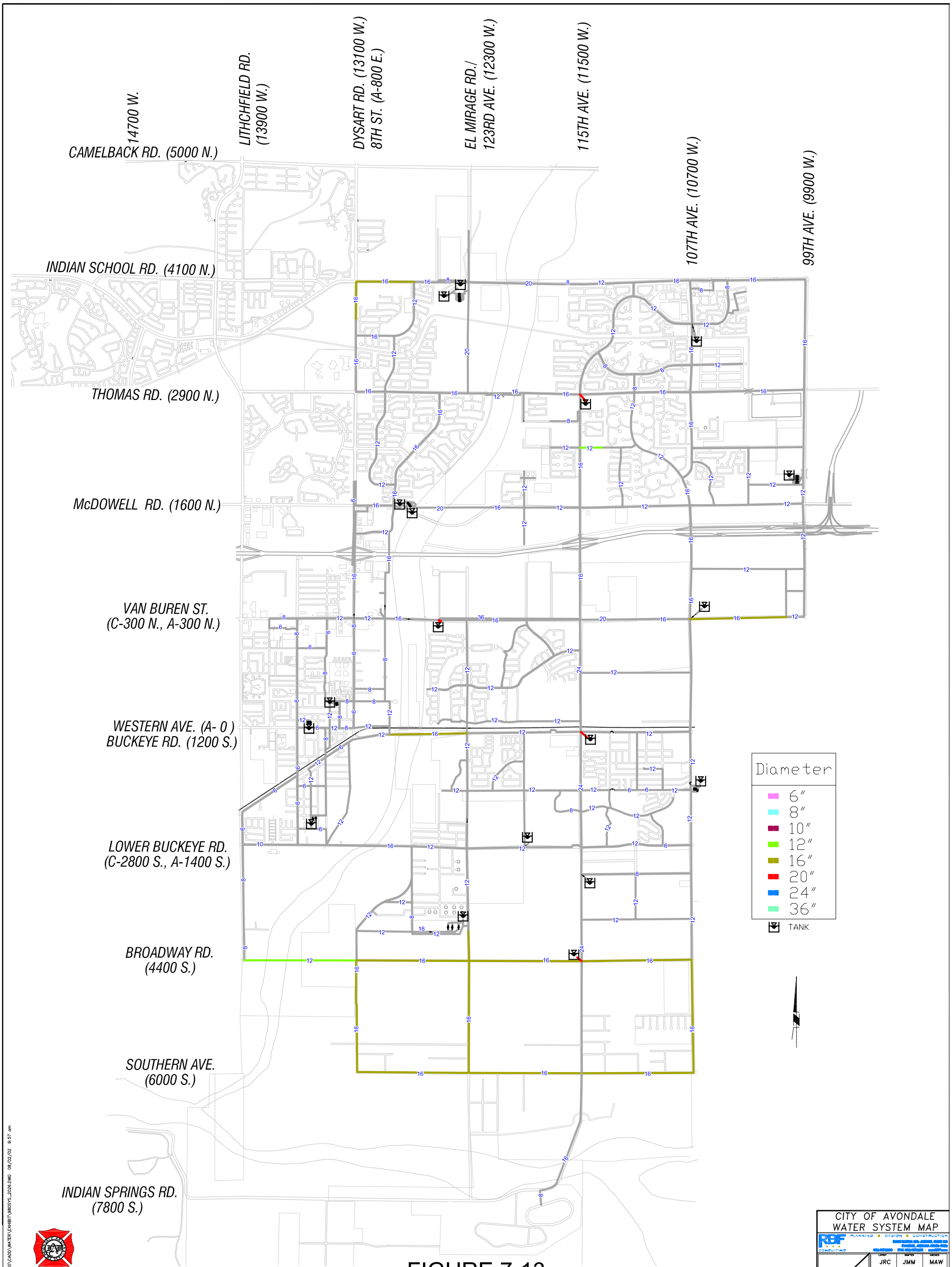
Table 7.5 2026 System Model Results

Scenario	Average Pressure	Minimum Pressure	Minimum Node	Maximum Pressure	Maximum Node
Average Day	64.31	52.96	J-83	82.92	252
Maximum Day	60.58	50.48	J-83	77.35	J138
Peak Hour	54.28	41.01	J-116	74.36	J138

7.8 2040 System Model

In order to meet the increasing demands for the 2040 system model, a series of improvements were implemented during the years 2026-2040. For modeling purposes, it is assumed that the following improvements have been completed by 2040.

- 1,000,000 gallon storage reservoir at Southern and 115th Ave
- 4,000 gpm booster pumps at Southern and 115th Ave
- 3,000 gpm supply at Southern and 115th Ave
- 4,100 gpm supply additional throughout the system
- 16-inch waterline in 99th Ave from McDowell to Thomas
- 12-inch waterline in 112th Ave from Virginia to Palm



Base Map Provided By:
AVONDALE FIRE DEPARTMENT

FIGURE 7-13
2026 SYSTEM IMPROVEMENTS

CITY OF AVONDALE
WATER SYSTEM MAP

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LAYOUT	DRAFTED	CHECKED
JRC	JMM	MAW
DESIGN	DRAFTED	CHECKED

DRAWING SCALE(S)
ORIGINAL PLAN DATE
LATEST REVISION
PROJECT NUMBER 45-100980
SHEET NUMBER 1 OF 1 SHEETS

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- 12-inch waterline in Encanto from 119th Ave to 116th Lane
- 16-inch waterline in El Mirage from Indian Springs to Southern
- 16-inch waterline in Indian Springs from El Mirage to PIR

The 2040 system model was set up according to the demands calculated for that year. A map of the 2040 system is shown in Figure 7.14, 2040 System Improvements. The model was run to evaluate the average day, maximum day, and peak hour scenarios. The results from this analysis can be seen in Table 7.6, 2040 System Model Results. A complete listing of results for the 2040 system is provided in Appendix C-5.

Table 7.6 2040 System Model Results

Scenario	Average Pressure	Minimum Pressure	Minimum Node	Maximum Pressure	Maximum Node
Average Day	63.74	51.99	J-83	82.20	252
Maximum Day	59.89	50.04	J-83	77.32	J138
Peak Hour	54.03	41.42	J116	74.28	J1010

7.9 Fire Flow Analysis

One of the main functions of a water distribution system is to provide the users of that system with adequate fire flow protection. Adequate fire protection has been defined as providing the minimum required fire flow at a minimum residual pressure of 20 psi. The minimum required fire flow is provided using the 2000 International Fire Code, and is described in more detail in Section 4.3.2, Recommended Fire Flow Protection Requirements.

In this fire flow analysis, the maximum fire flow possible was computed for each node in the system, excluding those nodes at the inlet side of a pump. The fire flow analysis was performed for the maximum day system for each of the design periods. The results for the fire flow analysis can be seen in Appendices C-1 through C-5.

Based on this analysis, for the current system the area with the lowest fire flow pressure is the Old Town Avondale area. This is due to the fact that this area typically has the smallest pipes, as well as high headlosses in the pipes due to their increased age. One of the recommendations set forth in this report is that a waterline upgrade and replacement program be implemented in this area to help increase the fire flow capacity. For the future modeling scenarios, the fire flow in this area improves, as the waterlines are upgraded.

There are two other locations which show fire flow problems, these are at nodes J1300 and 252. Both of these nodes are on dead end small diameter pipes at the southern end of the city. Fire flow to these nodes becomes adequate when the system is looped in these areas.

