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COMPARATIVE HYDRAULIC AND COST ANALYSES OF SINGLE AND DUAL WATER DISTRIBUTION SYSTEMS USING NONPOTABLE WATER FOR URBAN IRRIGATION IN NORMAN, OKLAHOMA

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COMPARATIVE HYDRAULIC AND COST ANALYSES OF SINGLE AND DUAL WATER DISTRIBUTION SYSTEMS USING NONPOTABLE WATER FOR URBAN IRRIGATION IN NORMAN, OKLAHOMA

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Abstract

Drought has caused water resource shortages throughout Western and Midwestern United States for many years, particularly in California, Nevada, Arizona, Texas, and Oklahoma. This has led many communities to evaluate alternate water sources, including water reuse and raw water for nonpotable uses. Commonly, reuse water is used for commercial or industrial uses, such as irrigating golf courses or providing cooling water in factories. However, water reuse and nonpotable water have rarely been evaluated for residential and commercial landscape irrigation (urban irrigation), exceptions being St. Petersburg, Florida and Rouse Hill, Australia. This is mainly attributed to the small scale of these users, the complexity of serving many users, and concern about cross contamination on a large scale; urban irrigation, however, can account for the majority of a community's water use during the summer and almost a third of the water use during spring and fall months. This research project uses hydraulic models to simulate using nonpotable water for urban irrigation in Norman, Oklahoma, with effluent from the Norman Water Reclamation Facility and nonpotable water from wells that do not meet the EPA Arsenic Rule. The hydraulics and costs are compared for a hypothetical dual distribution system with the existing single distribution system. The research also evaluates how reducing the potable demand affects the need for Norman to find additional water sources by 2062. The cost comparison includes developing new water sources, construction costs for increasing treatment capacity for the single and potable systems and installing a secondary distribution system, and operations and maintenance costs for the single and dual distribution systems. The results found that utilizing existing water resources (e.g., wastewater effluent and nonpotable wells) and installing a secondary distribution system to provide nonpotable water for a limited service area can ultimately

be more cost-effective than developing new water sources to supply the 2062 water demands for Norman. Ultimately, this project found that constructing a secondary water distribution network can be a cost-effective alternative to help reduce demands on the existing water distribution system and a way to improve reliability of the potable distribution system.

Chapter 1: Introduction

Background

Between 2010 and 2013, extreme drought throughout Oklahoma and the Southern United States strained water utilities and the already diminished water supplies. As a result, many communities struggled to meet the demands caused by population growth and increased irrigation. With surface water levels dropping precipitously with each drought and various contaminants entering ground and surface water supplies, it becomes more expensive and challenging to treat water to potable levels. An additional issue for water supply providers is that, during the summer months, increased demand caused by irrigation can constitute up to 50 percent of a municipal's total water demand (Water Sense, 2012). This has led to a renewed interest in the same issues that many municipalities faced in the 1960s and 1970s. As Dracup wrote in 1978, "[t]hese problems cannot be solved by simply ignoring them or wishing them away. They must be faced head on with imaginative yet realistic solutions that are implementable in today's complex environment." Just as then, we operate in a complex environment that has more and more stringent regulations and more demands on every system. Some communities enforce water rationing, while others actively seek new water supply sources. One new water supply source has been to reuse wastewater treatment plant effluent as reclaimed water for irrigating golf courses, cemeteries, and parks or for other nonpotable uses including toilet flushing and even firefighting in a secondary or dual distribution system. The American Water Works Association (AWWA) defines a dual distribution system as "two separate water piping systems distributing water to customers, one carrying potable water and the other conveying less-quality water (e.g., non-potable reclaimed water) for reuse purposes” (2009). However, very few communities around the world have dual distribution systems that use reclaimed water for residential or urban irrigation.

The majority of municipal irrigation water is used for residential irrigation. So, one imaginative yet hopefully realistic solution would be to use reclaimed water for residential irrigation. Using reclaimed water for residential irrigation could reduce the potable demand by up to 50 percent in the summer, which would consequently help conserve freshwater resources for future use. To incorporate this, the residential system would be most effective in cities or towns that already use reclaimed water to irrigate green spaces, for industrial uses, or in areas of new development. This could help offset the cost of constructing a completely new distribution system or it could save the costs of retrofitting existing neighborhoods for residential irrigation.

In the United States, 44 states have regulations or guidelines for water reuse (EPA, 2012). Out of these 44, the three main reuse states are California, Colorado, and Florida. All three started water reuse programs in the 1960s and 1970s. Since then, many states have adopted their own water reuse regulations and guidelines, while California and Florida continue to be the leaders in water reuse. Other states that are gaining prominence include Virginia, Texas, Arizona, and Nevada. Arizona is arguably the first state to use reclaimed water formally, with the Grand Canyon Village water reuse program in 1926 (Asano et al., 2007). Since then, Arizona has developed numerous water reuse programs for municipal, industrial, and agricultural uses. Nevada has developed its water reuse to the extent that 40 percent of its total water supply is reclaimed water (Southern Nevada Water Authority, 2014). In Virginia, Fairfax County provides reclaimed water to the community for nonpotable uses. In Texas, nonpotable reclaimed water has been used since the late 1960s and a current project in Big Spring has implemented the first direct potable reuse program in the U.S. (Alan Plummer & Associates, 2011). The other states with reclaimed water do not have such robust or notable programs as these aforementioned seven states.

In the United States, most regulations currently prohibit using reclaimed water for residential purposes due to difficulties in characterizing and treating contaminants, potential liability issues caused by public interactions, and the possibility of cross-connections with the potable water supply. Many of the dual water distribution systems use reclaimed water for industrial purposes, such as cooling water, or for restricted-access irrigation applications, such as private golf courses. The high cost of constructing a secondary system also deters many water providers and municipalities from installing a second system. However, as municipalities grow and fresh water resources decrease, the cost of finding alternate water supplies and building bigger and more complex water treatment plants may eventually offset the cost of a secondary system.

One of the primary reasons to implement reclaimed water is to substitute it for potable water (Asano et al., 2007). Residential water demand comprises the majority of typical municipal water demand. As stated earlier, summer irrigation demand constitutes up to 60 percent of the total demand (EPA, 2014). In the city of Norman, analysis has shown that summer demand can increase by 20 percent over the average day demand and almost 75 percent over the minimum month demand. The entirety of this water is treated to potable standards, even though the end use is nonpotable. Some of the large end-users make sense for nonpotable reuse, such as industrial cooling towers, parks, and golf courses. However, there may be some benefit to using reclaimed water for the smaller end-users along with large end-users to better reduce the potable demands.

Since interest in reclaimed water declined after the 1970s, Kang and Lansey (2012) were among the first to publish a recent evaluation or direct comparison of dual water distribution systems for residential end-use. Beyond this one study, little to no research has been conducted that couples a hydraulic analysis with a cost analysis to compare single versus dual water distribution systems for residential and small-scale commercial irrigation. Therefore, this research project will test the hydraulics of two systems, as well as provide a cost analysis using Norman, Oklahoma as the case study to determine at what size a second, nonpotable distribution system would be feasible in terms of hydraulics and costs for urban irrigation versus constructing a new water treatment plant (WTP) within 50 years.

Definitions

To assist the reader, some of the technical terms used throughout this research are defined in Appendix A.

Goals, Objectives, and Hypotheses

The overarching goal of this research project is to provide information on the hydraulics and costs of dual water distribution systems as an alternative method to traditional single distribution systems. A secondary goal is to determine a feasible scale to implement a dual distribution system to provide nonpotable water for residential and commercial irrigation, while using all available water supplies. This includes the water allocation from Lake Thunderbird, active and inactive groundwater wells, and effluent from the Norman Water Reclamation Facility (WRF). To achieve these goals, the objectives of this project are to evaluate and compare the hydraulics and costs of implementing and operating a dual water distribution system with a single distribution system. More specifically, the objectives are:

1) To create two hydraulic models of the Norman water distribution system—the existing single distribution system and a new dual distribution system;
2) To determine the costs associated with constructing and maintaining a dual distribution system, as well as building or upgrading a water treatment plant; and
3) To perform two comparative analyses:
    a. A hydraulic analysis to determine which system has more junctions with pressures within a preset range and which system can provide better fire flows; and
    b. A cost analysis to determine which system is more cost-effective.

The hypotheses for this research project are that a secondary water distribution network will be more expensive than a single distribution system to construct, but that it will be more cost effective in the long term; it will be better hydraulically than the existing system; and that it will prove to be a better system overall than the single system.

Methodology

The research for this project mainly evaluates the Norman water supply and distribution systems at a “current” and a “future date”. The industry-standard method is to establish a baseline for comparison in the “current” year and evaluate the same system 50 years in the future. The planning horizon of 50 years is typically chosen as the service life of major water infrastructure elements, such as water treatment plants. This research project was initiated in 2012 and, as a result, much of the initial data comes from 2012 and 2013. This includes the water supply data and the Norman GIS files. The final year of complete water supply data is from 2012, so 2012 will be used as the "current" year for this research project, and the "future" 50-year projection is 2062.

The city of Norman, Oklahoma was chosen as the case study for this research project for a two primary reasons:

1) Most of the studies on residential reclaimed water irrigation have either looked at small hypothetical populations or cities and towns with populations less than 100,000 people, and Norman has approximately 100,000 people as of 2010; and
2) Data was readily available for the research project and staff from the City of Norman was very willing to provide data as needed.

A brief overview of the methodology is as follows:

1) Perform the literature review to evaluate what studies have already been done and where research is still needed.
2) Determine the 2012 and 2062 populations in order to evaluate the current water demands and estimate the future water demands.
3) Evaluate the 2012 and 2062 water supply sources, including Lake Thunderbird, groundwater, OKC connection, and effluent from the Norman Water Reclamation Facility. Each source is evaluated for its current and future capacity and whether it is a potable or nonpotable source.
4) Evaluate the 2012 water demands on a system-wide basis and for categorical demands, such as residential, commercial, industrial, institutional, University of Oklahoma, and sprinkler demands. The categorical demands are determined as percentages of historical demand. These demands are also calculated for average

day demands, maximum month demands, maximum day demands, peak hour demands for a single system, peak hour demands for a potable system, and peak hour demands for a nonpotable system as applicable. The residential and commercial demands are also broken down into potable and nonpotable demands.

5) Project the 2062 water demands on a system-wide basis using the per capita demand method, which calculates a per capita demand for the service population and assumes a relatively constant per capita demand over the 50-year study period. The 2012 per capita demand is applied to the projected 2062 projected service population for a 2062 system-wide demand. The 2012 categorical demand percentages are also assumed to remain constant over the 50-year study period and used to calculate the various demand scenarios for each use category that were calculated for the 2012 water demands.
6) Allocate the categorical demands using parcel shapefiles from the city of Norman to estimate demands for each property based on average number of people per household (from the 2010 U.S. Census) and pervious area on the property. The residential baseline demand is determined on a per capita basis, which is applied to the average number of people per household. The commercial baseline demand is determined by the building footprint area. The irrigation demands for residential parcels, commercial parcels, and park parcels are calculated based on the pervious area on the parcel. The industrial, institutional, and OU demands are based on the building footprint area.
7) Create a hydraulic model of Norman's existing single water distribution system with a GIS shapefile of the 2014 distribution system. Import the water lines, fire

hydrants, water towers, and wells from GIS data. Import elevations from two-meter contour data. Load the categorical demands based on the spatial allocation determined in the previous step. Set the hydraulic grade line based on the overflow elevations for the water towers. Create scenarios for average day, maximum month, maximum day, and peak hour demands. Set up all simulations as steady-state scenarios.

8) Calibrate and verify the existing single distribution system model with fire flow tests using industry-standard software to run all the simulations.
9) Leave the potable demands on the existing potable distribution system. Create an initial secondary nonpotable distribution system based on the existing single distribution system. Load the secondary distribution system with the nonpotable demands.
10) Evaluate future growth maps from the 2025 Norman Land Use and Planning Report to choose where to expand the distribution systems for the 2062 simulations. Once the expansion areas have been identified, add new water lines to the hydraulic models of the distribution systems along section lines.
11) Perform hydraulic simulations of the single and dual distribution systems for 2012 and 2062 under average day demands, maximum month demand, maximum day demands, and peak hour demands.
12) Once the hydraulic simulations are complete, evaluate the pressures, available fire flow, and velocities for the distribution systems for 2012 and 2062 with the four demand scenarios. Perform additional hydraulic simulations as needed to better

refine the 2062 distribution system designs in terms of water line diameters and potential storage locations.

13) After the designs and hydraulic simulations are complete, estimate and compare the costs to install and maintain a dual distribution system and the costs to increase and maintain the WTP capacity for 2062 demands.

The report uses a variety of units to represent demand and supply. These include:

- Gallons per minute (gpm) – gallons per unit are most often used to represent flow rates in water lines and flow rates that would be too small to represent as gallons per day or millions of gallons per day.
- Gallons per day (gpd) – gallons per day are most often used when the demand or flow rates are between 1 gpd and 0.75 MGD.
- Millions of gallons per day (MGD) – Millions of gallons per day are most commonly used when the gallons per day is greater than 0.75 MGD.
- Gallons per capita day (gpcd) – gallons per capita day represent the number of gallons per day that each person use. This number can be particularly useful because water demands are often controlled by population, so knowing the population can be a useful basis for projected demands in terms of gallons per capita day.

When various demands or flow rates are compared, then the same units are used as much as possible to reduce any confusion and to make it easier to compare the demands or flow rates.

The hydraulic models are all created using Bentley® WaterGEMS V8*i* software (WaterGEMS), which is an industry-standard program. All shapefiles were modified and spatial analysis was performed with Esri ArcGIS software, which is another industry-standard program.

Report Format

Instead of having a section about methodology and a section with results, this research paper combines some of the preliminary methodology details and results together to make the research easier to follow, with the overall results presented separately. More specific details and the results from each of the steps above are presented in the following sections:

- Literature review
- Determine the population projections, water supplies, and water demands
- Create the hydraulic models
- Evaluate the hydraulics of the single and dual distribution systems for 2012 and 2062
- Evaluate the costs of the single and dual distribution systems for 2062
- Conclusions and future research opportunities

Chapter 2: Literature Review

Introduction

Water reuse is a very broad topic and, as such, encompasses many areas, including history, use, treatment, quality, regulations, policies, implementation, and design to name a few. Thus, a comprehensive literature review of water reuse topics is beyond the scope of this research project. To keep the literature review manageable, the following three subjects are covered:

- Brief History of Water Reuse and Regulations
- Dual Distribution Systems
- Water Demands

A Brief History of Water Reuse and Regulations

De facto water reuse has been in practice for thousands of years, while modern water reuse began in the 1900s when wastewater treatment methods were developed. One of the first water reuse examples in the United States was for industrial reuse by the Bethlehem Steel Company in 1942 (Asano et al., 2007). Urban water reuse began in the 1960s in California, Colorado, and Florida, which coincided with a large number of academic articles being published on the subject. After the 1970s, interest decreased and few dual distribution systems were implemented, possibly due either to the high costs of constructing the secondary system or to increased water resource availability in the areas most interested. Another barrier to water reuse was the lack of regulations and fear of water quality. As knowledge about water reuse and contaminants has increased, regulations have been created and updated. In most states and regions, tertiary (i.e., advanced) water treatment is required to produce higher quality wastewater prior to the reclaimed water being used.

For reclaimed water, the governing federal regulations are the Clean Water Act of 1977 and the Safe Drinking Water Act of 1974, which was updated in 2014. Beyond that, each state has its own regulations or guidelines, if it has any. California implemented the Wastewater Reclamation Criteria, more commonly known as Title 22 Regulations, in 1978 and these regulations have been the starting point for many states to write their own regulations for water reuse. Specifically in Oklahoma, the governing regulations can be found in the Oklahoma Administrative Code (OAC) Title 252 for the Oklahoma Department of Water Quality (ODEQ) Chapter 627 Water Regulations (OAC 252:627 Water Regulations). This document outlines four categories of water reuse, from Category 5 for secondary treated wastewater through Category 2 highly treated wastewater. Regulations for the fifth category, Category 1 water for potable reuse (either indirect or direct), are currently being developed by the Oklahoma State Legislature.