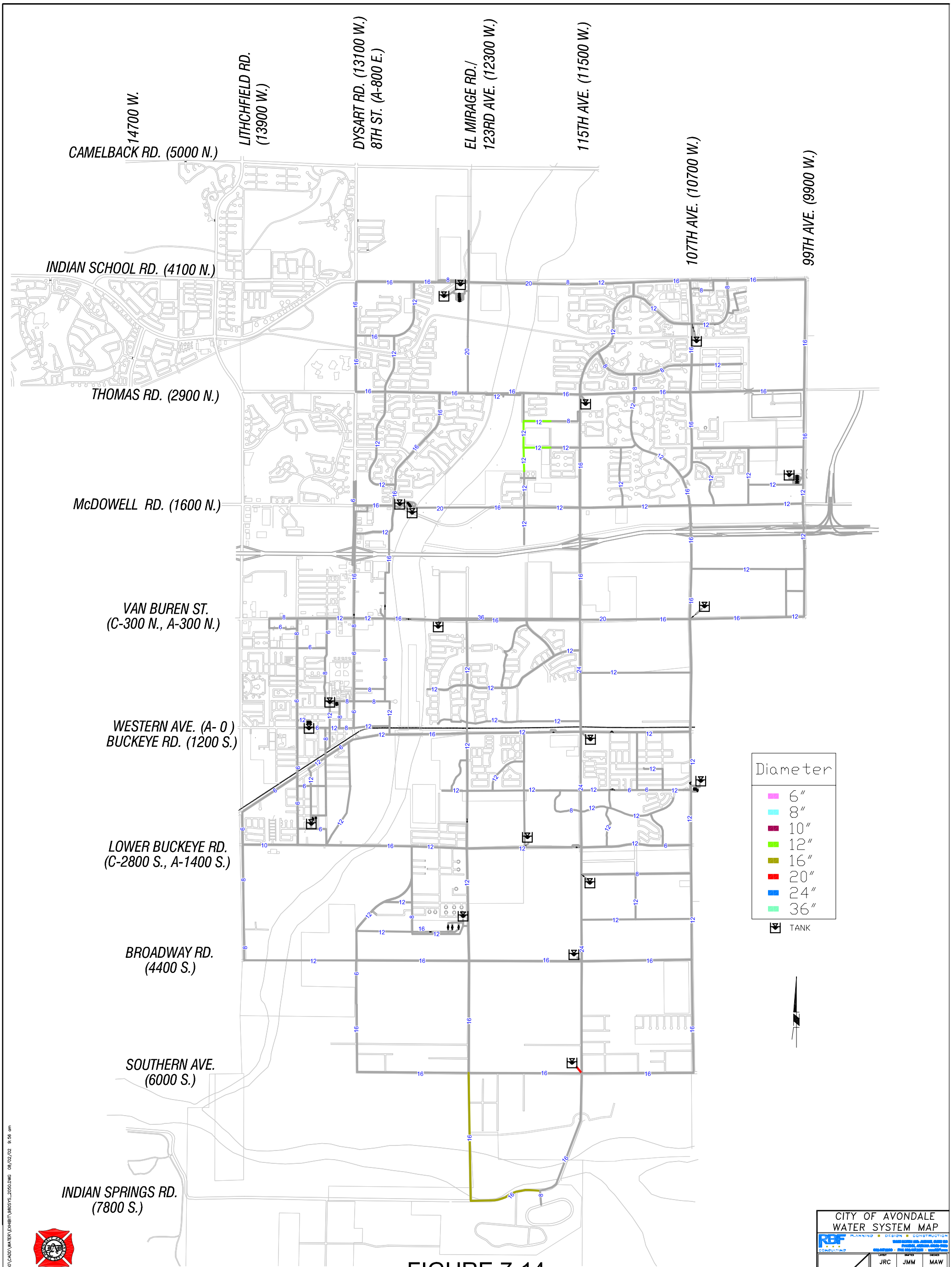


FIGURE 7-14
2040 SYSTEM IMPROVEMENTS

8.0 Recommendations

8.1 Recommended SCADA System

8.1.1 Path Recommendations

The recommended radio communication paths are the result of individual path profile studies and the microwave budget link analysis. Other considerations leading to this recommendation include a cost effective implementation and the flexibility required for future expansion. The results of this study indicate that the recommended paths utilizing the recommended antenna heights should provide reliable radio communication. A complete radio signal strength test should be conducted for each path prior to design.

The initial path concept was that the Master Radio at Coldwater Springs Booster Pump Station could communicate directly with Reservoirs 1, 2, 5, Cashion, and Bohne. Path studies from the Coldwater Springs site to these remote sites revealed several terrain interferences preventing a line of sight path.

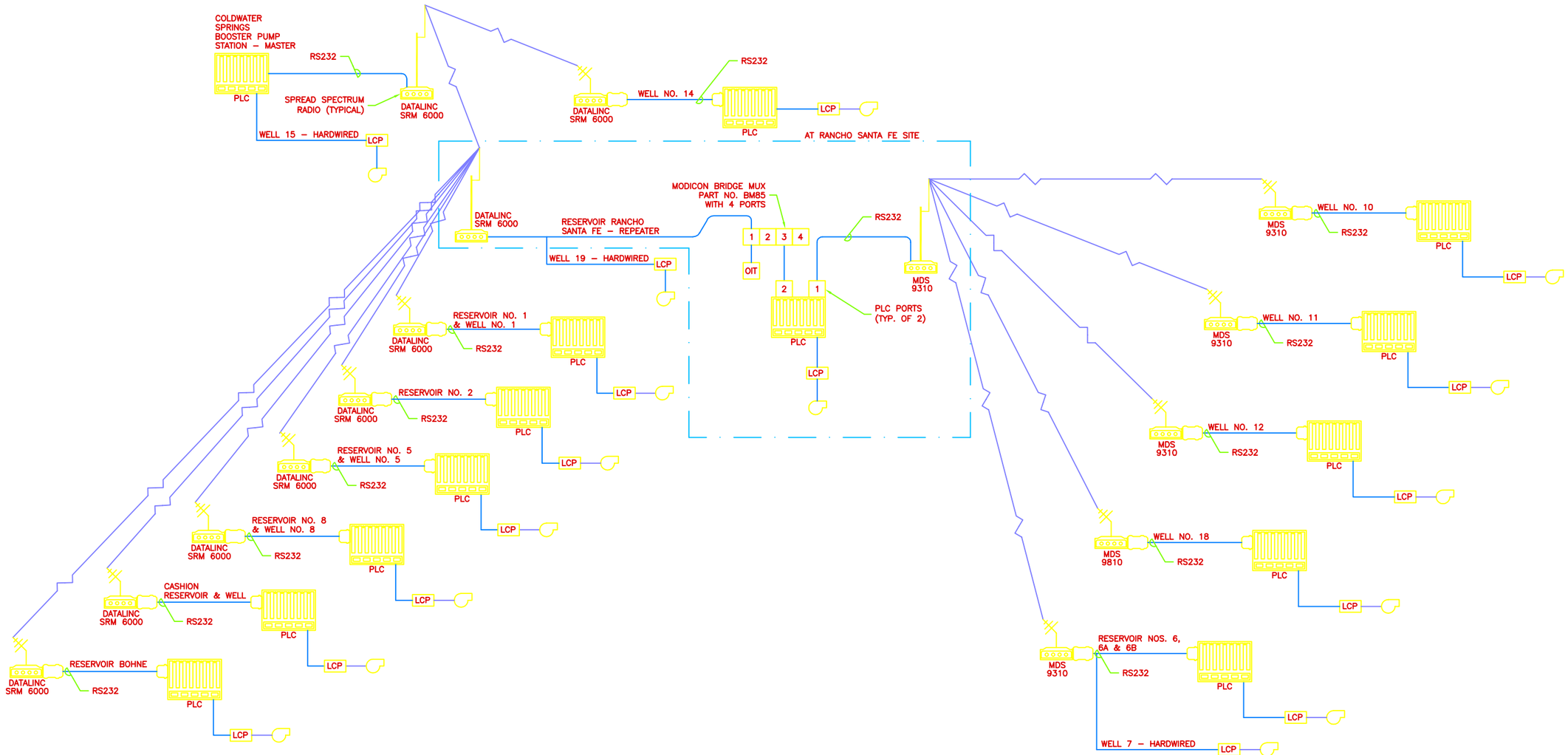
The base elevation at the Rancho Santa Fe site is approximately 30 feet higher than the elevation at Coldwater Springs. By utilizing Rancho Santa Fe as a repeater, a direct line of site path can be established from Coldwater Springs to Rancho Santa Fe and then to Reservoirs 1, 2, 5, Cashion, and Bohne.

Figure 8.1, Recommended Radio Communication Path, illustrates the recommended radio communication paths and major hardware items required in achieving these paths. From a programming perspective, several hardware items will become transparent as illustrated in the simplified Data Path for SCADA Communications, see Appendix E.

8.1.2 Proposed System Overview

A PC workstation running Human-Machine Interface (HMI) software and alarm dialer software will poll or communicate with each remote site to monitor status, alarms, and collect data. If alarm conditions occur, the PLCs report the alarm condition to the HMI during the next poll cycle, and update all other data in the HMI database. Automatic control of wells and booster stations shall be performed locally by the PLCs, or remotely by the operator using the HMI at the master station. PLC ladder logic programs will start and stop pumps based on tank level setpoints, and pressures at each site. Booster pumps on Variable Frequency Drives (VFDs) will be adjusted automatically by the PLC logic to maintain a desired pressure setpoint at each reservoir booster station. Where the booster pumps are not on VFD, the constant speed pumps will be staged on starting with the lowest horsepower required to maintain the pressure setpoints. Additionally, the system should monitor all PLC Input/Output (I/O) points and alarm conditions such as pump operation, tank level, system pressures and

FIGURE 8.1 – RECOMMENDED RADIO COMMUNICATION PATH



LEGEND:

INDICATES RADIO COMMUNICATION PATH

OMNI DIRECTIONAL ANTENNA

YAGI DIRECTIONAL ANTENNA

PUMP(S) WELL OR BOOSTER

ABREVIATIONS:

LCP LOCAL CONTROL PANEL
OIT OPERATOR INTERFACE TERMINAL
PLC PROGRAMMABLE LOGIC CONTROLLER

DATE: 02/21/02 2:46 PM D639_SK-1.DWG KJB

Rev'd By Date

DL engineering & controls, inc.
4620 North 16th St., #C115, PHOENIX, AZ 85016

CITY OF AVONDALE
MASTER TELEMERY
RECOMMENDED RADIO
COMMUNICATION PATH

Scale	NA
Job No.	D639
CADD File No.	CAD FILE
Designed	CG
Drawn	RCM
Checked	BDD
Date	JAN 2002
Sheet	•

flows, power status, generator alarms, and intrusion alarms. The MTU computer collects the remote site data by polling the well and reservoir sites sequentially. An operator can issue start and stop commands, override the PLC logic, and change setpoints remotely from the MTU workstations at Coldwater Springs.

Alarms are generated at the PLCs and communicated to the MTU. The alarm handler software prioritizes the alarms; the most critical alarms are sent to the alarm dialer. The alarm dialer automatically dials on-call maintenance personnel who acknowledge the alarm using the telephone keypad.

Alarms and events may be directed to any printer on the network, but are generally sent to dedicated printers in a control room. Color screen prints may be sent to any color printer, but are generally directed to the nearest location. Finally, application software documentation or very long reports can be directed to a printer.

Antenna Feed Cable, Connectors, and Protective Devices
Antenna feed cables for the radios of less than 50-feet in length should be Andrew Heliax LDF2-50 with Andrew Type N connectors and Andrew standard grounding kit. The cable assembly should include a factory prepared 3-foot section of superflexible FSJ2-50 jumper cable with Andrew Type N connectors, and a Polyphaser Series IS-50 lightning protector. Antenna feed cables of greater than 50-feet in length for the radios should be Andrew Heliax LDF4-50A, with Andrew Type N connectors and Andrew SureGround grounding kit. Grounding should be provided at the antenna mast and RTU enclosures.

Antenna

The unlicensed 902-928 MHz band, spread spectrum radios at the remote site should utilize broadband Yagi-type Scala Model TY900 antennas with 10 dBm gain. The master station and repeater sites should utilize omni-directional Scala Model OGB6-900 antennas with 6 dBm gain, or Maxrad MFB series antennas. Antenna masts should be manufactured by CEM-TEC Corporation, or equal.

8.1.3 Equipment Recommendations

The MDS 9810 and MDS 9310 radios currently in use have the capability of using a single radio as a repeater within a transmission route. This means that only one radio can be used as a repeater between two endpoints of a transmission route. For example: the MDS radio at Coldwater springs can communicate to Well 10 using Rancho Santa Fe as a repeater. Well 10 would be the end of the transmission route and could not be used as a repeater to a new site. The currently used MDS radios greatly limit the flexibility that may be needed for future expansion.

The Data-Linc SRM 6000 radios being recommended allow three radio repeaters to be used between the endpoints of a transmission route. This system would

allow the signal from Coldwater Springs to be repeated at Rancho Santa Fe to the Bohne Reservoir. The Bohne Reservoir could then act as a repeater sending a signal to a new site such as the one at the Phoenix International Raceway. From the Phoenix International Raceway the signal could be repeated to one additional site. Additionally, the Data-Linc radios will allow a greater flexibility for future expansion. Table 8.1 provides a list of site specific recommendations.

Table 8.1 Site Specific Equipment Recommendations

Site	Recommendation
Coldwater Springs Booster Pump Station	Install a Data-Linc SRM 6000 radio in place of the MDS 9820.
Reservoir 1 & Well 1	Install a Remote Telemetry Unit with a Data-Linc SRM 6000 radio and a Modicon 984-E241. This site will require an antenna height of 40 feet.
Reservoir 2 (Hill St. Booster Station)	Install a Remote Telemetry Unit with a Data-Linc SRM 6000 radio and a Modicon 984-E241. This site will require an antenna height of 35 feet.
Reservoir 5 & Well 5	Install a Remote Telemetry Unit with a Data-Linc SRM 6000 radio and a Modicon 984-E241. This site will require an antenna height of 30 feet.
Reservoir 8 & Well 8	Replace the existing MDS 9310 radio with a Data-Linc SRM 6000 radio. Antenna should be extended from 20 feet to 30 feet.
Cashion Reservoir	Modify the design of the Remote Telemetry Unit to include a Data-Linc SRM 6000 radio. This site will require an antenna height of 30 feet.
Reservoir Bohne	Replace the existing MDS 9310 radio with a Data-Linc SRM 6000 radio. Antenna should be extended from 20 feet to 35 feet.
Reservoir Rancho Santa Fe	Add an additional Data-Linc SRM 6000 radio and Modicon Bridgemux part number B85 to the existing telemetry unit. This site will require an additional Omni antenna installed 24 feet above the existing 16-foot tank for an effective height of 40 feet.
Well 10	No changes recommended.
Well 11	No changes recommended.
Well 12	Although this path is currently in operation, it is recommended that the signal strength be verified. If necessary increase the antenna height.
Well 18	No changes recommended
Reservoir 6	The landfill, approximately 0.2 miles south of Reservoir 6 may create line of sight problem if the wall height continues to increase. If this problem occurs, the antenna height should be increased to maintain line of sight.
Well 14	Modify the design of the Remote Telemetry Unit by replacing the MDS 9310 radio with a Data-Linc SRM 6000 radio.

The optimal system recommendation would include replacement of all MDS radios with Data-Linc SRM 6000 radios. In consideration of the cost associated with implementing the optimal plan, the MDS radios at Reservoir 6, Well 10, Well 11, and Well 12 need not be replaced at this time. However, it is recommended that as these MDS radios fail, they be replaced with Data-Linc SRM 6000 radios.

It is also recommended that a Radio Path Survey such as this one be included as part of the design services work in the process of locating future reservoir and well sites. By performing this study in the early phases of a project, the viability of a radio communication path can be determined prior to incurring development costs. Additionally, specifications should require antenna cables to be factory assembled prior to installation in the field. The specifications should also require the field system to be tested by a radio system contractor.

In order to plan for the costs of implementing the SCADA system within the City of Avondale, costs have been estimated for each of the sites in the City. The cost estimates in Table 8.2, Opinion of Probable Cost Table, provide a generalized guide as to what the City could expect to pay to implement a full scale SCADA system. The Opinion of Probable Cost covers the cost of SCADA related hardware for all sites in this study and the future sites of Garden Lakes Reservoir, Well 16, Well 17, and Phoenix International Raceway. The SCADA hardware includes antennas and related cabling, Data-Linc radios, 984 Compact PLCs, UPS, RTU enclosures, power supplies, flowmeters, pressure and level transducers, and antenna masts. Also included in the Opinion of Probable Cost is the Engineering, Programming and labor for installing SCADA equipment.

Not included in the Opinion of Probable Cost is the cost of equipment that is under design by others.

Table 8.2 Opinion of Probable Cost Table

Site Name	Costs		
	Hardware	**Engineering & Programming**	**Total**
Coldwater Springs	$3,000	$50,000	$53,000
Rancho Santa Fe	$9,000	$4,000	$13,000
Well 18	$0	$3,000	$3,000
Reservoirs 6, 6A, 6B	$8,000	$3,000	$11,000
Well 10	$0	$3,000	$3,000
Well 11	$0	$3,000	$3,000
Well 12	$0	$3,000	$3,000
Reservoir 1 & Well 1	$19,000	$9,000	$28,000
Reservoir 2	$17,000	$11,000	$28,000
Reservoir 5 & Well 5	$15,000	$9,000	$24,000
Reservoir 8 & Well 8	$4,000	$3,000	$7,000
Well 14	$0	$3,000	$3,000
Well 15	$0	$3,000	$3,000
Cashion Reservoir	$16,000	$5,000	$21,000
Reservoir Bohne	$8,000	$5,000	$13,000
Garden Lakes Reservoir	$14,000	$9,000	$23,000
Well 16	$14,000	$7,000	$21,000
Well 17	$14,000	$7,000	$21,000
PIR	$14,000	$9,000	$23,000
Sub Total	$155,000	$149,000	$304,000
Installation Labor			$30,000
20% Contingency			$67,000
Project Total			**$401,000**

8.2 Treatment Options

8.2.1 Regulations and Requirements

8.2.1.1 Primary Drinking Water Regulations

The Federal regulations, established by the U. S. Environmental Protection Agency (USEPA), were originally detailed in the Safe Drinking Water Act (SDWA) of 1974. The SDWA established drinking water standards for 32 contaminants. Since then, the act has been amended several times and the most recent amendment in 1996 increased the list of regulated contaminants with Maximum Contaminant Levels (MCLs) to 83 with treatment technique requirements for 9 additional contaminants. The Primary Drinking Water Maximum Containment Levels (MCLs) are presented in the Table 8.3, Primary Drinking Water Standards.

In addition to the regulations enacted into law, USEPA has established a number of drinking water rules that also require compliance.