In addition to developing regulations for water reuse, water conservation is also being heavily promoted. On the national level, at least two executive orders (EO) have been issued to reduce water consumption, including EO 13423 and EO 13514. Both EOs mandate reduced potable and/or overall water consumption instead of requiring water reuse, however water reuse is suggested as an option for reducing potable water demand (EPA, 2012). In response to these directives to reduce water use, the U.S. Army has begun developing a water reuse program for its bases (Scholze, 2011).

On the state level, Oklahoma passed a bill in 2012 to become the first state in the nation to limit 2060 freshwater consumption to that of 2012 freshwater consumption (OWRB, 2015). While Oklahoma passed a bill, other states have water conservation initiatives, including Florida (Florida Department of Environmental Protection, 2002), Maryland (Department of Legislative Service, 2001), and Minnesota (Minnesota Department of Natural Resources, 2015). The purpose of these water conservation initiatives is to preserve limited freshwater resources for future use (Department of Legislative Service, 2001; Florida Department of Environmental Protection, 2002; Minnesota Department of Natural Resources, 2015; OWRB, 2015). All these various regulations show that the states are working to allow reuse. They had not previously provided regulations for water reuse because it happened so infrequently that states could regulate the systems on a case-by-case basis. In the past several years though, water reuse has rapidly become more common, so regulations are being written to ensure that the treatment and distribution are done safely.

Elsewhere around the world, various countries have implemented water reuse regulations. In developing countries, farmers typically use unregulated wastewater for agricultural irrigation because water is so scarce (Asano et al., 2007). More developed countries, especially those in arid regions, have provided many advances to the field of water reuse and reclamation (Asano et al., 2007). European countries abide by the European Union Water Framework Directives, while Middle Eastern countries have well-established water irrigation programs (Asano et al., 2007). Australia also has many water reuse programs around the country, with corresponding regulations. These examples show that water reuse is becoming more common around the world as other countries incorporate and develop water reuse and nonpotable water in their water supply portfolios.

While many countries around the world use reclaimed water for irrigation, toilet flushing, and indirect potable reuse, only Namibia, Singapore, and the US have direct potable reuse (DPR). Namibia has been using DPR since the late 1960s and Singapore's NEWater has supplemented Singapore's water supply since 2002 (Asano et al., 2007). In the U.S., the only DPR facilities are in Big Spring and Wichita Falls, Texas. The Big Spring facility came online in May 2013 and the Wichita Falls facility began operations in June 2014 (EPA 2016). Both these facilities were constructed in response to the heavy drought that has affected West Texas during the 2000s. The importance of these facilities show that some communities are willing to invest in alternative water supply sources. For these three locations, the communities decided that the cost of developing water reuse for potable purposes was better than the cost to find new freshwater sources.

In Oklahoma, water reuse is becoming more prevalent. In the past few years, a few municipalities have initiated reuse programs, including Fort Sill, Norman, and Deer Creek. Norman began providing reuse water in 1995 for irrigation at the University of Oklahoma (Carollo, 2014). However, the University began irrigating from groundwater wells that did not meet arsenic standards and stopped using as much reuse water from the City of Norman. By 2015, Norman and Deer Creek had both investigated supplying indirect potable reuse (Crum, 2015). Deer Creek was in the process of conducting pilot studies and needed a way to raise the nearly $131 million dollars estimated to pay for a new treatment plant (Crum, 2015). Norman had already begun to upgrade the existing water reclamation facility and was stalled in negotiations with the Midwest City and Del City to discharge the water upstream of Lake Thunderbird, a common water supply source for the three cities (Cannon, 2015; Layden, 2015).

In 2012, Fort Sill began constructing a secondary distribution system to supply reuse water for irrigation, fill stations for construction, cooling tower makeup water, and toilet flushing in a barracks (Garver, 2016). The initial project upgraded the wastewater treatment plant to provide Category 2 water and a short line for cooling tower makeup water. Subsequent projects installed an 11,000 foot reuse main across the base, connected three main irrigation areas, five cooling tower locations, two fill stations, a connection for toilet flushing, and an elevated storage tower for storage and pressure equalization (Garver, 2016). These examples show that reuse is becoming more prevalent in Oklahoma as various communities around Oklahoma are beginning to consider and implement water reuse.

Dual Distribution Systems

Existing Dual Distribution Systems

As of 2013, 25 countries and 22 states had dual distribution systems for potable and nonpotable water (Grigg et al., 2013). In the United States, only four states had more than one or two dual distribution systems in the state:

- California:143
- Florida: 77
- Arizona: 32
- Texas: 17

The majority of the dual systems in the US do not provide reclaimed water for residential use and even fewer provide reclaimed water for firefighting. The first dual distribution system was in the Grand Canyon in 1924, and the first large-scale distribution system for residential use was built in 1977 in St. Petersburg, Florida (Asano et al., 2007).

The initial impetus for many secondary distribution systems was drought and stricter discharge requirements, which necessitated costly upgrades to existing wastewater treatment plants (Gallier, 1985; Crook, 2005). Academic articles on dual distribution systems were prevalent in the 1970s, when interest was high and several communities were experiencing drought. However, the drought ended in many areas and communities did not experience such severe water shortages until the past decade when extreme drought returned to the western United States and southern plains. The recent drought caused a resurgence of interest in dual distribution systems, reclaimed water, and water conservation in general, and many communities began to consider these three things much more frequently.

Even though single-family residential water use has generally been declining since about 1995 (DeOreo and Mayer, 2012), residential irrigation can constitute up to 60 percent of the overall demand in the arid Southwest (EPA, 2014). Even on the East coast, outdoor irrigation can reduce valuable freshwater sources. In the 1970s, the city of St. Petersburg, Florida constructed a secondary water distribution system and made Florida the first state to have a large-scale secondary water distribution system (Asano et al., 2007). A few other Florida municipalities have also constructed their own reclaimed water systems for residents. These towns and cities are typically located near St. Petersburg and along the coastlines, where discharge requirements tend to be very strict. The reclaimed water systems have allowed these Florida municipalities to reduce their discharge and also their potable demand.

The St. Petersburg, Florida dual distribution system was established in 1977 because it was cheaper to build a second WDS than it was to comply with stricter discharge requirements for Tampa Bay (Crook, 2005). These days, the system provides reclaimed water to approximately one-third of its residents, or 10,280 customers (St. Petersburg, 2012). Four facilities provide up to 37 million gallons a day to customers for lawn irrigation. However, the utility requires all irrigation customers to install a sprinkler system before connecting to the nonpotable system (St. Petersburg, 2012). In 1986, the system was expanded to include more customers at a cost of $110 million. The system has benefited St. Petersburg by significantly reducing the potable demand, though the city restricts end uses to lawn irrigation and firefighting, and prohibits car washing and toilet flushing with the reclaimed water (St. Petersburg, 2012).

Since the system was established, several nearby cities in Pinellas County have also established their own dual distribution systems, including Dunedin, Largo, Clearwater, and St. Pete's Beach. The majority of them provide reclaimed water for a variety of uses, including commercial irrigation and golf courses. The initial nonpotable system was created because the groundwater supplies were being depleted. Since the first distribution system was started in St. Petersburg, the groundwater supplies have continued to decrease so that the reclaimed water is now used for aquifer recharge to prevent saltwater intrusion into the aquifers. Other residential systems in Florida are in Cape Coral, Eustis, Ocala, Orlando, Oviedo, Tampa, and Seminole. Unfortunately, there is little information available about these systems beyond the fact that they exist. However, the key point is that various communities decided that it was a wise choice to install a secondary distribution system for nonpotable uses.

On the western coast of the United States, California has been following Florida's lead in installing dual distribution systems to offset potable demands and preserve freshwater sources. California has faced severe water shortages for decades. While agriculture uses the majority of freshwater demand at 77 percent, almost half of all residential use goes toward urban irrigation (Cohem, 2009). Urban irrigation equals approximately eight percent of the overall demand in California, including residential irrigation, commercial irrigation, and large landscapes (e.g., golf courses and parks). However, this value is affected by southern California, where urban irrigation accounts for approximately 25 percent of the total water demand (Cohem, 2009). Various water conservation methods have been implemented, including constructing many dual distributions to provide reclaimed water for urban irrigation because the communities decided that it was better to use available untreated resources for nonpotable uses instead of treated the available resources to supply nonpotable uses with potable water.

California has the largest number of dual distribution systems in the country, with 177 systems throughout the state. While the majority of dual systems supply agriculture and industry, there are systems for urban irrigation in Burbank, Oakland, Irvine, Livermore, Corte Madera, Redwood City, San Diego, and Santa Barbara. Burbank and Irvine installed the first two residential systems in 1967 (Grigg et al., 2013). After them, Oakland, Corte Madera, and Redwood City constructed their own secondary systems for residential use in the 1970s. More recently, additional residential secondary distribution systems were constructed around 2000 in San Diego and Santa Barbara. Initially, Redwood City faced public opposition to the secondary distribution due to fears of health and safety, but Redwood City involved the public in the decision process to achieve a successful resolution (Ingram et al., 2006). These examples show that cities and towns in California have decided that secondary distribution systems are worth the investment in order to reduce potable demands. Additionally, even though these systems were constructed over a nearly 50 year span, California requires that all residential systems comply with Title 22 to provide high quality, tertiary treated recycled water to minimize health concerns.

After Florida and California, Arizona has a strong history of reclaimed water distribution. Tucson, Arizona constructed a dual system in 1983 for urban and agricultural irrigation uses and commercial toilet flushing (Grigg et al., 2013). As of 2010, there were 704 residences connected to the system (Campbell and Scott, 2011). A report in 2009 estimated that only 47 percent of the total effluent was utilized (Huckelberry and Letcher, 2009) and so the dual system was constructed to offset utilizing the Colorado River. The potable system serves more than 200,000 customers with approximately 4,200 miles of pipe and has a capacity of 196 MGD (Grigg et al., 2013). The reclaimed system is smaller, with only 160 miles of pipe and provides an average of 15.4 MGD of reclaimed water to users, which is only 7.8 percent of the total potable capacity. Golf courses used almost 60 percent of the reclaimed water, with residential users only accounting for approximately seven percent of the total reclaimed water use (Grigget al., 2013). Even though no costs about installing the dual system were provided, Tucson decided that the small offset of the nonpotable water was worth the investment.

These examples of water reuse in the United States show that some locations in the United States have a long history of incorporating water reuse into their water portfolios. The systems that have incorporated water reuse and continue to use reused and nonpotable water demonstrate that reuse is feasible, even if the nonpotable systems only serve a small percentage of the potable system.

Outside the United States, Australia has the greatest number of reuse systems for residential purpose with nine recycled water systems (Australia Coordinator for Recycled

Water Use, 2014). Sydney Water operates the largest recycled water system in Australia, with 12 water recycling treatment plants (Australia Coordinator for Recycled Water Use, 2014). This system delivers water to a wide variety of end-users. Rouse Hill, a suburb of Sydney, has the largest dedicated residential system in Australia. The Rouse Hill Recycled Water Treatment Plant has supplied approximately 36,000 homes with recycled water since 2001 (Syndey Water, 2009). The population of Rouse Hill is only approximately 7,500 residents, but the treatment plant provides recycled water to approximately 60,000 people in ten nearby suburbs. The treatment plant provides high quality tertiary treated water with UV and chlorine disinfection (Sydney Water, 2013). The end uses for the Rouse Hill Plant include residential and commercial irrigation, toilet flushing, car washing, and other outdoor uses (Syndey Water, 2009). The utility noticed that residents with recycled water use 40 percent less potable water, which is significant for preserving water resources and saving money. This is important for two reasons. First, the urban irrigation demand is often a relatively high component of a municipality's water demand and this demand could be reduced simply by supplying it with nonpotable water instead of other conservation attempts. Second, the secondary distribution system could be extended to supply additional customers, which might in turn reduce their nonpotable demands.

Newer residential water systems in Australia are Yarra Valley Water in Victoria, City West Water in West Werribee, South East Water, and Mawsons Lake in Adelaide (Australia Coordinator for Recycled Water Use, 2014). A notable feature of the Mawsons Lake system is that recycled water is plumbed to every home, which ensures that the system helps conserve the available fresh water resources for potable uses. Australia plans to keep building recycled water treatment plants and expanding coverage for more residents, with a new Sydney plant set to open in 2015 (Australia Coordinator for Recycled Water Use, 2014).

In all, these various examples from around the United States and Australia show that secondary distribution systems do exist for many communities, both large and small. These secondary distribution systems range from only serving one-third of the potential customers in St. Petersburg, Florida, to serving ten other areas in Rouse Hill, Australia. This range of applications demonstrates that water reuse for urban irrigation is possible and already in use.

Previous Studies of Dual Distribution Systems

While the communities profiled above decided to construct dual distribution systems that serve urban irrigation, few comparative studies have been published on dual distribution systems even though feasibility studies were likely conducted before implementing any of the existing dual distribution systems mentioned above. In private conversations, most civil engineering consultants assume that constructing a dual distribution system would be exorbitantly expensive and that it is not worth investigating the feasibility so they do not advise it for their clients. This in turn means that many communities do not consider dual distribution systems.

The places with dual distribution systems mentioned previously decided that a secondary distribution system was worth the investment, at least at the time they began construction. However, there are several locations that conducted feasibility studies that ultimately chose not to pursue a secondary distribution system. A 1985 feasibility study for Castle Rock, Colorado determined that retrofitting the town existing system for nonpotable reuse was "economically unfeasible" (Gallier, 1985). However, the study determined that future growth would eventually made the cost "an economically viable alternative" for the town to invest in because the cost was negligible to upgrade the town's wastewater treatment plant for water reuse, and the town would deplete its water sources otherwise. The plan was never enacted and Castle Rock did not build the secondary distribution system. As a result, Castle Rock was once again studying how to use its treated wastewater to supplement diminished water resources in 2013 (9News, 2013).

In California, several of the municipalities, including Sunnyvale, Modesto, and Cotati, in the area performed feasibility studies prior to constructing a secondary WDS. Near San Francisco, the city of Sunnyvale already has an existing secondary WDS that was constructed in the early 1990s and a feasibility study was proposed prior to expanding the system (HydroScience, 2013). As of 2013, there were 120 reuse water connections for Sunnyvale and the study examined expanding the secondary WDS by an additional 181 connections serving non-residential customers (HydroScience, 2013). Near Sunnyvale, the city of Modesto initiated a feasibility study for what to do with the reclaimed water from the wastewater treatment plant (Modesto, 2011). The project established several alternatives for reclaimed water alignments to augment the existing system and chose one that is currently under construction (North Valley Regional Recycled Water Program, 2017). This new water line will serve five water systems in central California, though the main end use will be agriculture instead of residential irrigation (North Valley Regional Recycled Water Program, 2017).

Elsewhere around the United States, various municipalities and counties have conducted feasibility studies as well for water reuse systems, but there are no records that these communities have installed water reuse systems. These locations include Loudoun County, Virginia; Scottsdale, Arizona; Fairfield Village, Texas; and King County, Washington. The lack of action indicates that these communities decided that water reuse was not the best decision for their community, whether it was due to cost, public opposition, or another unknown factor.

There has also been little academic research conducted about comparing single and dual distribution systems. Kang and Lansey published the most notable and recent comparative study of a hypothetical and a real dual distribution system against a single distribution system (2012). Their approach used a triple-bottom line evaluation of economics, system reliability, and greenhouse gases. The end results of the study showed that fire flow is the major determinant for sizing the water lines, and that system reliability greatly increased costs.

The bottom line for the feasibility studies is that they are not very common. While a few locations listed above conducted them, there are many more locations that could benefit from incorporating nonpotable water into their water supply portfolios that have not conducted feasibility studies or had engineering reports prepared for them. This lack of reports demonstrates the need and opportunity for reports.