Table 8.3 Primary Drinking Water Standards

PARAMETERS (mg/L unless otherwise noted)	EPA Primary MCL[1]
AESTHETIC PARAMETERS	
Turbidity (ntu)	0.3
MICROBIOLOGICAL CONTAMINANTS	
Giardia Lamblia	TT[2]
Legionella	TT[2]
Standard Plate Count (cfu/mL)	TT[2]
Total Coliforms	<5% Positive
Viruses	TT[2]
DISINFECTION BY-PRODUCTS	
Total Trihalomethanes (TTHM)	0.1
INORGANICS	
Antimony	0.006
Arsenic	0.05
Asbestos (million/L)	7
Barium	2
Beryllium	0.004
Cadmium	0.005
Chromium	0.1
Copper	TT[2]
Cyanide	0.2
Fluoride	4
Lead	TT[2]
Mercury	0.002
Nickel	0.1

Notes:
1.Maximum Contaminant Level
2.Indicates that a treatment technique is required in order to show compliance with regulations.

Table 8.3 Primary Drinking Water Standards (continued)

PARAMETERS (mg/L unless otherwise noted)	EPA Primary MCL[1]
Nitrite + Nitrate (as N)	10
Nitrite (as N)	1
Selenium	0.05
Thallium	0.002
RADIONUCLIDES	
Beta Particle and Photon Activity (millirem/yr)	4
Combined Radium 226 & 228 (pCi/L)	5
Gross Alpha (pCi/L)	15
ORGANICS	
Volatile Organics	
Benzene	0.005
Carbon Tetrachloride	0.005
Dichloroethane (1,2-)	0.005
Dichloroethylene (1,1-)	0.007
Dichloroethylene (cis-1,2-)	0.07
Dichloroethylene (trans-1,2-)	0.005
Dichloromethane	0.005
Dichloropropane (1,2-)	0.005
Ethylbenzene	0.7
m-Dichlorobenzene	0.6
Monochlorobenzene	0.1
o-Dichlorobenzene	0.6
p-Dichlorobenzene	0.075
Styrene	0.1
Tetrachloroethylene	0.005

Notes:
1.Maximum Contaminant Level
2.Indicates that a treatment technique is required in order to show compliance with regulations.

Table 8.3 Primary Drinking Water Standards (continued)

PARAMETERS (mg/L unless otherwise noted)	EPA Primary MCL[1]
Toluene	1
Trichlorobenzene (1,2,4-)	0.07
Trichloroethane (1,1,2-)	0.005
Trichloroethane (1,1,2-)	0.005
Trichloroethane (1,1,1-)	0.2
Trichloroethylene (TCE)	0.005
Vinyl Chloride	0.002
Xylenes (Total)	10
Synthetic Organics	
2,3,7,8-TCDD (Dioxin)	$3x10^{-8}$
2,4,5-TP (Silvex)	0.05
2,4-D	0.07
Acrylamide	TT[2]
Adipates	0.4
Alachlor (Lasso)	0.002
Aldicarb Sulfoxide	0.007
Aldicarb (Tenik)	0.007
Aldicarb Sulfone	0.007
Atrazine (Atranex, Crisazina)	0.003
Benzoapyrene (PAH)	0.0002
Carbofuran (Furadan 4F)	0.04
Chlordane	0.002
Dalapon	0.2
Dibromo-chloropropane	0.0002
Diethylhexyl Phthalate	0.006
Dinoseb	0.007

Notes:
1.Maximum Contaminant Level
2.Indicates that a treatment technique is required in order to show compliance with regulations.

Table 8.3 Primary Drinking Water Standards (continued)

PARAMETERS (mg/L unless otherwise noted)	EPA Primary MCL[1]
Diquat	0.02
Endothall	0.1
Endrin	0.002
Epichlorohydrin	TT[2]
Ethylene Dibromide	0.00005
Glyphosate	0.7
Heptachlor Epoxide	0.0002
Heptachlor (H-34,Heptox)	0.0004
Hexachlorobenzene	0.001
Hexachlorocyclopentadiene	0.05
Lindane	0.0002
Methoxychlor (Marlate)	0.04
Oxyamyl (Vydate)	0.2
Pentachlorophenol	0.001
Pichloram	0.5
Polychlorinated Biphenyls	0.0005
Simazine	0.004
Toxaphene	0.003

Notes:
1.Maximum Contaminant Level
2.Indicates that a treatment technique is required in order to show compliance with regulations.

8.2.1.2 Secondary Drinking Water Regulations

Secondary Drinking Water Regulations essentially function as guidelines for water utilities because they are not enforceable by law. This is in contrast to Primary Drinking Water Regulations discussed previously in this chapter. A total of 15 contaminants are covered by the Secondary Drinking Water Regulations. Only those with particular applicability to Avondale are presented below.

Taste and odor is an aesthetic issue which is addressed by the Threshold Odor Number (TON). The secondary standard for TON is 3. The TON test involves diluting several aliquots of the finished water with odor free water, and warming them to 110 degrees F (45ºC). One or more individuals smell the samples and the highest dilution in which an odor can be perceived is identified. The dilution of that sample is reported as the TON. Typically, the results for this test are an integer number with the lowest being 1 (i.e. undiluted sample).

Color of the water is limited to 15 Color Units (CU). Color is measured by comparing the water to known standards and determining the closest match. Color in water is usually associated with organics in the water or specific metals such as iron or manganese. Organic removal is now required by the various surface water treatment and disinfection byproduct rules. As noted below specific secondary standards exist for iron and manganese.

To avoid either corrosion or scaling of the distribution system a pH range of 6.5 to 8.5 is included in the secondary standards.

The secondary standards limit Total Dissolved solids (TDS) to 500 mg/L to improve palatability and to reduce negative physiological reaction in transient consumers. Similarly, chloride and sulfate ions are limited to 250 mg/L each. In addition, these limits tend to reduce corrosion.

Fluoride ion is limited to 2.0 mg/L to prevent negative effects on children's teeth. It should be noted that there is also a primary standard for fluoride ion of 4.0 mg/L as indicated in Table 8.3.

Iron and manganese are both limited to reduce color in the water and to prevent staining of clothing during washing. Iron is limited to 0.3 mg/L and manganese is limited to 0.05 mg/L.

8.2.2 Well Head Treatment

This section provides general information on well head treatment within the City of Avondale. More detailed treatment information, along with specific well water quality data, as well as detailed treatment costs is provided in Section 6.3.2 of the 2001 *Water Resources Master Plan*. The City of Avondale typically has good well water with the exception of two parameters, nitrate and arsenic. Nitrate

exceeds the regulatory limits in three of the existing wells and arsenic exceeds the new standard of 10μg/L in six of the wells. Wells 1, 8, and 11 exceed the nitrate standard, while Wells 1, 6, 7, 8, 14, and 15 exceed the arsenic standard. In the future, the City of Avondale will be required to locate wells which meet the water quality standards, or provide well head treatment.

The costs of wellhead treatment for either nitrate or arsenic is dependent on a number of factors: well capacity, contaminant concentration, concentrations of other parameters in the water, and physical site constraints. Nitrate removal costs are based on an ion exchange facility with brine regeneration and waste brine disposal. Assuming that approximately 60% of the future wells will require nitrate treatment, the total cost at buildout in today's dollars could be about $125,000,000 for treatment only. Annual operation and maintenance costs could be in the range of $6,000,000 to $10,000,000.

Arsenic removal costs are based on coagulation and pressure filtration with backwash disposal to the sewer. These costs are based on cost estimates for similar facilities. Annual operation and maintenance costs are typically between $0.08 and $0.20 per thousand gallons of treated water. Assuming that approximately 60% of the new wells would need arsenic removal. The total cost at buildout could be $36,000,000 to $40,000,000. Annual operation and maintenance costs could be in the range of $1,000,000 to $2,000,000 not including any additional sludge costs at the wastewater plant.

8.2.3 Surface Water Treatment Rule (SWTR)

On June 29, 1989, the EPA published the final Surface Water Treatment Rule (SWTR). For drinking water systems using surface water sources, the SWTR requires that treatment be provided to reduce turbidity, *Giardia*, *Legionella*, viruses, and heterotrophic plate count bacteria (HPC). Specifically, the SWTR established treatment and performance standards to provide a minimum reduction of 99.9 percent (3-log) for *Giardia* cysts, and 99.99 percent (4-log) for viruses. The overall reduction of *Giardia* and viruses is to be achieved through a combination of physical removal by pretreatment and filtration, and inactivation by disinfection. Treatment effectiveness under this rule is determined through turbidity measurements:

- The turbidity of representative samples of a system's combined filtered water must be less than or equal to 0.5 NTU in at least 95 percent of the measurements taken each month.
- The turbidity level of representative samples of a system's combined filtered water must at no time exceed 5 NTU.

Well operated conventional treatment plants which at least meet the 0.5 NTU effluent turbidity standard are credited with a 2.5-log removal of *Giardia* cysts and a 2-log removal of viruses. The remainder of the overall 3-log *Giardia* cyst and 4-log virus treatment is be provided by inactivation using disinfection. Compliance

with the disinfection requirement must be demonstrated by meeting minimum "CT" requirements, where C is the residual disinfectant concentration in mg/L, and T is the effective contact time in minutes with the disinfectant.

8.2.4 Interim Enhanced Surface Water Treatment Rule (IESWTR)

This rule, finalized in December 1998, is the first regulation to specifically address chlorine resistant pathogens such as *Cryptosporidium*. In addition to the requirements of the Surface Water Treatment Rule, this rule establishes a Maximum Contaminant Level Goal (MCLG) of zero for *Cryptosporidium*, the organism which causes *cryptosporidiosis*. This rule introduces a number of new monitoring requirements related to halogenated disinfection byproducts. It also lowered the combined filter effluent turbidity to less than or equal to 0.3 NTU in 95 percent of all measurements. Systems that meet the turbidity standard are assumed to provide at least 2-log *Cryptosporidium* removal through filtration. This rule also establishes criteria for systems that must establish a disinfection profile by collecting additional data related to the disinfection process and DBP formation.

8.2.5 Long-Term Enhanced Surface Water Treatment Rule and Filter Backwash Rule (LT1FBR)

This rule was finalized June 8, 2001 and is referred to as the LT1FBR, since it combines the LT1SWTR and Filter Backwash Rule. The purpose of the rule is to minimize *Cryptosporidium* concentrations in the treated water as a result of recycling sludge supernatant and filter backwash waste water to the head of the treatment plant. The major requirements of the proposed rule are as follows:

- Systems that recycle backwash waste must do so prior to the point of application of primary coagulant.
- Direct filtration plants could be required to provide detailed recycle treatment information to the State (which could then require modifications).
- Conventional treatment plants with 20 or fewer filters must conduct a one-month (one-time) self-assessment, including hydraulic flow monitoring.

8.2.6 Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)

The LT2ESWTR is scheduled for finalization in May 2002 and should become effective in May 2005. This rule requires proportional treatment levels or watershed based treatment levels based on *Giardia* and *Cryptosporidium* levels in the source water. Other pathogens such as *Cyclosporeae* and *Legionella* may also be included in this rule, if information on occurrence, health effect, and treatment demonstrate a need for these regulations.

The additional treatment requirements for *Cryptosporidium* inactivation are presented in Table 8.4, *Cryptosporidium* Inactivation Requirements Per LT2ESWTR. They are based, in part, on the assumption that conventional treatment plants in compliance with the IESWTR achieve an average of 3-log removal of *Cryptosporidium*. The total *Cryptosporidium* removal requirements for the action bins with 1-log, 2-log and 2.5-log additional treatment correspond to total *Cryptosporidium* removals of 4-log, 5-log and 5.5-log, respectively.

Table 8.4 *Cryptosporidium* Inactivation Requirements Per LT2ESWTR

Bin No.	Average Source Water *Cryptosporidium* Concentration	Additional treatment requirements for systems with conventional treatment that are in full compliance with IESWTR
1	*Cryptosporidium* < 0.075/L	No action
2	0.075/L < *Cryptosporidium* < 1.0/L	1-log treatment (systems may use any technology or combination of technologies from toolbox as long as total credit is at least 1-log)
3	1.0/L < *Cryptosporidium* < 3.0/L	2-log treatment (systems must achieve at least 1-log of the required 2-log treatment using ozone, chlorine dioxide, UV, membranes, bag/cartridge filters, or in-bank filtration)
4	*Cryptosporidium* ≥ 3.0/L	2.5-log treatment (system mush achieve at least 1-log of the required 2.5-log treatment using ozone, chlorine dioxide, UV, membranes, bag/cartridge filters, or in-bank filtration)

8.2.7 Stage 1 and 2 Disinfectants/Disinfection By-Products Rules (D/DBPR)

The Stage 1 D/DBPR has been finalized and becomes effective for public water systems serving more than 10,000 people in December 2001. This rule replaces the previous total trihalomethane (TTHM) standard of 0.1 mg/L (100 μg/L) with 0.08 mg/L (80 μg/L). It also regulates the sum of five haloacetic acids (HAA-5) at 60 μg/L, and establishes the MCL for bromate ion (BrO^{-}_{3}) at 10 μg/L. This rule also requires total organic carbon (TOC) monitoring and TOC removal to be investigated by enhanced coagulation or enhanced softening. The rule further specifies the percentage of influent TOC that must be removed based on the raw water TOC, ultraviolet (UV) light absorbance, and alkalinity levels, as shown in Table 8.5, Percentage of TOC Reduction Requirements Per Stage 1 D/DBPR.

Table 8.5 Percentage of TOC Reduction Requirements Per Stage 1 D/DBPR

Raw Water TOC (mg/L)	Raw Water Alkalinity (mg/L as $CaCO_3$)		
	< 60	**60 – 120**	**> 120**
> 2 – 4	35%	25%	15%
> 4 – 8	45%	35%	25%
> 8	50%	40%	30%

TTHMs include the following four compounds:
- Chloroform
- Bromoform
- Bromodichloromethane
- Dibromochloromethane

The five regulated haloacetic acids are:
- Chloroacetic acid
- Dichloroacetic acid
- Trichloroacetic acid
- Bromoacetic acid
- Dibromoacetic acid

The Stage 2 version of this rule is scheduled for finalization in May 2002 and should become effective in May 2005. MCLs will remain at 80 μg/L for TTHM and at 60 μg/L for HAA-5 but must be calculated using the Locational Running Annual Average (LRAA). Each site of sampling must not exceed these MCLs. In addition, the MCL for bromate ion remains at 10 μg/L, based upon current alternative technology utilization and upon current understanding of bromate ion formation as a result of bromide concentrations. EPA is committed to review the bromate ion MCL as part of a 6-year review to determine whether the MCL should remain at 10 μg/L or be reduced to 5 μg/L or a lower concentration.

8.2.8 Radionuclides Rule

On December 7, 2000, the EPA announced updated standards for radionuclides and a new standard for uranium, as required in the amendments to the 1986 Safe Drinking Water Act (SDWA). The revised standards are as follows:

- Combined Radium, 226 and 228 5 pCi/L
- Total Beta emitters 4 mrem
- Gross Alpha, MCL 15 pCi/L
- Uranium, MCL 30 μg/L

This rule becomes effective on December 8, 2003. The monitoring requirements are being phased in from December 2000 to December 2003. Water systems will determine initial compliance under the new monitoring requirements using

the average of four quarterly samples, or at State direction, using appropriate grandfathered data.

8.2.9 Arsenic Rule

On January 22, 2001, EPA proposed a reduction in the current arsenic standard from 50 μg/L to 10 μg/L. Due to delays in the announcement of the proposed rule the original effective date of February 22, 2001 was extended until February 22, 2002. The proposed new standard has significant cost implications for water utilities, and as such, significant debate surrounding the revised MCL is ongoing. The Bush administration is reviewing the rule, and various State Senators have proposed amendments. Currently, EPA wishes to retain the original compliance date of January 22, 2006, by compressing the 5-year compliance timeframe provided in the January 22, 2001 rule.

8.2.10 Lead and Copper Rule

The USEPA established the Lead and Copper Rule in 1991. This rule is different from the others because it applies at the customer's tap rather than at the plant or in the distribution system as the other rules do. Action levels for lead and copper, 0.015 mg/L and 1.3 mg/L respectively, have been established to minimize corrosion in the distribution system. In addition the rule seeks source water treatment, if appropriate, public education and lead service line replacement. The City of Avondale tests for lead and copper in their water system every three years. The last set of testing was completed in July 2001.

8.2.11 Groundwater Disinfection Rule

This rule will establish disinfection requirements against microbial contamination for groundwater systems. For systems which are not disinfecting sanitary surveys, hydrologic sensitivity studies and, potentially, source water monitoring will be required. For systems that are disinfecting compliance monitoring for 4-log (99.99%) virus inactivation or removal will be required. This rule was proposed in May of 2000 and should be finalized in early 2002.

8.3 Entrepreneurial Opportunities

The most obvious and immediately available additional water resource is the City's reclaimed water. The reclaimed water can be recharged and credited against ground water pumping. As shown in Section 6.2 of the 2001 *Water Resources Master Plan*, approximately 38% of the potable water pumped into the water distribution system becomes reclaimed water available for recharge water credits. If water conservation initiatives are put into place to limit outdoor water use in the future, this percentage can be expected to continue. At build-out, recharge of the reclaimed water would reduce the annual deficit to about 3,000 acre-feet per year. Recharge of the reclaimed water in the current recharge area should begin as soon as possible to begin banking long term storage credits if they are not required to offset current year groundwater pumping deficits.

The City of Avondale recharges all of its surface water supply to offset its ground water pumping and has additional recharge capacity. Avondale should maximize this recharge to obtain maximum water banking credits within the limits of the annual requirement for zeroing out the SRP groundwater supplied and used, as well as other groundwater used On and Off-Project.