Implementing a Dual Distribution System

In order to install a dual distribution system, various aspects must be evaluated, including design criteria, costs, and extent of the dual distribution system. The below subsections discuss the available literature for these three aspects.

Design Criteria

The recommended design criteria for dual distribution systems are similar to those for single distribution systems (AWWA, 2009). The design criteria can be divided into three main areas:

- Pressure design criteria
- Piping system design criteria
- Storage design criteria

The first area – pressure design criteria – varies a little between the potable distribution system and the nonpotable distribution system. Potable distribution systems have three recommended design pressures and a set range of pressure fluctuation: maximum pressure (100 psi), minimum pressure (35 psi), and minimum pressure during fire flow (20 psi) (AWWA, 2012). Nonpotable distribution systems, assuming that they do not carry fire flow, must be more sensitive to the end users and end purposes. Minimum pressures can be between 10 psi and possibly up to 100 psi, depending on the end user (AWWA, 2009). For example, residential sprinklers, typically operate between 15 and 60 psi, with an optimal pressure of approximately 30 psi (Rain Bird Corporation, 2017).

The second area of piping system design criteria is very similar for both potable and nonpotable distribution systems. Potable systems should have velocities under four to 6 fps during average day demands and minimize head losses (AWWA, 2012). Nonpotable systems must maintain a minimum velocity to avoid any settlement for solids that may be in the water, depending on the nonpotable water quality (AWWA, 2009). Both systems must choose the pipe material so to minimize any reactions between the pipe material, the water quality, and the surrounding soils (AWWA 2009, AWWA 2012).

Pipe size is another piping system design criterion to consider for both potable and nonpotable distribution systems because the pipe size greatly affects both velocity and head loss, and subsequently pressure. For potable distribution systems,

> *"the size of the pipe is determined by the maximum flow rate carried, typically either the maximum day plus fire flow condition or the maximum hour condition for distribution mains. Either the maximum hour flow rate or the maximum storage replenishment rate is typically the maximum flow for the large transmission mains. The pipe size is selected to ensure that velocity and head loss limits previously described are not exceeded. (AWWA, 2012)*

Nonpotable distribution systems must also consider the design flows, velocity, and head loss when sizing pipes, though nonpotable systems have different design flows than potable systems do. For example, the typical peak hour factor for single and potable distribution systems is 3.63 (Chin, 2013), while it is 5.00 for nonpotable distribution systems (Asano et al., 2007).

The third area, storage design criteria, varies the most between potable and nonpotable distribution systems. Potable distribution systems provide storage for "three primary components: equalization storage, fire storage, and emergency storage" (AWWA, 2012). The equalization storage provides storage for peak demands, because the water supply is typically sized for maximum day demands. The fire storage is similar in that it provides the storage for any fire flows that occur so that the water supply source does not have a sudden, large demand on it. Finally, emergency storage provides a measure of safety for communities in case something happens to the water supply source so that the community still has water available. In many cases, nonpotable systems do not need to account for fire storage or emergency storage (in most cases). Instead, seasonal storage for nonpotable distribution systems helps provide a balance between the constant wastewater flow throughout the year and the seasonal irrigation demands (AWWA, 2009). Since wastewater effluent occurs year-round and the majority of the irrigation/nonpotable demand occurs during only a few months of the year, seasonal storage allows “surplus effluent, produced during periods of low demand, can be impounded for subsequent withdrawal during periods of high demand” (AWWA, 2009).

Costs

Dual distribution systems have often been constructed to preserve freshwater sources, even if it was not the most economically sound decision. In some cases, the potential economic cost of constructing the secondary system is a major barrier to providing reclaimed water (King County Wastewater Treatment Division, 2008). Even still, several systems have realized economic benefits to reclaiming water. While it is initially more expensive to construct a second system, preliminary studies show that it can be cheaper than expanding the potable system (Grigg et al., 2013).

While the economic incentive does not usually appear to be obvious at first due to the relatively high implementation cost, the financial advantages include “extending scarce water supplies, reducing wastewater discharges, and deferring or avoiding capital investments in treatment facilities” (Grigg et al., 2013). Dual distribution systems can provide a way to use an existing water source: wastewater treatment effluent. This can sometimes help a system avoid obtaining additional water rights, which can be costly. The benefit to reducing wastewater discharges is that some systems can avoid environmental quality violations, depending on the wastewater quality effluent and the receiving stream quality. In some cases, cities like Cary, NC have achieved zero discharge to receiving streams (Grigg et al., 2013). Other cities pride themselves on reducing pollution in the receiving streams (van Leeuwen et al., 2003).

The cost savings from implementing a dual system range from no benefit to the equivalent of constructing a modern large seawater desalination plant (Cohem, 2009). However, the savings must be scaled based on the percentage of participating population. When implementing a water reuse program, the initial costs typically include upgrading the wastewater treatment plant and installing the distribution system. Because many municipalities often must treat their wastewater for discharge requirements, this cost would be incurred whether or not there were a reuse program. As a result, the cost for the reuse distribution system "can be the most significant component of costs for nonpotable reuse systems" (National Research Council of the National Academies, 2012).

Size

One potential limitation to designing a secondary distribution system is the extent of the service area. In St. Petersburg, Florida, the secondary system only supplies reclaimed water to one-third of the residents because the wastewater from three households only provides enough reclaimed water for one lawn (St. Petersburg, 2012). The sizes of other systems range from only about 300 connections in Sunnyvale, California to over 60,000 users near Rouse Hill, Australia. The main factors between the systems is the available nonpotable water supply and the willingness of the public to use nonpotable water.

Water Demands

Water demands control the entire distribution system. The volume of water consumed and where it is consumed must be known to realistically model the distribution system, understand it, address any deficiencies in the system, and plan for the future. The volume of water is typically obtained from or estimated based on historical water meter and water supply records. The location of water demands can be determined from water meter records or approximated using property parcel-level shapefiles. The most accurate results come from the most specific data.

Estimating Current Water Demands

It is standard to determine the current water demands in order to estimate future water demands because the current water demands often are the basis for estimating the future water demands. In some cases, detailed information from monthly water meter data is available that can provide specific water demands and a spatial representation of where those demands occur. However, much of the time, only system-wide demand records are available, such as total water supplied by the water treatment plant or flow from groundwater wells. This information may be reasonably appropriate for less detailed levels of master-planning, but spatial demand information is required for more detailed studies.

The domestic water demands (i.e., water supplied from municipal water sources) must be broken into end-uses in order to better determine the demands. These end uses are often broadly categorized as residential, commercial, institutional, and industrial. The categories can then be sub-divided, much like zones in cities. Once the categorical demands are determined, they are then spatially located in the distribution system. A geographic information system (GIS) software can make it easier to spatially locate and analyze the demands with regards to other contributing factors.

Categorical Demands

To more accurately represent the various demands in a hydraulic model, the total demand must be divided into various categorical demands. Every community has its own specific water demand categories and the four most common categories are residential, commercial, institutional/municipal, and industrial (Chin, 2013). Once the various categories of water demand have been identified, they must be appropriately applied to the hydraulic model to achieve realistic results. This is now often done by integrating the GIS data with the hydraulic model data. Modeling dual water distribution systems adds another layer of complexity because it is necessary to subdivide the categorical demands into potable and nonpotable components. Typically, potable demands include water used for drinking, washing, faucets, food preparation, and showering, and nonpotable demands include water used for urban irrigation, agricultural irrigation, toilet and urinal flushing,[1] and car washing. Fire flow demand can be placed on the nonpotable system, though only five of 37 surveyed cities provided nonpotable water for firefighting (Grigg et al., 2013). This shows that fire flow is not a common use for nonpotable distribution systems, likely because fire flow controls the design for a distribution system, potable water lines are already sized for fire flow, fire flow must be provided for the entire extent of the distribution system, and nonpotable systems may not be used year-round.

Predicting Future Demands

It is challenging to predict future water use because a wide variety of factors must be taken into account, such as population growth, advances in technology, and climate change (AWWA, 2012). In most cases, historical water use and population growth

[1] Toilet and urinal flushing can be supplied with nonpotable water because little skin-to-water contact is anticipated, but the main challenge is retrofitting existing structures for this end use. Thus, using nonpotable water for toilet and urinal flushing is recommended only in new construction.

projects are combined to predict future water demands. The historical water use can either come from water production data (i.e., water supplied) or water demanded (i.e., water consumed). The difference between these two sources is that water supplied accounts for water loss and non-metered water users, while water demanded reflects the amount of water that the consumers use.

Without specific water use data, Chin recommends using a percentage of the overall demand to estimate the various components of water demand, including residential, commercial, public, industrial, institutional, and loss demands (Chin, 1999). In Florida, studies used detailed parcel-level data to estimate water demands for each parcel (Morales et al., 2013; Friedmanet al., 2013). For residential parcels, Morales et al. used parcel-level GIS data to estimate the number of people and bathrooms per residence to estimate details about how much water each toilet used and how many times a day each toilet was flushed, while commercial demands were estimated by the square footage of heated space per building (2013). Another study used census and parcel data in Portland, Oregon to study variations in single-family residential water consumption (Chang et al., 2010). Out of nine parameters evaluated, the study found that building size had the strongest correlation with water consumption, followed by building density, and then income (Grigg et al., 2013).

Mapping Demands

Water demands and trends are best represented by specific information about the end uses, so the total water demands can be paired with GIS information to estimate or extract valuable water information about a system. In many studies that modeled distribution systems, a property parcel-level shapefile was used to estimate how much water each property used (Xie, 2009; Changet al., 2010; Lowry Jr. et al., 2011; Al-Kofahi et al., 2012; Panagolpoulos et al., 2012; Friedman et al., 2013; Morales et al., 2013; Friedman et al., 2014). Some of the studies also incorporated land coverage raster or aerial maps, however these studies had to perform data processing before they could categorize the land uses (Xie, 2009; Lowry Jr.et al., 2011; Panagolpoulos et al., 2012; Al-Kofahi et al., 2012). The aerial maps could provide very detailed results, but the processing was often complex.

The GIS software allows a variety of data sets to be combined in one place to analyze the information better and to see how it all corresponded with each other. Several studies reviewed many parameters to see how they corresponded with water consumption, including:

- Property information: building size, building age, number of rooms, building height, and zoning (Chang et al., 2010)
- Neighborhood/Census information: building density (Chang et al., 2010)
- Socio-economic information: population, income, level of college education, household age, and household size (Chang et al., 2010)
- Transportation information: road network distance and distance from city center (Friedman et al., 2013; Friedman et al., 2014)
- Land cover (Lowry et al., 2007; Al-Kofahi et al., 2012)
- Existing water supply (Friedman et al., 2013; Friedman et al., 2014)

Of these various factors, building size and income had the highest positive correlation with water consumption while building density had the highest negative correlation with water consumption (Chang et al., 2010). The researchers found that building size and income could be used almost interchangeably and that either one would be a suitable indication of the water use at a particular residential property. This is important because these, both building size and income, can be obtained relatively easily from either parcel shapefiles, county commissioner records, or census shapefiles.

The parcel-level shapefiles and geocoded meter information can help determine the water demands for a distribution system and where they occur in the system. Geocoded meters are the best resource because they can provide historical information on exactly where the demand occurs. However, this information is not readily available, so parcel-level shapefiles can be used to locate the demands, using a geographic information system (GIS) to help map demands for a distribution system (Friedmanet al., 2013; Morales et al., 2013; Friedman et al., 2014). It is common enough that Bentley included a method for incorporating GIS to spatially allocate demands and load a hydraulic model (Bentley, 2013).

GIS systems are used for a wide variety of water resources-related spatial analysis, including drinking water management strategy (Singh et al., 2010), site suitability (Jain and Subbaiah, 2007; Rangzan et al., 2008; Anane et al., 2012), storm water management (Inamdar et al., 2013), urban planning (Sipes, 2005), expanding distribution systems (Suarez-Vega et al., 2011; Sitzenfrei et al., 2013), and water conservation measures (Xie, 2009; Lowry Jr.et al., 2011; Al-Kofahi et al., 2012). Most usefully, GIS has been used to map demands (Chang et al., 2010; Panagolpoulos et al., 2012). The demands were estimated and mapped by combining several layers, including census population data, roads, parcels, and buildings. The studies found that the various attributes of each parcel corresponded with water demands and that the predicted demands closely correlated with the metered demands when the methods were verified (Chang et al., 2010; Panagolpoulos et al., 2012). Thus, this method of using parcel-level shapefiles to spatially locate demands in a distribution system has successfully been used multiple times and is a valid method for spatially locating demands.

Potable and Nonpotable Demands

Potable Demand

The potable system in a dual water distribution system receives the potable water demand for the service area. At minimum, this includes drinking and cooking water, though it also typically encompasses any water that has the potential for human contact. In many cases, the potable system is often the residential demand and any relevant commercial and industrial demand. For most cities with dual distribution systems, the fire flow is also placed on the potable system. Sizing the potable system for only the domestic flows preserves the water quality in the potable system due to less required storage and shorter residence times (which could potentially limit disinfection byproduct formation) (Asano et al., 2007).

Residential indoor water use tends to be constant throughout the year and is not as susceptible to weather variations as irrigation; however, overall indoor water use has been declining since at least 1999 from approximately 187 gpd per household to 167 gpd per household in California and 132 gpd per household in other parts of the United States (DeOreo and Mayer, 2012). They hypothesize that this is likely due to more efficient fixtures, such as showerheads and toilets, and appliances, such as washing machines and dishwashers (DeOreo and Mayer, 2012). This has helped and will continue to help reduce the per capita demand, which will in turn help limit the overall potable demand even as it increases with the population increase.

Nonpotable Demand

The nonpotable system receives the remainder of the demand that is not placed on the potable system. This typically means the non-residential irrigation demand and possible industrial demands, such as cooling. The irrigation demand can be estimated either as a set percentage of the overall demand per parcel or based on the approximated demand as a function of irrigable area and the evapotranspiration rate. In many cases, the irrigation demand is more appropriately determined as a function of irrigable area. One study found that a house's size is a decent proxy for the amount of irrigation (Chang, Parandvash and Shandas 2010). Thus, it is advisable to use the amount of irrigable area to estimate varying irrigation demands during the year, instead of assuming a set percentage of the overall demand.

The irrigation use can also vary by end user. For example, in arid southern California, residential use is over half of the system-water demand, with single-family residences using about 38 percent of their water demand compared to the multifamily residences' 20 percent water demand for irrigation (Cohem, 2009). This is important because different end-users, such as single-family homes and apartments, may use different amounts of water even though they are both residences.

Single Distribution Systems versus Dual Distribution Systems

For single distribution systems, all the demand is placed onto one system without dividing it into potable and nonpotable demands. As stated previously, the demand can be divided between categories, such as residential, commercial, public, industrial, and loss. However, all the demands are still on one system and comprise the total flow. Placing all the demands on one system simplifies sizing the distribution lines because the demands do not need to be divided into potable and nonpotable demands. However, one system is not as reliable as two systems are, because there would only be one water supply for the potable system. Two systems potentially provide redundancy for nonpotable demands because potable water can be used for nonpotable demands, though no redundancy for the potable demands (Kang and Lansey, 2012).

Summary

The available literature for water reuse, dual distribution systems, and water demands show that there is not much information available for these topics – particularly for dual distribution systems. Most of the information available for dual distribution systems either pertains to using nonpotable water for industrial or municipal use or is from an engineering report, instead of academic research. This demonstrates that there is a lack of academic research available on dual distribution systems for urban irrigation or urban uses, particularly for mid-size distribution systems serving approximately 100,000 people. So, this research attempts to provide a specific case study examining urban irrigation for a mid-size distribution system.

Chapter 3: Population, Water Supply, and Water Demands

Introduction

Chapter 3 evaluates the current and future populations, the water supply sources, and the water demands. The "current" population and water demands are calculated for 2012 because this is the most recent year with data for the water supply and water demands. The "future" population and water demands are calculated for 2062, which is 50 years after 2012.

First, the historical population is analyzed in order to extrapolate the future 2062 population, which is primarily used to project future water demands. Then, the available water supplies and their respective capacities are evaluated. For this research project, water supply has two definitions:

1) The water supply source (e.g., Lake Thunderbird and groundwater wells)
2) The volume of water supplied to meet water demands (e.g., the volume of water treated at the water treatment plant)

The main potable water supplies for the City of Norman are Lake Thunderbird, groundwater wells, and a connection to Oklahoma City. The main nonpotable water supplies for the City of Norman are groundwater wells and effluent from the Norman WRF. The water sources are important because they must supply the water demands and, if there is not enough capacity, then more sources must be found.

Finally, the water demands are analyzed on a system-wide basis and then divided into six use categories: residential, commercial, industrial, institutional, University of Oklahoma, and sprinkler demands. The demands for each category are calculated as percentages of the entire system for various demand scenarios:

- Baseline Demand
- Average Day Demand (ADD)
- Maximum Month Demand (MMD)
- Maximum Day Demand (MDD)
- Peak Hour Demand (PHD)

Each of these demand scenarios is then divided into the potable and nonpotable components. Both the potable and nonpotable demands are currently on the single existing system, and in simulations later in this research, they will be divided for potable and nonpotable distribution systems.

Population

Water demands are often calculated based on population, so it is imperative to know the current and future populations in order to estimate current and future water demands. Because the Census only records the population every 10 years, the 2012 population must be projected as well. Thus, the 2012 and 2062 populations are projected based on historical population data and validated with other population projects, primarily the 2060 Strategic Water Supply Plan (Carollo, 2014). The service populations for 2012 and 2062 are also estimated because the Norman distribution system does not service the entire population.

2012 Population

2012 Total Population

Population values from the 1960, 1970, 1980, 1990, 2000, and 2010 census were obtained for the city of Norman (U.S. Census Bureau, 2013). These values were analyzed to determine population growth over the 50-year span of available data. The estimated

2012 population was also obtained from the US. Census Bureau (U.S. Census Bureau, 2013). These values are available in the figure below.

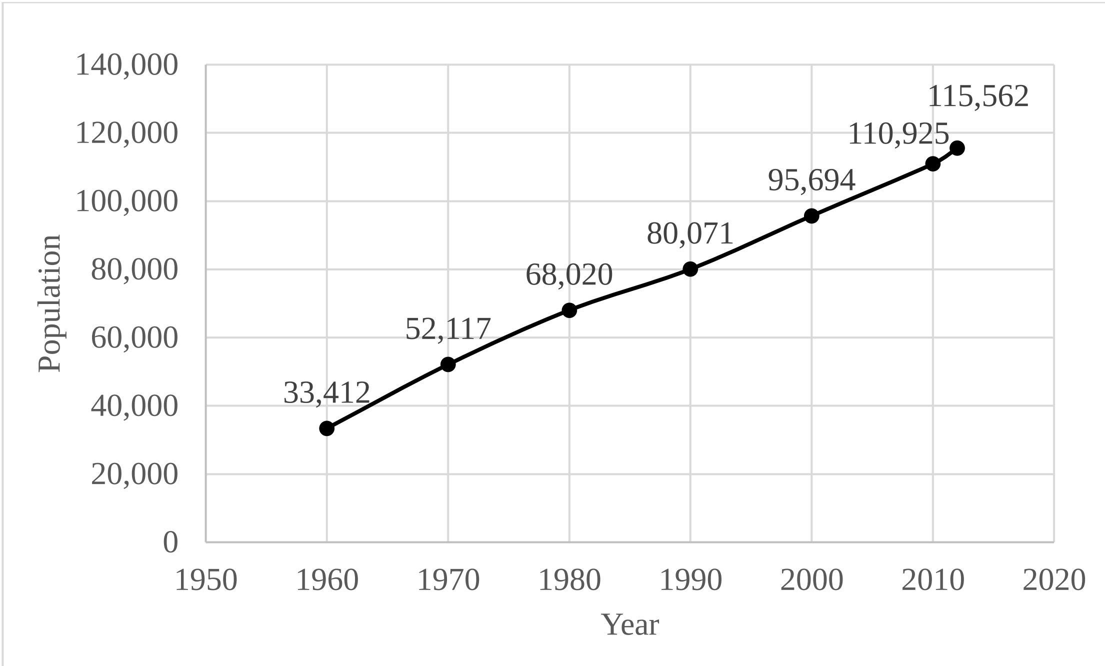

Figure 1: Past Populations of the City of Norman and 2012 Estimated Population (U.S. Census Bureau 2013)

As the chart shows, Norman has experienced an average constant growth of approximately 2.8 percent each year since 1960. As of 2012, the U.S. Census Bureau estimated that Norman's total population was 115,562 (2013).

2012 Service Population

According to the 2040 Master Plan, the service population is approximately 88 percent of the total population (CDM, 2001). This number was validated by analyzing the census population that lives in the water distribution service area. There are 29 census tracts for the city of Norman, with seven only partially within the city limits. The densest part of the Norman population is concentrated near downtown and the University of Oklahoma. The two largest census tracts, situated on the far eastern side of Norman, are partially covered by Lake Thunderbird and have very low population densities. The figure below shows the 29 census tracts, which have been clipped to the city limits.

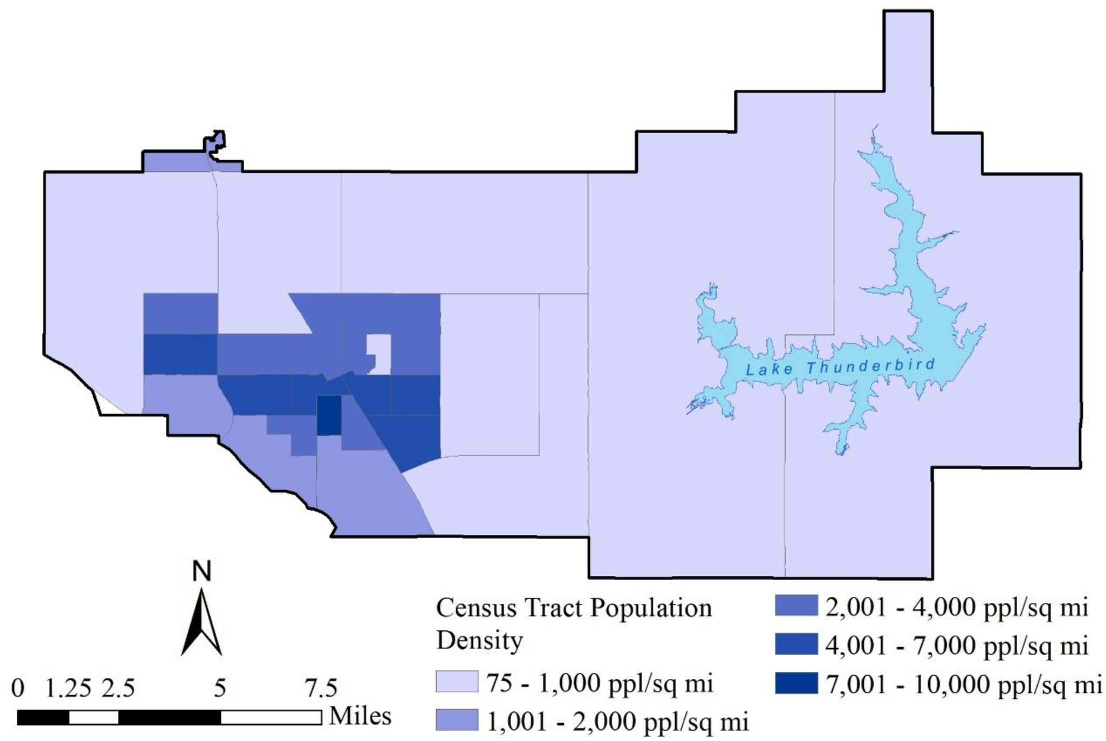

Figure 2: Map of Norman City Limits and Census Tracts by Population

The more rural parts of Norman are outside the water distribution service area, as shown in the map below.

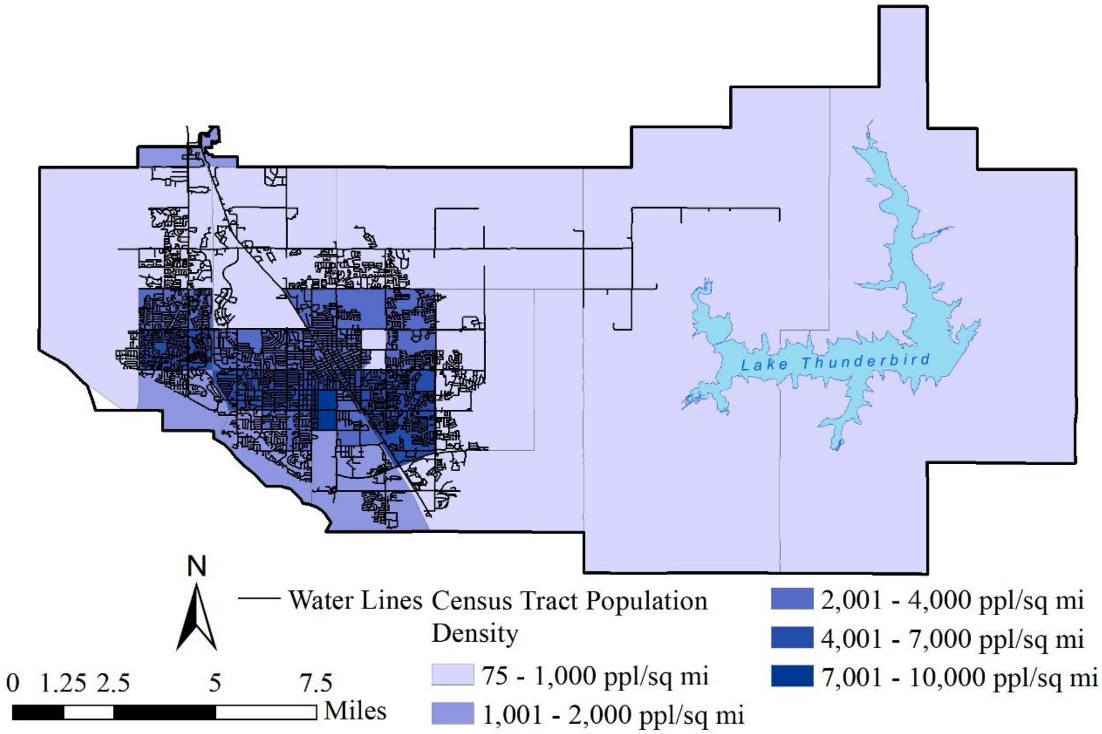

Figure 3: Map of Norman Showing Census Tracts by Population and Water Main Coverage

The water distribution service area, as shown above, only covers approximately 73 square miles while the entire City of Norman covers 189 square miles. As of 2010, the entire population inside the city limits, according to the census tract shapefile is 109,684 people. However, the 2010 population that lived in the water distribution service area is a little less than 100,000 people (accounting for parts of the census tracts that are only partially served by the water distribution system). This equates to approximately 88 percent of the population and validates the 88 percent service population from the CDM study.

Based on these calculations and the 2040 Master Plan, the 2012 service population is 101,695 people.

2062 Population

2062 Total Population

The chart below shows the population projections through 2065 for the City of Norman, using U.S. Census populations, Oklahoma Department of Commerce population projections, and projection information from the Norman 2025 Land Demand Analysis (Clarion Associates Team, 2004). A linear population growth was assumed for each year based on historical data and the collected population projections. This growth rate is validated by the growth rate determined for the 2060 Strategic Water Supply Plan for Norman (Carollo, 2014). Figure 4 shows the past and projected total populations.

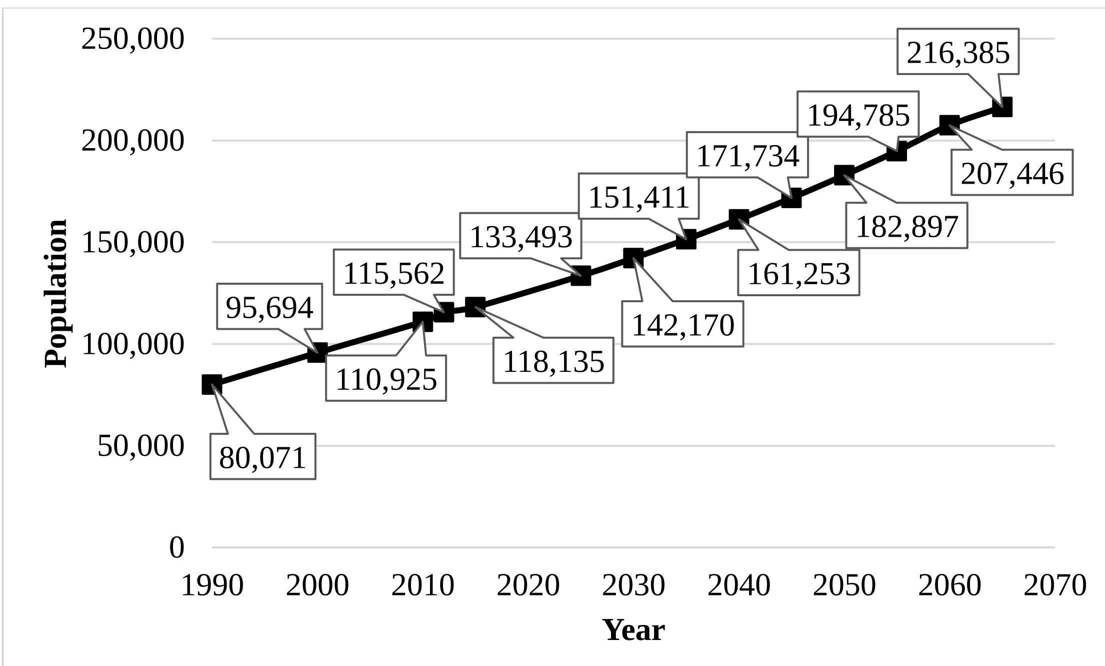

Figure 4: Past and Projected Populations for the City of Norman

The chart shows a steady increase, which reflects historical data and does not account for changing economic factors over time. However, the projected values will be used for this project, assuming constant growth that is consistent with previous growth.

The small increase shown on the chart is the 2012 estimated population from the U.S. Census Bureau (U.S. Census Bureau 2011).

2062 Service Population

Assuming that the service population remains at approximately 88 percent of the total population, the service population values for the projected years are provided in the table below.

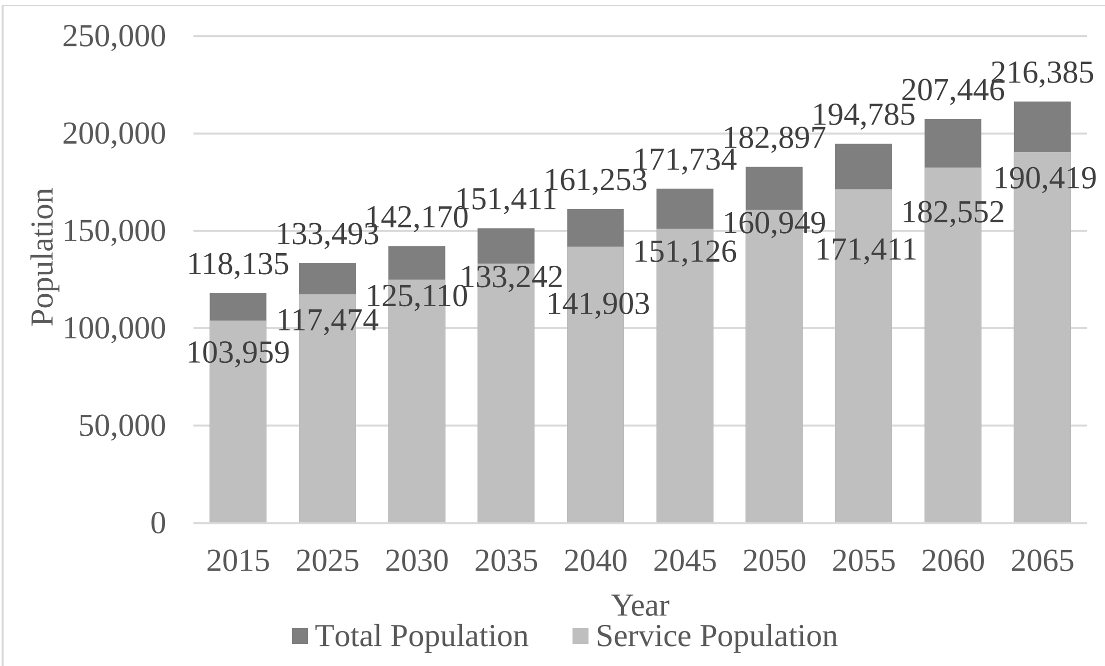

Figure 5: Estimated Total and Service Populations from 2015 through 2065

These values assume that the total population will continue to grow outside the water distribution system coverage age in proportion with the expansion of the water distribution system coverage, and that some of the future population will in-fill within Norman. The 2065 service population of 190,419 is also a conservative estimate when compared with the 2060 Strategic Water Supply Plan, which estimated a range of future service populations from 85 percent to 100 percent of the total population. For the remainder of this research, the projected total and service populations presented in Figure

5 will be used for water supply and demands, and to estimate the spatial population distribution.