Surface water sources available to Avondale include Central Arizona Project (CAP) water and Salt River Project (SRP) water. Avondale has an assured water supply allocation of 4,099 acre-feet per year of CAP water plus another 647 acre-feet per year of CAP water from the McMicken Irrigation District. Opportunities to enhance this supply include:

- Purchase and recharge of incentive water. This is water that can be purchased from year to year, as water in excess of other subcontracted demands is available. This water can be recharged and banked for future credit against groundwater pumping. Avondale should actively pursue this water source while it is available.
- Reallocation. Another reallocation of CAP water is expected in the next few years. Avondale should apply for a portion of the reallocation. Based on past reallocations Avondale might expect to acquire less than 200 acre-feet per year and perhaps significantly less.
- Lease of Indian CAP Water Rights. Some Indian water is currently available for lease and it appears that additional Indian lease water will be available in the future. A long-term lease of Indian water is probably the most stable source of supply available to Avondale and should be pursued. Care must be exercised to ensure compliance with the limits on the quantity of lease water that can be used as a part of assured water supply.
- Groundwater savings (Also known as “In-lieu” recharge). While not a source of water as such, groundwater savings can be an effective way to obtain the benefits of incentive water or Indian lease water without transporting the water to Avondale. These sources of water can be incorporated into a Groundwater Savings Facility permit to bank credits for future use. In this method, surface water owned by Avondale is supplied to a user currently using ground water so that the user reduces groundwater use by the amount of water supplied by Avondale. Avondale then gets the credits for the groundwater not pumped. Maricopa Water District and SRP are potential entities for groundwater savings.

Based upon 1990 population data, Avondale has an SRP assured water supply allocation of 8,463 acre-feet per year under build-out conditions. This is the amount of water available to Avondale from SRP during the worst drought conditions. SRP water is limited to use on Project and it cannot be stored for use

in subsequent years. The potential opportunity for additional water supply from SRP or other entities with canals is for Avondale to pay to line some or all of a canal and obtain the rights to the water saved. Avondale should also request SRP to update the Assured Water Supply Study for Salt River Project Member Lands. Based upon increased population and development within Avondale since the last report was prepared, the SRP assured water supply allocation could increase significantly.

Avondale is a member of the Central Arizona Groundwater Replenishment District (CAGRD) and has an assured water supply from them of 0 to 3,444 acre-feet per year.

In addition to the possibility of recharge of the reclaimed water, discussed above, reclaimed water could also be exchanged with Palo Verde, RID or other water users. While these are possibilities they would probably be more costly than a pump station and pipeline from the wastewater plant to the recharge site.

Other possibilities include purchase of surface water rights from RID, if they have any, as their demand for water decreases as the land is urbanized. Also there are assured water supply credits for sale from grandfathered groundwater pumping rights. As these sources become known, Avondale should explore them to increase the City's assured water supply. Another possibility is the water rights associated with the land under the I-10 freeway. If these rights have not been claimed by some entity they might be available to Avondale. Further, Avondale can require as a part of the platting process, that all irrigation grandfathered rights associated with all new subdivision lands be transferred to Avondale.

Another possibility would be to pump and treat water from the water logged area along the southern boarder of the City. This would only be viable if Arizona Department of Water Resources (ADWR) would consider the reduction of the water logging as a positive action and not include the water from this area in the groundwater pumped and charged against the City. Also, the City would have to evaluate the amount of water that could be pumped prior to investing in this approach. This approach would potentially be breaking new ground with DWR so it might take some extra effort.

8.4 Lost and Unaccounted for Water

Lost and unaccounted for water is the difference between the water pumped into the water system and the water used, or metered, in the water system. No lost and unaccounted for water estimates are available for the year 2000 due to problems in resolving differences in metered amounts for the year.

Even though the quantity of lost and unaccounted for water has been indeterminate, Avondale experienced water loss in 2000. The lost and

unaccounted for water in 1999 was reported as 8.1%, which is below the ADWR goal of 10% (See City of Avondale Annual Water Report). For purposes of this Master Plan, and until a reported amount can be determined, the lost and unaccounted for water is assumed to be 8% for the year 2000.

Although Avondale is currently in compliance with ADWR's goal for lost and unaccounted for water, the City is interested in finding ways to lower the percentage of lost and unaccounted for water. The following suggested programs will help the City to reduce this percentage.

1. Leak Detection and Repair

In 1998, the City completed an aggressive program to identify and eliminate leaks within the water system. Additional water loses will develop over time. Therefore, the city should consider incorporating an annual leak detection program to identify and repair leaks in the system.

The City may also consider monitoring or auditing the computer-generated water management reports to identify unusual excursions from normal water use. The reports can be used to track water use by individuals as well as by categories.

2. Meter Testing and Replacement

The City completed a multi-year meter replacement program in 2001 to convert the City's meters from manual read to touch read meters. The City should consider the implementation of a regular meter testing program for all production (well) meters and commercial meters over 3-inches in size. In addition, it is recommended that residential meter testing be performed based on an inventory of the different meter ages within the City, as well as the different meter brands and types.

Meter replacements, based on meter test results, will allow the City to maintain an efficient water system and reduce the percentage of lost and unaccounted for water.

3. Public Education

Public education can be one of the best tools to effectively reduce lost and unaccounted for water. Educated citizens can provide valuable information about water leaks and unauthorized water uses. The water conservation materials, distributed by the City, should provide contact information for reporting water line breaks or other water wasters within the City. A crew that can field phone calls and analyze and repair reported problems should be formed.

4. Legal Unmetered Water Use

Public use, fire services and fire-flow testing, street cleaning, irrigation, hydrant flushing, valve exercising, and other operational uses may go unmetered. It is

expected that these uses produce some lost and unaccounted for water, but programs could be implemented to keep the percentage to a minimum.

It is suggested that fire services and fire-flow tests be metered when possible. This program would require City personnel to monitor fire flow testing within the city. A hydrant meter should be placed at testing locations, and a record of water usage should be kept.

In addition, the City should inventory City-owned landscape areas that are not metered. Meters can be installed in locations with significant demand. Flows may also be estimated based on irrigation program parameters.

5. Illegal Unmetered Water Use

A program should be established to identify illegal or undetected connections to the City's water system. Unmetered and/or unaccounted for withdrawals are costs that the City absorbs with essentially no recourse for compensation. City field personnel, including inspectors and public safety forces, should be trained to identify and respond to illegal connections to the City water system. One possible source of unaccounted for water may originate from contractor use of unmetered construction water. This is a source that can easily be identified by City forces.

The preceding programs provide general guidance for minimizing lost and unaccounted for water. A detailed water audit would isolate actual sources of lost water and, therefore, provide a more detailed and specific plan for improvement. If future lost and unaccounted for water exceeds standards set by ADWR, it is recommended that a detailed water audit be performed.

8.5 Reclaimed Water Plan

In order to establish sufficient water resources for water users within the City of Avondale, it is recommended that the City of Avondale implement a reclaimed water plan to take advantage of this constant and renewable water resource. A detailed discussion of a reclaimed water plan which has been developed for the City of Avondale is provided in Section 6.2, Reclaimed Water Plan, of the 2001 *Water Resources Master Plan*.

8.6 Water Master Plan Update

Over the next 5 years, many changes will take place in the City of Avondale. The City will continue to grow and expand as residential and commercial growth continues to occur. In order to ensure that the plans presented in this Water Infrastructure Master Plan continue to provide the most effective guidance to the City of Avondale, it is recommended that this Water Infrastructure Master Plan be revised and updated every 5 years.

8.7 Redundant Water Supply

Possibilities for developing a redundant water supply for the City of Avondale were investigated. The primary option that was investigated was the development of a backup water agreement with another City or water company. The first possibility that was investigated would be the development of a backup agreement with the City of Phoenix. The City of Phoenix water company was investigated to evaluate the feasibility of this option, and they said that it would be a feasible option. In order to establish this agreement, a connection to the City of Phoenix water system would need to be established. Additionally, an intergovernmental agreement would need to be established between the City of Phoenix and the City of Avondale. The exact terms of this agreement would need to be negotiated.

8.8 Development Fees

In order to help fund water system expansion and improvements, the City of Avondale charges a water development fee (for infrastructure improvements such as well pumps, reservoirs, booster stations and distribution systems) and a water resources development fee (for development of water supplies). For a residential development, the City charges $750 each per unit for the water resources and water infrastructure fees. Therefore, a total development charge of $1,500 for water improvements for each residential unit is assessed. A comparison of water related development fees for surrounding communities is provided in Table 8.6, Water Development Fees.

Table 8.6 Water Development Fees

City	Water Resources	Water Infrastructure	Total
Avondale	750	750	$ 1,500
Goodyear	3,000	1,000	$ 4,000
Peoria	4,022		$ 4,022
Surprise*	824	1,770	$ 2,594

*In addition to the water development fees listed in the table, the City of Surprise also requires the developer to construct 3,000 gpm of well supply, a 1,000,000 reservoir, and any associated transmission mains for each square mile of development. The City of Surprise will then reimburse the developer for these improvements up to the amount of impact fees paid by the developer.