Population Summary

The City of Norman is rapidly growing is expected to nearly double in population between 2012 and 2062. It is assumed that the water distribution service area will continue to grow in proportion with the population and that the service population will remain approximately 88 percent of the total population. The numbers used for subsequent calculations are summarized in the table below.

Table 1. Population Summary

Year	Total Population	Service Population
2012	115,562	101,695
2062	212,840	187,299

Water Supply

Because one purpose of this project is to determine a feasible scale for implementing a dual distribution system while using available potable and nonpotable water supplies, the available supply must be analyzed. Currently, the City of Norman receives water from three primary sources: Lake Thunderbird, groundwater, and a supplemental connection from Oklahoma City. Nonpotable water could also be obtained from the Norman Water Reclamation Facility (WRF).

The Norman Utilities Authority (NUA) provided water supply and demand data for the City of Norman, with records from January 1, 2000 through September 30, 2013. The supply data included the monthly plant production (which treats water from Lake Thunderbird, groundwater well production, and Oklahoma City supply. Norman WRF daily effluent flows were also obtained, from January 1, 2004 through December 31, 2012. These sources are evaluated in more detail below.

Lake Thunderbird and the Water Treatment Plant

Lake Thunderbird, in Norman, Oklahoma, supplies most of the water for Norman. The U.S. Army Corps of Engineers owns and operates the lake while the Central Oklahoma Master Conservancy District (COMCD) controls the water distribution. The COMCD has 21,600 acre-feet/year in surface water rights to Lake Thunderbird (OWRB 2017), which it divides between Midwest City, Del City, and Norman (COMCD 2013). Out of the COMCD surface water rights, the City of Norman get 43.8 percent, which equals a total of 9,461 acre-feet/year in surface water rights (Carollo 2014). Norman's total water rights are an average of approximately 8.45 MGD, but the City of Norman can withdraw more than 8.45 MG per day as long as the total withdrawal in a calendar year is not more than 9,461 acre-feet.

Norman has regularly exceeded its withdrawal allocation (Carollo, 2014), including every year between 2001 and 2012. During this time, Norman withdrew a yearly average of 11,132 acre-feet, or an average of 9.9 MGD, from Lake Thunderbird. The figure below shows the annual withdrawal from Lake Thunderbird compared to Norman's annual allocation.

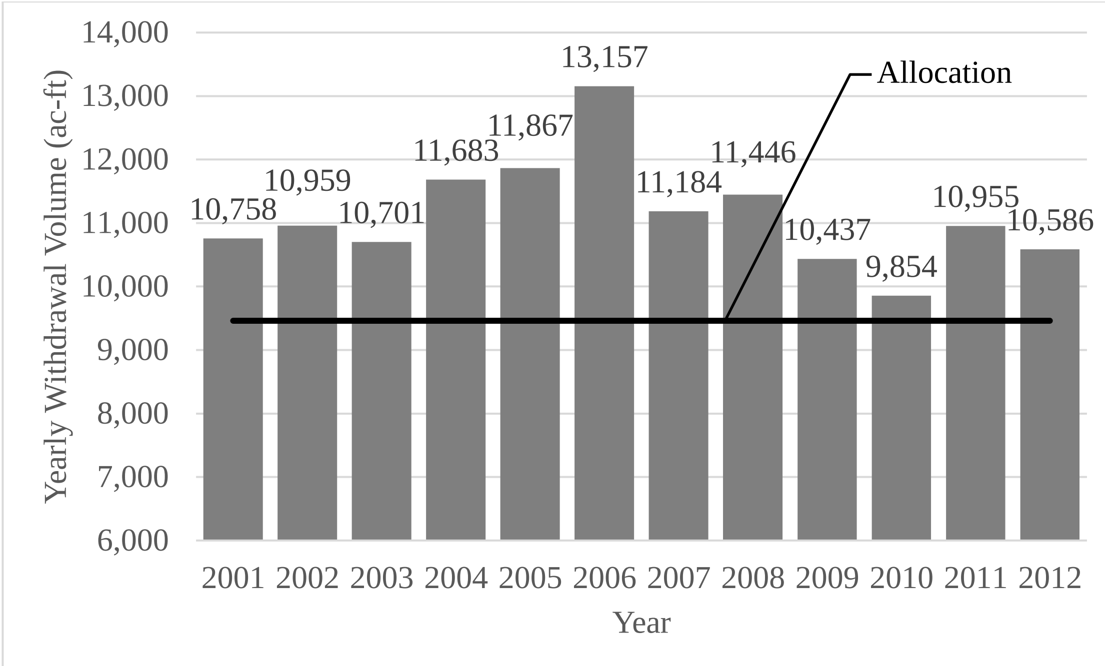

Figure 6: Norman's Annual Withdrawal from Lake Thunderbird

All the water withdrawn from Lake Thunderbird for the City of Norman is treated at the Vernon Campbell Water Treatment Plant (WTP). The WTP is currently designed to treat up to 17.0 MGD.

Groundwater Wells

The City of Norman also uses groundwater from the Garber-Wellington Aquifer to supply the water demand. The City of Norman has a total of 38,045 acre-feet in groundwater rights to withdraw water from the Garber-Wellington Aquifer, which is divided between prior right, regular, and irrigation permits, shown in the table below.

Table 2. Summary of Norman Groundwater Rights

Permit Type	Allocation (acre-feet/year)
Prior Rights	8,131
Regular	29,747
Irrigation	167
Total	38,045

To withdraw the water, Norman has 48 groundwater wells – 31 active wells and 15 inactive wells. The inactive wells were taken off-line on January 23, 2006 because they had elevated levels of arsenic, which were higher than the revised EPA arsenic rule total maximum contaminant level (EPA, 2012). The active wells are mainly located on the northeast part of the distribution system and the inactive wells are located on the northern side and north-central area of the distribution system, as shown in Figure 7.

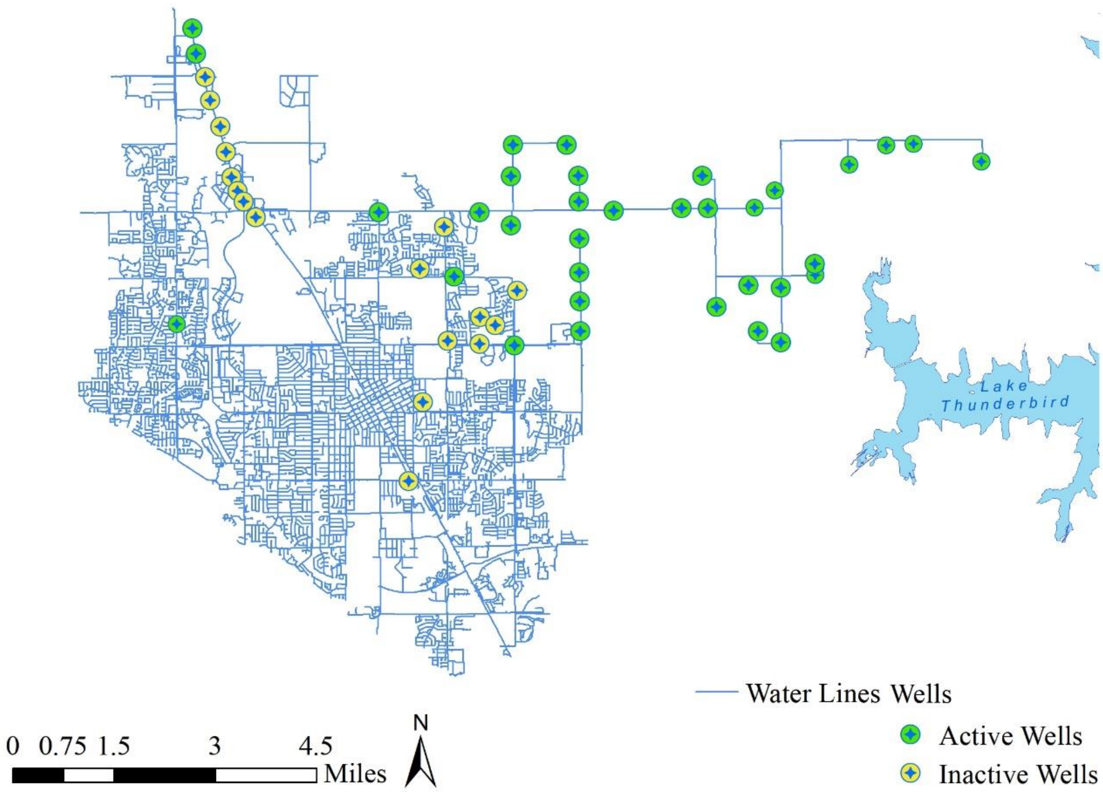

Figure 7. Well Locations

Since July 2000, the wells have provided a relatively constant supply, which is likely because the City of Norman received additional groundwater rights permits in 2005 and 2007 for a total of 7,938 acre-feet/year to help replace the capacity for the 15 wells that were taken offline after January 2006. Norman constructed additional wells on the dedicated lands for the 2005 and 2007 permits. The new wells allowed the well supply to remain relatively constant and even increase over time, as shown in Figure 8.

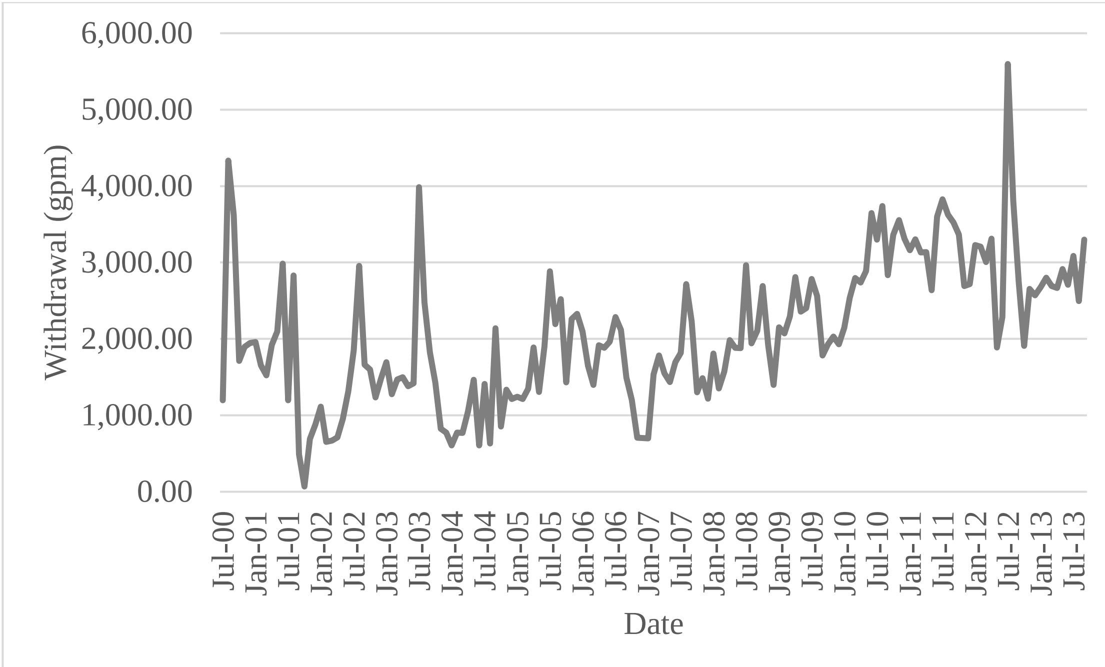

Figure 8: Historical Monthly Groundwater Withdrawal Compared to Well Capacity

While the groundwater rate has only slightly increased over the years, the wells have provided an increasing percent of the total demand for Norman users. On average from 2001 to 2013, groundwater contributed 24 percent of the total water supply, with a maximum of 53 percent in February 2012, as shown in the figure below.

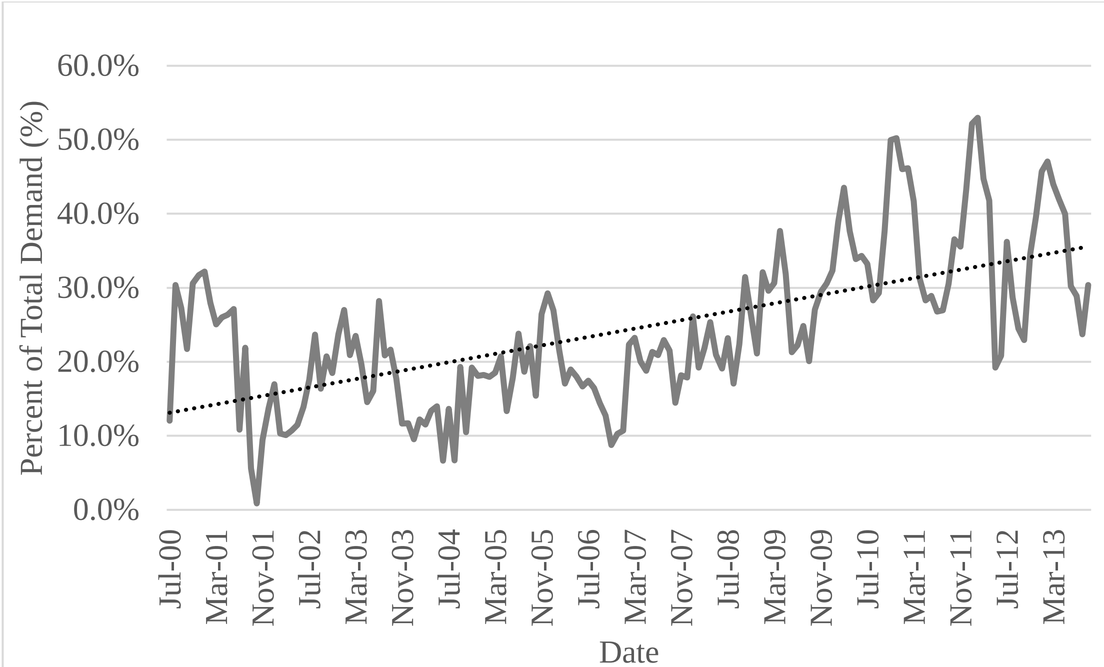

Figure 9: Groundwater Supply as Percent of Total Water Demand

The trend line shows that the percent of total demand has increased over time, even as the withdrawal rate remains steady. It is important to note that Norman may not be able to rely on the existing wells to continue supplying the same volume of water as in the past because the Oklahoma Water Resources Board (OWRB) is currently reevaluating the allowable withdrawal per acre for the Garber-Wellington. As of 2017, the Garber-Wellington has a temporary permitted withdrawal rate of 2.0 acre-feet/year/acre, but this value may decrease when the OWRB issues a permanent permitted withdrawal rate (Anthony Mackey, March 13, 2017, e-mail to author). Among industry experts, this value is expected to decrease, though the final value is unknown. For this research and the purposes of water supply, the total capacity of the groundwater wells will remain at 2013 levels, as it is assumed that Norman will acquire more dedicated lands if necessary to continue withdrawing water from the Garber-Wellington Aquifer.