Based on the significant amount of growth occurring in the City as well as a comparison to surrounding communities, the City of Avondale is considering increasing their water development fees in order to better offset the increased costs associated with these new developments. It is important to note that the development fees listed above do not reflect the costs associated with Arsenic treatment. It is likely that surrounding communities will increase their water development fees in order to comply with the Arsenic rule requirements that become effective in the year 2006. The City of Avondale performed a development fee analysis to determine the maximum supportable development fees within their City.

The development fee analysis performed by the City of Avondale determined that the maximum supportable development fee for a single-family home is $3,289. It is recommended that the City increase their water resources development fee to this amount. Table 8.7, Development Fee Revenue Analysis, provides a comparison of the amount of revenue that will be generated over each of the planning periods for both the current and proposed water development fees. Table 8.8, Total CIP Project Cost shows a breakdown of the total costs for each of the CIP's. It can easily be seen that even by increasing the water development fees, they alone will not pay the costs of the recommended capital improvements. The City will utilize other methods to pay for the remaining projects listed in the CIP.

Table 8.7 Development Fee Revenue Analysis

Fee Schedule	2006	2011	2026	2040
Current ($1,500)	$10,800,000	$10,012,500	$30,465,000	$8,256,000
Proposed ($3,289)	$23,680,800	$21,954,075	$66,799,590	$18,102,656

Table 8.8 Total CIP Project Cost

5 Year	10 Year	20 Year
$45,826,000	$37,095,000	$64,450,000

In addition to development fees, the City of Avondale should utilize other revenue sources to fund the capital improvement projects. Some of these other sources may include the General Fund, a portion of the City's sales tax, grants, and other methods deemed appropriate by the City.

9.0 Capital Improvement Program

Based on the water system hydraulic analysis, population growth and associated demands, the reliability test recommendations, and discussions with City staff, a Capital Improvement Plan (CIP) has been developed. The program which has been developed in this report consists of a 5-year CIP (2002-2006), a 10-year CIP (2007-2011), and a Long Term CIP (2012-2026). This CIP provides an inventory of the projects recommended for construction in order to meet the water system demand and pressure requirements. The CIP also provides a schedule of recommended improvements and corresponding cost estimates. Figure 9.1, CIP Progress, shows the proposed increase in storage and supply along with the projected demands. In order to have a sufficient water supply, the supply must always be greater than the average demands as shown in the figure below. Both the average day demands and maximum day demands are shown on the graph. Supply exceeds the max day demands in order to have back up redundancy in the system, and to meet the peak hour demands.

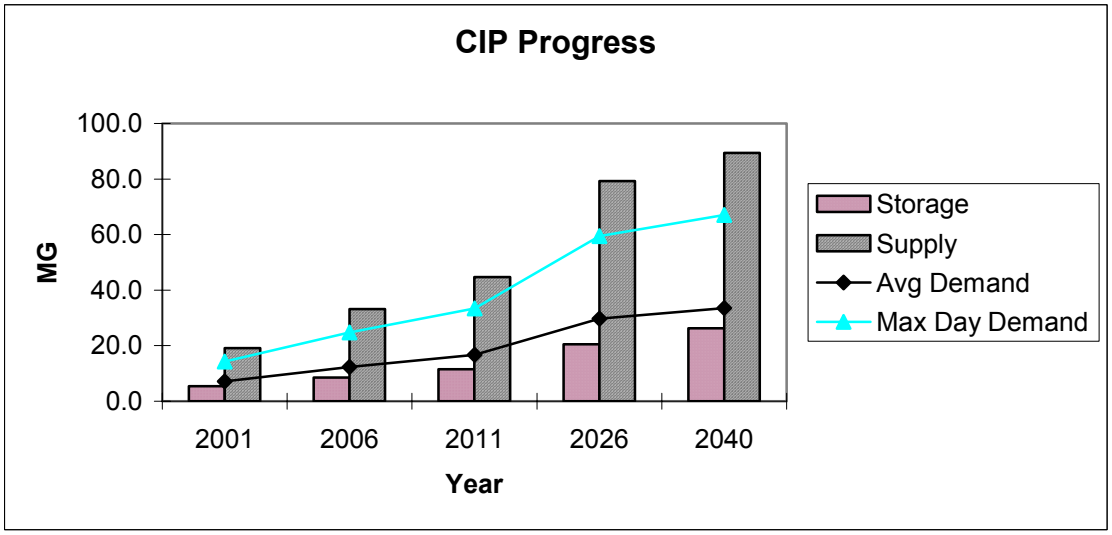


Figure 9.1 CIP Progress

9.1 Cost Estimate Background

The cost estimates which are provided in the CIP provide a general planning level cost for each of the projects which are recommended. It is recommended that a detailed cost analysis be performed with the design of the projects. The costs computed were based on actual project costs for similar projects that were constructed in the area. These costs were compared to the *Means Standard Cost Estimating Handbook* for standardization purposes. All costs provided are present day costs. The method for determining the costs for each of the project types is described below:

9.1.1 Waterlines

The waterline cost estimates take into consideration the size of pipe and installation, the length, pavement removal and replacement, engineering design, construction management, and a contingency. Additionally, project specific costs were estimated, these included river or railroad crossings. The cost for each of the waterlines was estimated using these factors, and are shown in the tables below.

9.1.2 Reservoirs and Booster Stations

The cost estimates for reservoirs and booster stations were determined by the size and number of reservoirs, land acquisition, booster pump requirements, design, construction management, and a contingency. It was assumed that for each reservoir, one acre of land would be required for each reservoir on site plus one additional acre of land for the site improvements. Therefore, a site with two reservoirs would require three acres. The costs for each of the reservoirs and booster stations are shown in the tables below.

9.1.3 Wells

The cost estimates for wells are broken down into three separate phases, with the improvements being shown as taking place over a three year time period. The first phase cost includes drilling the well, and installing the casing. The second phase cost includes equipping the well. The third phase cost represents nitrate and arsenic treatment facilities for the well. The equipping phase of the well also includes the cost of the constructing up to a half-mile transmission main for each well. Table 9.1, Itemized Well Cost Estimate, provides a detailed breakdown of the well cost estimate. More specific information on locating and acquiring wells can be found in Section 5 of the 2001 *Water Resources Master Plan.* The treatment costs vary based on the estimated production capacity for each of the wells. For cost estimating purposes, nitrate treatment facilities costs are shown as $2,000,000 per 1,000 gpm and arsenic treatment facilities as $1,000,000 per 1,000 gpm. It is important to note that any specific well may or may not require nitrate and arsenic reduction. There are possibilities other contaminants may be present that require treatment. It was also estimated that approximately 60% of new wells will require treatment facilities. Therefore, for estimating purposes 60% of the wellhead treatment costs are included with each well. The treatment costs are one of the areas where the greatest variability may be found. For instance the quality of water is not known, as well as future federal regulations. The costs presented here are to provide a planning estimate only.

Table 9.1 Itemized Well Cost Estimate (1,000gpm)

Item	Cost
Pilot hole, casing, & testing	$600,000
Equiping	$300,000
Transmission line (1/2 Mile)	$100,000
Subtotal	$1,000,000
Treatment (60% of Nitrate & Arsenic)	$2,000,000
Total	$3,000,000

9.2 5-year CIP (2002-2006)

The recommended 5-year CIP is described in Table 9.2 below. The CIP is delineated into three separate categories: transmission mains, storage tanks (includes booster station), and wells. The immediate improvements which were described in Section 7.4, Existing System, have also been included in this CIP.

9.3 10-year CIP

A 10-year CIP was also developed for the City's water distribution system based on the water model which was performed. The 10-year CIP follows the same format as the 5-year CIP. The 10-year CIP can be seen in Table 9.3.

9.4 Long Term CIP

The Long Term CIP provides a more generalized plan of items that will need to be constructed as the City develops. The Long Term CIP is to be used to provide guidance as to the amount of money that needs to be budgeted, and the types of projects that may need to be undertaken. Due to the many changes which may occur in the City over the next 20 years, RBF recommends that the City update the water master plan every five years, in order to accurately reflect the progress that has been made, and the areas that need to be improved. The Long Term CIP can be seen in Table 9.4.