Active Wells

The 31 current active wells provide an average flow of approximately 175 gpm, ranging from 82 gpm to 325 gpm (Carollo, 2014). The wells have an estimated firm yield of 6.0 MGD (Carollo, 2014). These active wells provide potable water and are connected to the potable distribution system.

Inactive Wells

Norman has 15 inactive wells in addition to the 31 active wells. Effective January 23, 2006, the EPA enacted a new arsenic rule that lowered the standard from 50 parts per billion (ppb) to 10 ppb (EPA, 2012). The 15 wells in the Norman well-field produced water with arsenic content above 10 ppb and had to be taken off-line due to the high cost to treat arsenic (Carollo, 2014). The production capacity for each well, previous to its inactivation, was provided by the City of Norman Utilities Department, with an average withdrawal rate of approximately 163 gpm, ranging from 112 gpm to 249 gpm. It is estimated that the inactive wells have a firm yield capacity of 2.1 MGD (Carollo, 2014).

If these wells were treated, they could contribute to the potable supply; however, they can supply nonpotable demands without additional treatment to reduce the arsenic levels in the water.

Oklahoma City Connection

Oklahoma City has provided a necessary water source for Norman to meet its peak water demands. Between July 2000 and September 2013, Oklahoma City supplied Norman with an average of 1.5 percent of the total demand. This value is a bit misleading though because there were several months when Oklahoma City supplied between 5 and 16 percent. In actual supply volumes, between 2001 and 2012, Oklahoma City provided a yearly average of 72.6 MG, with peak volumes in 2006 and 2011 during the extreme droughts, as shown in Figure 10.

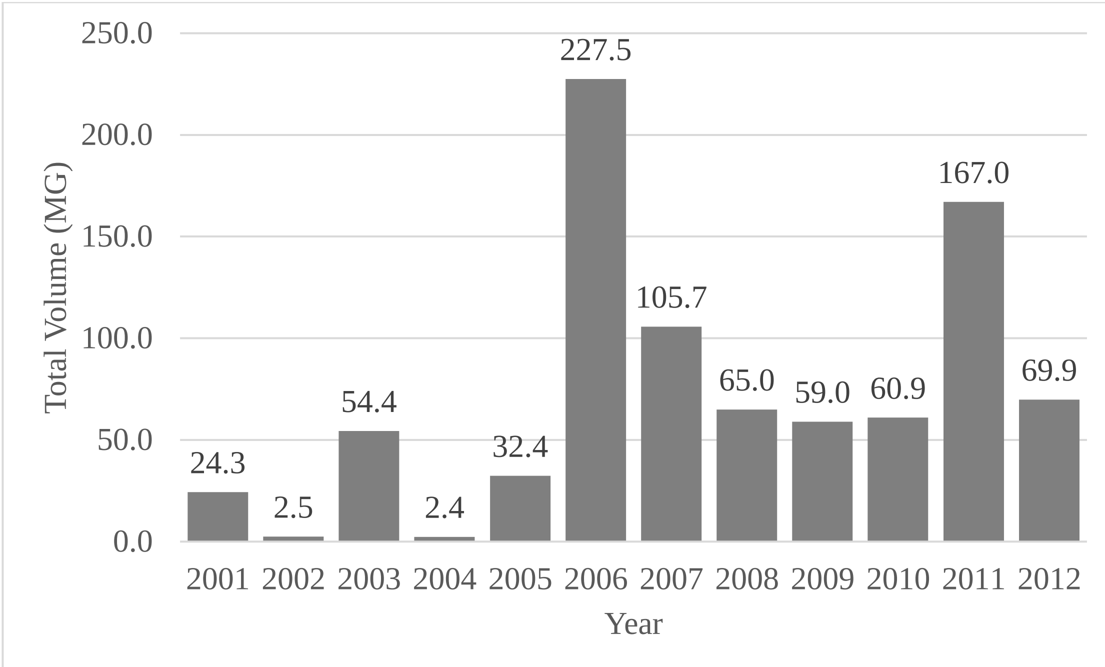

Figure 10: Yearly Volume of Water Supplied by Oklahoma City, 2001 – 2012

This water supply is supplemental and Norman would like to be independent of it, if possible (Carollo, 2014). Thus, this water supply is not included as a water source for future evaluations in this research. Instead, the volume of water supply is included only for estimating the water demands.

Water Reclamation Facility

This research project focuses on potable and nonpotable water, so nonpotable water sources are also evaluated. This includes the Norman WRF, which discharged an average of 9.9 MGD between 2004 and 2012. The discharge was relatively constant between 2004 and 2012, even as the population was growing. This is consistent with the water supplied by the potable sources during this time period. The average daily discharge rates are shown in the figure below.

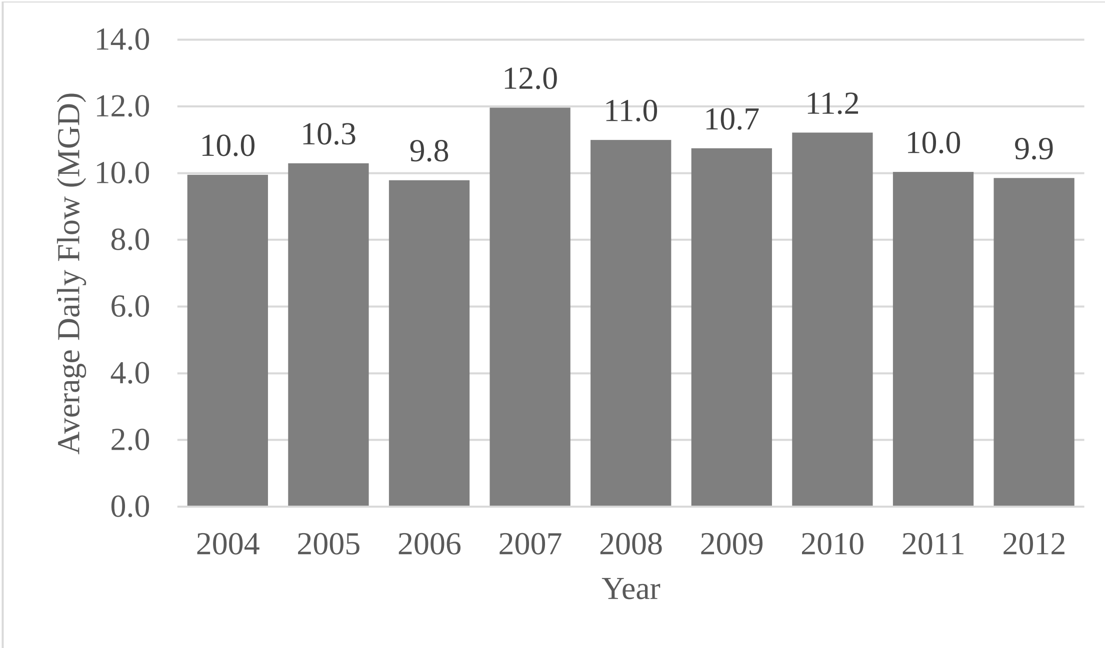

Figure 11. Yearly Average Daily Effluent Flows from 2004 to 2012

As expected, the average flow during the year varied from month to month, with the lowest flows during the winter and the highest flows during the summer, as shown in the figure below.

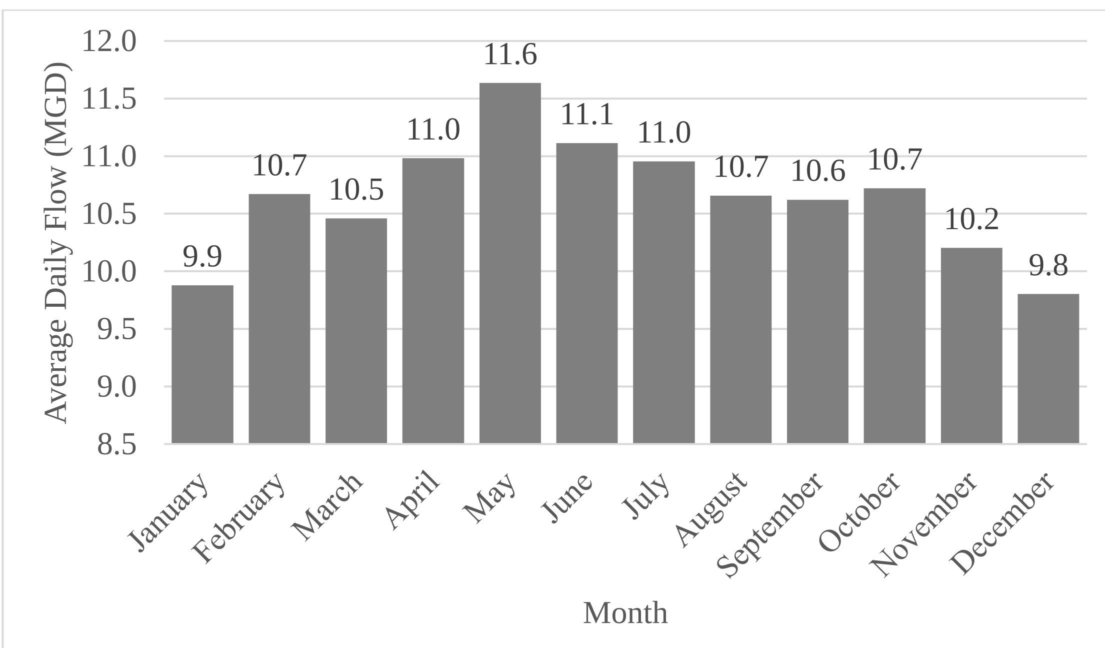

Figure 12. Monthly Average Daily Effluent Flows

The Norman WRF discharges into the South Canadian River, which is accustomed to receiving the average day flow. There is no minimum discharge requirement to maintain flows in the South Canadian River, but the river and the associated ecology have likely adapted to an average flow of 9.9 MGD. It might be detrimental to the river if that minimum flow were to be diverted, so the WRF discharge will remain at 9.9 MGD and the excess flow above 9.9 MGD will be available for urban irrigation. For this research, the available capacity of the Norman WRF for nonpotable flows will range from 0 MGD during the low effluent months to 1.7 MGD during the summer.

Water Supply Summary

The above Water Supply sections have evaluated the available water supplies for both potable and nonpotable sources. Based on the estimates, the total potable supply is 23.0 MGD, the total nonpotable supply is 3.8 MGD, and the total supply available is 26.8 MGD. These values, shown in the table below, will be used for the remainder of this research to evaluate the supply capacity for Norman water demands.

Table 3: Summary of Available Water Supply Sources

Category	Potable Supply (MGD)	Nonpotable Supply (MGD)	Total Supply (MGD)
WTP	17.0	0.0	17.0
Wells	6.0	2.1	8.1
WRF	0.0	1.7	1.7
Total	**23.0**	**3.8**	**26.8**

Water Demand

The water demand is the other side of the water supply coin. The above section analyzed how much water each source can supply, while this section analyzes how much water is demanded by the various customers in Norman. The NUA provided supply data for the WTP, the wells, and the Oklahoma City connection. This information comprises the entire supplied demand. The NUA also provided monthly meter record summaries for six use categories: residential, commercial, industrial, institutional, the University of Oklahoma, and sprinklers. This data is analyzed as a percentage of the entire supplied demand to estimate current and future water demands. The system-wide and various categorical demands also have different demand scenarios for the average day, maximum month, maximum day, and peak hour demands.

The different demand scenarios for each use category are further comprised of potable and nonpotable demands. The potable and nonpotable demands are estimated by assuming a "baseline" demand for the system, residential, and commercial demands, which is determined using the demand during winter months. The nonpotable demand is then calculated as the difference between the baseline demand and the demand scenario, whether it is average day or maximum day, for example. The peak hour demands are calculated differently because irrigation peak hour demands have different peaking factors, because irrigation (especially nonpotable irrigation) is often restricted to nighttime hours in order to minimize potential contact with skin. These analyses and calculations are described in more detail below.

System-Wide Demand

Yearly Demands

Most approaches to water modeling are fairly simplistic, with aggregated demands placed at major junctions and excluding all pipes less than six inches in diameter. This project needs a more detailed approach to observe how the system responds to various demands in the residential and commercial areas. Thus, accurate data is needed to create accurate models for the single and dual distribution systems. Individual monthly meter demands were not available, but the NUA provided monthly categorical demand from December 2007 through November 2013.

Monthly supply data from July 2000 through September 2013 was used to analyze usage patterns for the City of Norman. This includes the total water treatment plant production, active well production, and purchases from Oklahoma City. The resulting average demands from 2000 through 2013 ranged from 11.7 MGD in 2002 to 14.0 MGD in 2006, inversely correlated with total annual precipitation, as shown in Figure 13. The ADD during this time was 12.48 MGD and the average for the full calendar years of 2001 through 2012 is 12.56 MGD. Because these two averages are nearly equal, the 2001-2012 average of 12.56 MGD is used because it is slightly more conservative for future demand projections.

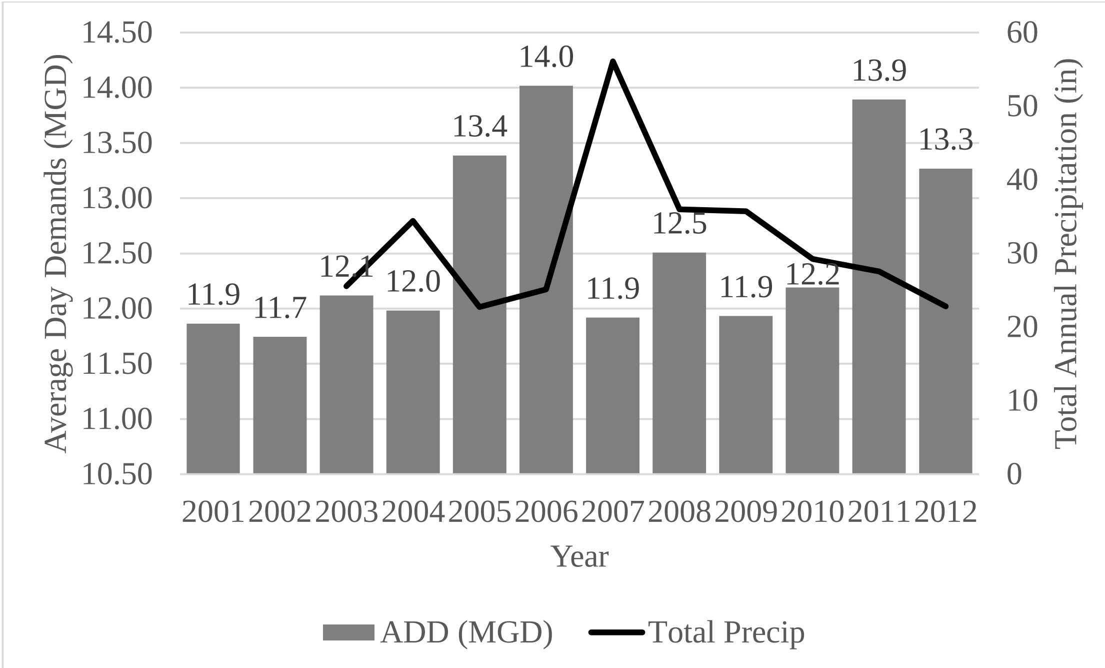

Figure 13: Average Day Demands by Year from 2001 to 2012 with Annual Precipitation

Figure 13 shows that the demands are relatively consistent from year to year, with peaks happening during low-precipitation years. This happened even though the City of Norman instituted mandatory water conservation measures in 2011, 2012, and 2013. The mandatory conservation measures temporarily reduced the peak demands, but did not have much effect on the yearly demands. The figure also shows that the annual precipitation has been gradually decreasing, though more recent data show that the total annual precipitation fluctuates between low and high values, with few average years (Mesonet, 2017). Because the demands inversely correlate with the precipitation and the precipitation fluctuates between low and high, the average demand will be used for future projections. This will help to balance between the years with low and high precipitation and demands.