Table 9.2 5 year CIP

Waterline						
Waterline	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Van Buren from Agua Fria River to 115th Ave – 36" Waterline	2,000,000					2,000,000
Indian School from El Mirage to 119th Ave – 20" Waterline	950,000					950,000
99th Ave from Well #8 to Thomas Rd - 16" Waterline	500,000					500,000
Thomas Rd from 107th Ave to 99th Ave – 16" Waterline		900,000				900,000
Lower Buckeye Rd from west bank of Agua Fria River to El Mirage – 16" Waterline			750,000			750,000
Encanto from 103rd Ave to 99th Ave - 12" Waterline			250,000			250,000
107th Ave from Buckeye Rd to Van Buren– 16" Waterline				800,000		800,000
Brinker Dr. alignment from 115th Ave to 107th Ave – 12" Waterline					375,000	375,000
Buckeye Rd from 119th Ave to 115th Ave – 12" Waterline					200,000	200,000
Dysart Rd from Buckeye to Lower Buckeye – 12" Waterline					1,500,000	1,500,000
Waterline Replacement and Upgrade – Old Town and East Avondale*	50,000	50,000	50,000	50,000	50,000	250,000
Subtotal	3,500,000	950,000	1,050,000	850,000	2,125,000	8,475,000

Table 9.2 5 year CIP-(Continued)

Storage and Booster Stations						
Storage and Booster Stations	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Garden Lakes Reservoir and Booster Station – 2.5 MG Storage and 3,000 gpm Booster Pumps		1,500,000				1,500,000
Coldwater Springs Reservoir and Booster Station – Phase 2 – 2.5 MG Storage and 4,000 gpm Booster Pumps			1,500,000			1,500,000
Well 8 Reservoir – Phase 2 – 1.0 MG Storage				750,000		750,000
Lower Buckeye Reservoir and Booster Station – 119th Ave and Lower Buckeye Rd – 1MG Storage and 2,500 gpm Booster Pumps					1,000,000	1,000,000
Subtotal	-	1,500,000	1,500,000	750,000	1,000,000	4,750,000
Wells						
Wells	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Coldwater Springs Well – 2,000 gpm	600,000	400,000	4,000,000			5,000,000
Garden Lakes Well – 4,000 gpm		600,000	400,000	8,000,000		9,000,000
Additional Wells – 2,000 gpm		600,000	400,000	4,000,000		5,000,000
PIR Well - 1,000 gpm			600,000	400,000	2,000,000	3,000,000
Annual Hydrogeological Testing	250,000	250,000	250,000	250,000	250,000	1,250,000
Subtotal	850,000	1,850,000	5,650,000	12,650,000	2,250,000	23,250,000

Table 9.2 5 year CIP (Continued)

Miscellaneous						
Miscellaneous	**2002/2003**	**2003/2004**	**2004/2005**	**2005/2006**	**2006/2007**	**Total**
Well No. 6 Wellhead Treatment - upgrade existing well to comply with arsenic rule	1,500,000					1,500,000
Well No. 7 Wellhead Treatment - upgrade existing well to comply with arsenic rule		1,500,000				1,500,000
Existing Infrastructure SCADA system upgrade		201,000	200,000			401,000
Well No. 14 Wellhead Treatment - upgrade existing well to comply with arsenic rule			800,000			800,000
Well No. 15 Wellhead Treatment - upgrade existing well to comply with arsenic rule				750,000		750,000
Well Head Treatment – existing wells - upgrade existing wells for nitrate removal				2,000,000	2,000,000	4,000,000
Water Master Plans Update					400,000	400,000
Subtotal	1,500,000	1,701,000	1,000,000	2,750,000	2,400,000	9,351,000
Total	**$5,850,000**	**$6,001,000**	**$9,200,000**	**$17,000,000**	**$7,775,000**	**$45,826,000**

Table 9.3 10-year CIP

Waterline						
Waterline	**2007/2008**	**2008/2009**	**2009/2010**	**2010/2011**	**2011/2012**	**Total**
111th Ave Alignment from Miami to Ellwood – 12" Waterline			365,000			365,000
Elwood from 115th Ave to 107th Ave – 12" Waterline				365,000		365,000
115th Ave from Lower Buckeye to Southern Ave – 24" Transmission Line				2,000,000		2,000,000
115th Ave from Southern to PIR bridge and Indian Springs Rd. from PIR bridge to El Mirage – 16" Waterline				1,300,000		1,300,000
Indian School from 107th Ave to 99th Ave – 16" Waterline					900,000	900,000
99th Ave from Thomas Rd to Indian School Rd – 16" Waterline					900,000	900,000
127th Ave from Lower Buckeye to Pioneer St					365,000	365,000
Waterline Replacement and Upgrade -East Avondale, Old Town and North Avondale	50,000	50,000	50,000	50,000	50,000	250,000
Subtotal	50,000	50,000	415,000	3,715,000	2,215,000	6,445,000
Storage and Booster Stations						
Storage and Booster Stations	**2007/2008**	**2008/2009**	**2009/2010**	**2010/2011**	**2011/2012**	**Total**
Roosevelt Park Reservoir and Booster Station – 107th Ave and Van Buren – 2.5 MG (2 – 2.5 MG) Storage and 4,000gpm Booster Pumps	1,700,000					1,700,000
Miami Reservoir and Booster Station – 115th Ave and Miami – 2.5 MG (Storage and 3,000 gpm Booster Pumps			1,700,000			1,700,000
PIR Reservoir and Booster Station at Indian Springs and El Mirage – 1 MG Storage and 1,000 gpm Booster Pumps				1,000,000		1,000,000
Subtotal	1,700,000	-	1,700,000	1,000,000	-	4,400,000

Wells						
Wells	**2007/2008**	**2008/2009**	**2009/2010**	**2010/2011**	**2011/2012**	**Total**
Roosevelt Park Well – 107th Ave and Van Buren – 3,000 gpm	600,000	400,000	6,000,000			7,000,000
Lower Buckeye Well – 119th Ave and Lower Buckeye Rd – 2,000 gpm		600,000	400,000	4,000,000		5,000,000
Miami Well – 115th Ave and Miami – 3,000 gpm			600,000	400,000	6,000,000	7,000,000
Subtotal	600,000	1,000,000	7,000,000	4,400,000	6,000,000	19,000,000
Miscellaneous						
Miscellaneous	**2007/2008**	**2008/2009**	**2009/2010**	**2010/2011**	**2011/2012**	**Total**
Annual Hydrogeological Testing	250,000	250,000	250,000	250,000	250,000	1,250,000
Reclaimed Water System			2,500,000	2,500,000	1,000,000	6,000,000
Subtotal	250,000	250,000	2,750,000	2,750,000	1,250,000	7,250,000
Total	**$2,600,000**	**$1,300,000**	**$11,865,000**	**$11,865,000**	**$9,465,000**	**$37,095,000**

Table 9.4 Long Term CIP

Long Term CIP -- Ultimate	
Improvements	**Cost**
Waterline	
Broadway Rd from Dysart Rd to 107th Ave – 16" Waterline	$ 1,400,000
Southern Ave from Dysart Rd to 107th Ave – 16" Waterline	$ 1,750,000
107th Ave from Broadway to Southern Ave – 16" Waterline	$ 900,000
Dysart Rd from Southern Ave to Broadway – 16" Waterline	$ 900,000
Van Buren from 107th Ave to 100th Ave – 16" Waterline	$ 900,000
Indiana School Rd from Dysart Rd to Sante Fe Trail – 16" Waterline	$ 500,000
Encanto from 115th Ave to 112th Ave – 12" Waterline	$ 200,000
Buckeye Rd from west bank of Agua Fria River to El Mirage – 20"	$ 750,000
Dysart Road from Sage Creek to Indian School Rd. - 16" Waterline	$ 250,000
Broadway alignment from Dysart to Litchfield Rd- 12" Waterline	$ 900,000
El Mirage Road from Indian Springs to Southern Ave. - 16" Waterline	$ 950,000
Indian Springs Road from El Mirage Rd to PIR. - 16" Waterline	$ 250,000
Subtotal	$ 9,650,000
Reservoirs and Booster Stations	
Southern Ave Reservoir & Booster Station - 115th Ave & Southern Ave – 1.0 MG Storage & 3,000 gpm Booster Pumps	$ 800,000
Thomas Rd.Reservoir & Booster Station – 115th Ave & Thomas Rd – 2.5 MG Storage & 4,500 gpm Booster Pumps	$ 1,900,000
Littleton Reservoir & Booster Station – 115th Ave & Buckeye Rd – 2.5 MG Storage & 9,000 gpm Booster Pumps	$ 2,000,000
Lakin Reservoir & Booster Station – 115th Ave & Broadway – 2.5 MG Storage & 9,000 gpm Booster Pumps	$ 2,000,000
Coldwater Springs Reservoir & Booster Station – Phase 3 – 2.5 MG Storage & 6,000 gpm Booster Pumps (upgrading existing pump)	$ 1,900,000
Subtotal	$ 8,600,000
Wells	
Southern Ave Well – 115th Ave & Southern – 3,000 gpm	$ 7,000,000
Thomas Rd Well – 115th Ave & Thomas – 3,000 gpm	$ 7,000,000
Littleton Well – 115th & Buckeye Rd – 7,000 gpm	$ 10,000,000
Lakin Well – 115th & Broadway Rd – 7,000 gpm	$ 10,000,000
Additional Wells – 12,000 gpm	$ 12,000,000
Subtotal	$ 46,000,000
Miscellaneous	
4th St from Riley to Lawrence – 8" main	$ 200,000
Subtotal	$ 200,000
Total	**$ 64,450,000**

10.0 References

10.4 Section 1

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Water Infrastructure Master Plan

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