Monthly and Seasonal Demands

Monthly and seasonal demands are determined in addition to the average daily demand during the year because demands fluctuate from month to month and season to season. The demands for each month were averaged from 2001 through 2013 to estimate the monthly demands. During this time, the City of Norman periodically instituted mandatory odd/even watering in attempts to reduce the overall demands in 2010, 2011, and 2012 (Norman, 2012).[2] Due to lack of daily data, it is unknown how much effect this had on the system-wide demands, but it may have reduced the maximum month and maximum day demands such that they were consistent with previous years' peak demands. When the demands were averaged by month, the average monthly demands ranged from 9.48 MGD in December to 17.45 MGD in July, as shown in Figure 14. The absolute maximum monthly demand was 23.0 MGD in July 2012 and the absolute minimum monthly demand was 7.9 MGD in February 2001.

[2] Mandatory odd/even watering was instituted in 2015 (Norman, 2016).

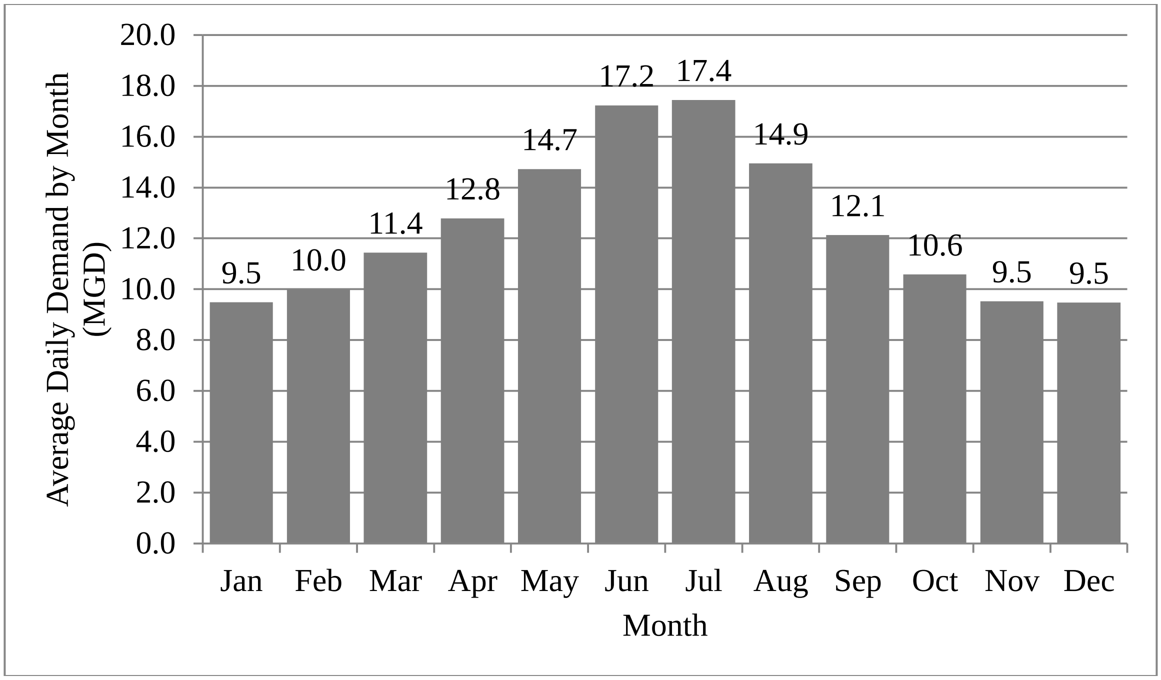

Figure 14: Daily Average Demand by Month from 2000 to 2013

These demands are comprised of potable and nonpotable demands. A common method for isolating the potable demand is to average the demand from the two lowest months, typically December, January, and/or February (King County Wastewater Treatment Division, 2008; Richardset al., 2008; Chang et al., 2010; Friedman et al., 2013; HydroScience, 2013). This method was used for this study as well, because more detailed information is not available. Figure 14 shows that the months with the lowest demand are January, November, and December. Based on these three months, the system-wide, baseline potable demand is 9.5 MGD.

The baseline demand is comprised of all water uses excluding irrigation. Typical municipalities have a few categories of water use: industrial, residential, municipal, commercial, and loss (Chin, 1999). In addition to these categories, the University of Oklahoma is a major water consumer in Norman. Because detailed billing information was not available for this project, many of the demands had to be estimated.

Assuming a 9.5 MGD baseline potable demand for all users combined, the average irrigation demand was calculated by subtracting the baseline demand from the total for each month, shown in Table 4 below. The average monthly demands for January and December are less than the baseline demand, so it is assumed that there is negligible irrigation demand during these months.

Table 4: Average Total, Potable, and Irrigation Demand by Month

Month	Total Demand (MGD)	Baseline Demand (MGD)	Nonpotable Demand (MGD)	Nonpotable Percent of Total Demand
January	9.49	9.5	0	0
February	9.99	9.5	0.34	3.4%
March	11.44	9.5	1.79	15.6%
April	12.79	9.5	3.14	24.5%
May	14.73	9.5	5.07	34.5%
June	17.23	9.5	7.58	44.0%
July	17.45	9.5	7.79	44.7%
August	14.95	9.5	5.30	35.4%
September	12.13	9.5	2.48	20.4%
October	10.58	9.5	0.93	8.8%
November	9.53	9.5	0	0.0%
December	9.48	9.5	0	0.0%

For commercial uses, irrigation typically comprises 22 percent of the total demand (EPA, 2012). For residential uses, irrigation typically comprises between 30 percent and 60 percent, depending on the location (EPA, 2014). The potable demand has been approximated at 40 percent to 60 percent of a home's total demand (EPA, 2014). The table above shows that the typical irrigation demand in Norman ranges from zero to almost 45 percent of the total demand, which is within the typical range determined by the EPA.

Peaking Factors

Water demands vary from season to season and from hour to hour. These variations must be accurately represented in order to create a realistic hydraulic model. Peaking factors are ratios that are estimated or determined from historical records that compare various flow conditions to average day demand conditions. The various flow conditions typically include (Chin, 2013):

- The daily average in a maximum month (MMD),
- The daily average in a maximum week,
- The daily maximum (MDD),
- The hourly maximum (peak hour demand (PHD)), and
- The hourly minimum.

The flow conditions will be referred to as demand scenarios for the rest of this project. This research project uses the MMD to validate the hydraulic models, and the MMD and MDD to estimate the PHD factor – and subsequently the PHD. Typical MMD factors range from 1.10 to 1.50, with a typical value of 1.20; typical MDD factors range from 1.50 to 3.00, with a typical value of 1.80 (Chin, 2013). Larger peaking factors are typically seen in smaller water systems because the populations tend to be more homogenous and use water at the same times, while larger water systems have a smaller peaking factors because the demands typically occur through the day (Chin, 2013).

For Norman, historical records were used to calculate the MMD and MDD peaking factors from the ADD for 2001 through 2012, and the results are compared with the typical ranges from Chin (2013). The calculated results are provided in Table 5.

Table 5: Historical Peaking Factors from 2001 to 2012

Year	Total Demand (MG)	ADD (MGD)	MMD (MGD)	MMD Peaking Factor	MDD (MGD)	MDD Peaking Factor
2001	4,330.47	11.86	18.62	1.57	22.74	1.92
2002	4,286.36	11.74	17.98	1.53	22.11	1.88
2003	4,422.81	12.12	20.34	1.68	23.70	1.96
2004	4,373.79	11.98	15.96	1.33	21.32	1.78
2005	4,886.50	13.39	17.43	1.30	22.52	1.68
2006	5,116.56	14.02	18.86	1.35	24.26	1.73
2007	4,350.17	11.92	17.04	1.43	22.33	1.87
2008	4,565.66	12.51	18.42	1.47	23.38	1.87
2009	4,355.11	11.93	18.02	1.51	22.12	1.85
2010	4,450.13	12.19	19.03	1.56	22.24	1.82
2011	5,070.94	13.89	20.58	1.48	23.94	1.72
2012	4,843.03	13.27	22.27	1.68	24.82	1.87
Average				**1.49**		**1.83**

As stated above, the range of MMD peaking factors is 1.10 to 1.50, with a typical value of 1.20. The table shows that the average MMD peaking factor is high at 1.49, which is at the upper end of the range of typical factors (Chin, 2013). In fact, the MMD peaking factor in six of the 12 years were greater than the maximum range value, and every year was greater than the typical MMD peaking factor. As stated above, high peaking factors are more common for small water systems; however, Norman is not a particularly small system with approximately 110,000 users. This indicates though, that the population in Norman may be more homogenous than the numbers alone indicate or that the peaking factors could be driven by Norman's large summer irrigation demands – despite the mandatory water conservation measures undertaken briefly during 2011, 2012, and 2013. Interestingly though, the average MDD peaking factor is close to the typical value at 1.83, and every year is relatively close to the average value. The fairly typical MDD factor indicates either that:

- The population is not as homogenous as thought previously based on the MMD factor, or
- The MDD is close to the MMD, which would reduce the difference between the two values.

The difference between the two values is interesting, but unfortunately there are no conclusive reasons as to why the difference exists.

According to Chin, the normal range for peak hour factors is 2.00 to 4.00, with a typical value of 3.25. Because the MMD factor and MDD factor are high and typical, respectively with their ranges, the estimated PHD factor will be the average of the typical and high values of PHD peaking factors. This equals 3.63 and will be used for the single distribution system and the potable distribution system during the scenarios.

For intermittent uses, such as irrigation, the peak hour flow is the most important flowrate criterion (Asano et al.; 2007). The peak hour flow for potable demand or single distribution systems is often calculated with a peaking factor that accounts for the entire day; for irrigation demand, the peak hour flow for public use areas such as parks and golf courses is calculated by dividing the full flow rate by the time allotted for irrigation. For example, parks are often irrigated at night to allow the parks to be available during the day, so the full irrigation demand must be provided during a four to five hour time period between sunset and sunrise. For residential irrigation, the PHD can be between four to six times greater than the ADD (Asano et al., 2007). For this research project, the irrigation PHD factor will be set at the midpoint of this suggested range by Asano et al., i.e., at 5.00.

In summary, the peaking hour factors (rounded to two significant figures due to the uncertainty inherent in estimated water demands) that are used for current and future demands are provided in Table 6.

Table 6: Summary of Peaking Factors

Demand Scenario	Peaking Factor
Maximum Month Demand	1.5
Maximum Day Demand	1.8
Peak Hour Demand (Single System and Potable System)	3.6
Peak Hour Demand (Nonpotable system)	5.0

Overview of Categorical Demands

The NUA categorizes water users and tracks demands in four main categories:

- Residential
- Commercial
- Institutional
- Industrial

These four categories align with industry-standard water demand categories and Norman zoning land use categories, which are used to spatially allocate demands. Norman also tracks demand for sprinklers and the University of Oklahoma (OU) because OU has historically been a very high demand user. NUA provided monthly demand data from January 2008 through December 2013 for the four bulleted categories above, OU, and sprinklers.

When the metered demands were analyzed in relation to the total monthly supply data, the metered demand did not align with the supply data very well. For most distribution systems, the difference between the supply data and the metered demand equals non-revenue water, which is comprised of non-metered demand (e.g., fire hydrants, some municipal users) and water loss. In Norman, the calculated non-revenue water ranges from -3.1 percent (i.e., there was more demand than supply, which is physically impossible) to 24.3 percent. Additionally, the non-revenue water percentage varied wildly from month to month. For example, the non-revenue water percentage was 2.3 percent in April 2012, then it increased to 10.0 percent in May 2012, then decreased to -5.6 percent in June 2012. For Norman, these discrepancies indicate that there was some sort of problem with the meter data and the supply data; however, no concrete explanation was found.[3] As a result, the metered demand data was chosen as the basis for each category as a percentage of the total demand from January 2008 through September 2013 (the beginning of the meter records and the end of the supply records). These percentages were then used with the calculated ADD of 12.56 MGD and peaking factors to determine the respective categorical demands.

Residential demands include single-family homes and some apartment complexes where the apartments are metered individually. Commercial demands include businesses, apartment complexes without individual meters, and mobile home parks. Institutional demands include schools, universities, religious houses of worship, and municipal use. Industrial demands include industries. These demands are explained in more detail in subsequent sections.

[3] During the initial analysis phase of this research, the discrepancies were reported to the City of Norman. The staff discovered several irregularities in how their data was recorded and reported, and has led to changes in how the information is collected, recorded, and reported.

Residential Demand

The residential demand uses the meter demands considered "residential" by the NUA for single-family homes and the apartment complexes with individual meters. The demands are calculated for average day, maximum month, maximum day, and peak hour demands, then divided into a baseline potable demand and nonpotable demands.

Residential Average Day Demand

As mentioned previously, the NUA provided the monthly meter record summary for residential meters for January 2008 through December 2013. Because all of the demands were not consistent with the supply, the residential demands are calculated as a percentage of the total supply from January 2008 – the beginning of the meter records – through September 2013 – the end of the supply records. During this time period, the metered residential demand averaged 73.9 percent of the total supply, with a range of 55.6 percent in July 2008 to 91.3 percent in September 2010, as shown in Table 7.

Table 7. Residential Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	69.6%	73.0%	74.7%	81.1%	80.7%	83.9%	77.2%
February	74.6%	72.8%	75.1%	76.5%	76.3%	82.5%	76.3%
March	65.1%	61.5%	66.8%	64.6%	72.1%	73.3%	67.2%
April	63.4%	76.3%	63.8%	61.0%	68.1%	80.1%	68.8%
May	60.7%	75.1%	61.6%	73.3%	62.1%	69.4%	67.0%
June	74.4%	56.4%	62.7%	61.4%	67.1%	62.0%	64.0%
July	55.6%	73.1%	77.9%	69.0%	61.3%	63.3%	66.7%
August	82.4%	79.4%	59.5%	73.6%	86.5%	68.9%	75.0%
September	74.0%	83.9%	91.3%	85.3%	90.6%	70.2%	82.6%
October	77.1%	75.0%	74.0%	85.1%	85.7%	-	79.4%
November	75.3%	72.0%	85.7%	81.6%	80.8%	-	79.1%
December	78.4%	79.5%	87.4%	83.1%	88.2%	-	83.3%
Average	**70.9%**	**73.2%**	**73.4%**	**74.6%**	**76.6%**	**72.6%**	**73.9%**

Over the 68 months, the residential demand constituted an average of 74.0 percent of the total supply. Norman is a relatively residential community with few industries, so 74 percent residential demand is logical. It corresponds with a study from California where residential demands constituted over half the total demand in primarily-residential communities (Cohem, 2009). The 74.0 percent value is used to calculate the residential average day demand from the total system demand of 12.56 MGD, and results in a 9.3 MGD residential average day demand.

Residential Baseline Demand

To calculate the potable and nonpotable residential demands, the baseline potable demand must first be established. The baseline potable demand for all demands combined was 9.5 MGD. The residential demand comprises approximately 74.0 percent of this 9.5 MGD, which equates to 7.05 MGD.

Residential Single System and Potable Peak Demands

For the residential demands, the maximum month, maximum day, and peak hour demands are calculated by applying the peaking factors to the average day demands. It is assumed throughout the rest of this section that the city-wide peaking factors determined above apply to the categorical demands. The maximum month, maximum day, and single system peak hour demands are applied to average day demand. The potable system peak hour demand is applied to the baseline demand. The nonpotable demand is calculated separately. The numbers are provided in the table below.

Table 8. Residential Single System and Potable System Peak Demands

Demand Scenario	Peaking Factor	Demand (MGD)
Residential Baseline	-	7.05
Residential Average Day	1.00	9.30
Residential Maximum Month	1.49	13.86
Residential Maximum Day	1.83	17.02
Residential Peak Hour (Single System)	3.63	33.76
Residential Peak Hour (Potable)	3.63	25.60

Residential Nonpotable Demands

This research primarily focuses on the effect of nonpotable residential demands, so the nonpotable demands must be calculated separately. The residential nonpotable demands are calculated by first subtracting the baseline demand from the average day demand, maximum month, or maximum day demands. These result in the average day irrigation demand, maximum month irrigation demand, and maximum day irrigation demand, respectively. The peak hour irrigation demand is calculated by multiplying average day irrigation demand by the peaking factor for peak hour nonpotable use (5.0). These demands are summarized below.

Table 9. Residential Nonpotable Demands

Demand Scenario	Demand (MGD)
Residential Average Day	9.30
Residential Baseline	7.05
Average Day Residential Irrigation	2.25
Maximum Month Residential Irrigation	6.81
Maximum Day Residential Irrigation	9.97
Peak Hour Residential Irrigation	11.25

Commercial Demand

The commercial demand uses the meter demands that are considered "commercial" by the NUA for any non-industrial business, including offices, restaurants, car washes, hotels, grocery stores, boutique stores, mobile home parks, large apartment complexes, homeowner associations, and superstores (e.g., Walmart, Lowe's, and Office Depot). The demands are calculated as average day, maximum month, maximum day, and peak hour demands. These demands are then broken down into a baseline potable demand and nonpotable demands.

Commercial Average Day Demand

As mentioned previously, the NUA provided the monthly meter record summary for commercial meters for January 2008 through December 2013. These records were calculated on a monthly basis as a percentage of the total supply, as discussed above. From January 2008 through September 2013, the metered commercial demand averaged 13.5 percent of the total supply, with a range of 8.1 percent in July 2008 to 16.3 percent in December 2010, as shown in Table 10.

Table 10. Commercial Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	13.2%	12.9%	15.5%	14.3%	14.6%	16.4%	14.5%
February	13.7%	13.6%	13.3%	13.5%	14.5%	15.6%	14.0%
March	12.5%	10.8%	12.5%	12.1%	13.7%	14.6%	12.7%
April	12.4%	13.3%	12.2%	10.2%	12.7%	16.0%	12.8%
May	10.9%	13.1%	11.1%	11.6%	11.6%	13.0%	11.9%
June	15.7%	9.5%	10.6%	10.1%	13.5%	14.1%	12.2%
July	8.1%	10.2%	12.9%	10.5%	9.3%	11.3%	10.4%
August	11.9%	12.1%	9.7%	11.0%	13.4%	13.4%	11.9%
September	34.9%	14.3%	14.1%	13.1%	14.6%	12.8%	17.3%
October	13.5%	13.6%	13.8%	13.9%	14.8%	-	13.9%
November	14.0%	13.8%	15.6%	14.4%	14.4%	-	14.5%
December	13.6%	14.5%	16.3%	15.7%	16.7%	-	15.4%
Average	**14.5%**	**12.6%**	**13.1%**	**12.5%**	**13.6%**	**14.2%**	**13.5%**

The entire distribution system has an average day demand of 12.56 MGD, so the commercial average day demand is 1.69 MGD using the 13.5 percent average of the total supply, as indicated in Table 10.

Commercial Baseline Demand

To calculate the potable and nonpotable commercial demands, the baseline potable demand must first be established. The baseline demand for all demands combined is 9.5 MGD and the commercial demand comprises 13.5 percent of the total demand. This equates to a commercial baseline demand of 1.32 MGD.

Commercial Single System and Potable Peak Demands

For the commercial demands, the maximum month, maximum day, and peak hour demands are calculated by applying the system-wide peaking factors to the average day demands. In reality, commercial demands are comprised of a wide variety of businesses with their own peaking factors. For example, restaurants have different peaking factors than offices and apartment complexes. However, the specific meter accounts for the various businesses were not provided so the commercial demands cannot be further broken down for the specific types of business. As a result, the commercial peaking demands are simplified by assuming that the maximum month, maximum day, and single system peak hour demands are applied to average day demand. The potable system peak hour demand is applied to the baseline demand. The nonpotable demand is calculated separately. The results are provided in the table below.

Table 11. Commercial Single System and Potable System Peak Demands

Demand Scenario	Peaking Factor	Demand (MGD)
Commercial Baseline	-	1.32
Commercial Average Day	1.00	1.69
Commercial Maximum Month	1.49	2.52
Commercial Maximum Day	1.83	3.10
Commercial Peak Hour (Single System)	3.63	6.15
Commercial Peak Hour (Potable)	3.63	4.79

Commercial Nonpotable Demands

This research also includes nonpotable commercial demands with the nonpotable residential demands. The commercial nonpotable demands are calculated by first subtracting the baseline demand from the average day demand, maximum month, or maximum day demands. These result in the average day irrigation demand, maximum month irrigation demand, and maximum day irrigation demand, respectively. The peak hour irrigation demand is calculated by multiplying average day irrigation demand by the peaking factor for peak hour nonpotable use (5.0). These demands are summarized below.

Table 12. Commercial Nonpotable Demands

Demand Scenario	Demand (MGD)
Average Day Commercial Irrigation	0.37
Maximum Month Commercial Irrigation	1.20
Maximum Day Commercial Irrigation	1.78
Peak Hour Commercial Irrigation	1.85

Industrial Demand

The industrial demand uses the meter demands that are considered "industrial" by the NUA for the industries in Norman, including Johnson Controls, the U.S. Postal Training Center, Astellas Pharma Technologies, and Dolese. Some of these demands have the potential to use nonpotable water, but this potential is very dependent on the specific industry; as such, the industrial nonpotable demand is not evaluated in this research. The following sections calculate the industrial demands for average day, maximum month, maximum day, and peak hour demands.

Industrial Average Day Demand

When the NUA industrial meter records are analyzed as a percentage of the total supply from January 2008 to September 2013, the industrial demand averages at 2.2 percent of the total supply. The percentages range from 1.6 percent in July 2012 to 2.7 percent in September, November, and December 2010, and September 2012, as shown in the table below.

Table 13. Industrial Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	1.7%	2.0%	2.1%	2.5%	2.3%	2.4%	2.2%
February	2.3%	2.3%	2.4%	2.2%	2.4%	2.4%	2.3%
March	1.9%	1.9%	2.3%	2.1%	2.0%	2.3%	2.1%
April	1.9%	2.3%	2.3%	1.7%	2.0%	2.4%	2.1%
May	1.7%	2.3%	2.1%	1.9%	2.0%	2.1%	2.0%
June	2.2%	1.8%	1.9%	1.8%	2.2%	2.1%	2.0%
July	1.8%	2.3%	2.6%	2.2%	1.6%	1.9%	2.1%
August	2.2%	2.5%	1.9%	2.2%	2.3%	2.2%	2.2%
September	2.2%	2.6%	2.7%	2.4%	2.7%	1.9%	2.4%
October	2.3%	2.2%	2.3%	2.3%	2.4%	-	2.3%
November	2.0%	2.2%	2.7%	2.4%	2.2%	-	2.3%
December	2.1%	2.4%	2.7%	2.4%	2.4%	-	2.4%
Average	**2.0%**	**2.2%**	**2.3%**	**2.2%**	**2.2%**	**2.2%**	**2.2%**

The entire distribution system has an average day demand of 12.56 MGD, so the industrial average day demand is 0.28 MGD using the 2.2 percent average of the total supply, as shown in Table 13.

Industrial Peaking Demands

For the industrial demands, the maximum month, maximum day, and peak hour demands are calculated by applying the peaking factors to the average day demands. While the industrial demands typically have their own peaking factors, the system-wide peaking factors are applied to the industrial demands. Even though industries are typically more uniform in their water use than residential or commercial demands, peaking demands are calculated for the industrial demand because the industrial demands still have maximum month and maximum day demands when there may be a rush order or high production period. These peaking demands are calculated by applying the maximum month, maximum day, and single system peak hour factors to the average day demand. The numbers are provided in the table below.

Table 14. Industrial Single System and Potable System Demands

Demand Scenario	Peaking Factor	Demand (MGD)
Industrial Average Day	1.00	0.28
Industrial Maximum Month	1.49	0.41
Industrial Maximum Day	1.83	0.51
Industrial Peak Hour	3.63	1.01

Institutional Demand

The institutional demand uses the meter demands that are considered "institutional" by the NUA for organizations like schools, churches, synagogues, nonprofits, and hospitals. Some of these demands have the potential to use nonpotable water, but this potential is very dependent on the specific institution; as such, the institutional nonpotable demand is not evaluated in this research. The following sections calculate the institutional demands for average day, maximum month, maximum day, and peak hour demands.

Institutional Average Day Demand

When the NUA institutional meter records are analyzed as a percentage of the total supply from January 2008 to September 2013, the institutional demand averages at 0.87 percent of the total supply. The percentages range from 0.39 percent in July 2012 to 1.3 percent in November 2010, as shown in the table below.

Table 15. Institutional Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	1.13%	0.75%	0.98%	1.19%	0.86%	1.16%	1.01%
February	1.14%	0.94%	1.00%	0.84%	1.09%	1.25%	1.04%
March	0.90%	0.69%	1.00%	0.69%	0.79%	1.12%	0.87%
April	0.72%	0.86%	0.78%	0.57%	0.71%	1.21%	0.81%
May	2.68%	0.99%	0.80%	0.74%	0.62%	1.00%	1.14%
June	0.61%	0.64%	0.72%	0.53%	0.48%	0.77%	0.63%
July	0.55%	0.48%	0.63%	0.41%	0.27%	0.39%	0.46%
August	0.44%	0.81%	0.46%	0.43%	0.55%	0.46%	0.53%
September	0.80%	0.90%	1.01%	0.64%	0.94%	0.83%	0.85%
October	0.98%	1.08%	1.12%	0.84%	1.14%	-	1.03%
November	0.99%	1.17%	1.30%	0.82%	1.04%	-	1.06%
December	0.84%	1.12%	1.15%	1.13%	1.09%	-	1.07%
Average	**0.98%**	**0.87%**	**0.91%**	**0.74%**	**0.80%**	**0.91%**	**0.87%**

The entire distribution system has an average day demand of 12.56 MGD, so the institutional average day demand is 0.11 MGD using the 0.87 percent average of the total supply, as shown in Table 15.

Institutional Peaking Demands

For the institutional demands, the maximum month, maximum day, and peak hour demands are calculated by applying the peaking factors to the average day demands. While the institutional demands may have their own peaking factors, the system-wide peaking factors are applied to the institutional demands. The maximum month, maximum day, and single system peak hour demands are applied to average day demand. The numbers are provided in the table below.

Table 16. Institutional Single System and Potable System Demands

Demand Scenario	Peaking Factor	Demand (MGD)
Institutional Average Day	1.00	0.11
Institutional Maximum Month	1.49	0.17
Institutional Maximum Day	1.83	0.21
Institutional Peak Hour	3.63	0.42

University of Oklahoma Demand

The University of Oklahoma is the single largest consumer in Norman, with a 2015 enrollment of 27,445 students in Norman (OU Institutional Research & Reporting, 2016). As a result, the NUA tracks OU meter accounts as its own category along with the four other main categories discussed in the sections above.

OU Average Day Demand

When the OU meter records are analyzed as percentages of the total supply from January 2008 to September 2013, the OU demand averages at 1.91 percent of the total supply, which is twice the institutional demand percentage. The OU percentages range from 0.98 percent in April 2011 to 3.21 percent in January 2010, as shown in the table below.

Table 17. OU Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	1.82%	2.07%	3.21%	1.23%	1.41%	1.95%	1.95%
February	1.97%	1.64%	2.21%	1.18%	1.52%	2.26%	1.80%
March	2.07%	1.84%	2.28%	1.21%	1.70%	2.15%	1.88%
April	1.78%	2.07%	2.05%	0.98%	1.59%	1.86%	1.72%
May	1.85%	2.09%	1.98%	1.46%	1.63%	2.14%	1.86%
June	1.66%	1.46%	1.62%	1.22%	1.71%	1.37%	1.51%
July	1.33%	1.65%	2.31%	1.12%	1.12%	1.04%	1.43%
August	2.04%	1.88%	1.48%	1.28%	1.59%	1.55%	1.64%
September	2.04%	2.17%	3.06%	1.90%	1.92%	1.72%	2.13%
October	2.56%	3.05%	1.69%	2.31%	2.46%	-	2.41%
November	2.60%	2.54%	1.86%	2.53%	2.61%	-	2.43%
December	2.09%	2.50%	1.60%	2.55%	2.36%	-	2.22%
Average	**1.98%**	**2.08%**	**2.11%**	**1.58%**	**1.80%**	**1.78%**	**1.91%**

The entire distribution system has an average day demand of 12.56 MGD, so the OU average day demand is 0.24 MGD using the 1.91 percent average of the total supply, as shown in Table 17.

OU Peak Demands

OU already uses reclaimed water from the Norman WRF for nonpotable use, so all of the demand analyzed above is their potable demand. Thus, OU does not have a baseline demand or separate irrigation demand. However, OU does still have peak demands, including maximum month, maximum day, and peak hour demands. Because OU is a state university, the actual maximum month, maximum day, and peak hour demands do not typically occur at the same time as the rest of the distribution system. According to the meter records, OU maximum demands occur during the school year instead of during the summer. However, the distribution system is analyzed as a whole, so the system-wide peaking factors are applied to the OU average day demand instead of calculating separate peaking factors for the OU demands. The results are presented in the table below.

Table 18. OU Single System and Potable System Demands

Demand Scenario	Peaking Factor	Demand (MGD)
OU Average Day	1.00	0.24
OU Maximum Month	1.49	0.36
OU Maximum Day	1.83	0.44
OU Peak Hour	3.63	0.87

Sprinkler Demand

Sprinkler Average Day Demand

The NUA tracks sprinkler demands in addition to the other five categories. All of the sprinkler demands can be considered nonpotable demands because sprinklers are used for irrigation, and irrigation is the primary nonpotable use considered in this research. The sprinkler meter demands were analyzed like the other five demand categories as a percentage of the total system-wide demand from January 2008 through September 2013. The sprinkler demand was an average of 4.5 percent of the total supply, with a range from 0.33 percent in March 2010 to 10.8 percent in September 2012, as shown in Table 19.

Table 19. Sprinkler Demand as Percentage of Total Supply by Month

	2008	2009	2010	2011	2012	2013	Average
January	0.82%	0.90%	0.59%	1.31%	1.49%	1.77%	1.15%
February	0.75%	0.67%	8.68%	0.72%	0.99%	0.81%	2.10%
March	0.57%	1.42%	0.33%	1.38%	1.26%	0.62%	0.93%
April	1.24%	2.87%	0.87%	3.27%	2.07%	1.87%	2.03%
May	1.99%	3.12%	2.54%	6.18%	3.55%	1.88%	3.21%
June	4.09%	2.99%	3.82%	5.77%	5.68%	5.29%	4.61%
July	3.79%	6.87%	6.38%	7.50%	6.77%	5.13%	6.08%
August	8.58%	9.14%	5.35%	7.78%	9.89%	6.45%	7.87%
September	6.79%	9.03%	10.35%	9.69%	10.81%	6.39%	8.84%
October	6.25%	5.25%	8.02%	9.89%	9.46%	-	7.78%
November	4.53%	3.62%	6.83%	7.09%	6.44%	-	5.70%
December	2.05%	2.23%	3.64%	3.58%	4.30%	-	3.16%
Average	**3.46%**	**4.01%**	**4.78%**	**5.35%**	**5.23%**	**3.36%**	**4.45%**

The entire distribution system has an average day demand of 12.56 MGD, so the sprinkler average day demand is 0.56 MGD using the 4.45 percent average of the total supply, as shown in Table 19.

Sprinkler Peak Demands

The sprinkler peak demands are calculated similarly to the residential and commercial demands, in that the system-wide peaking factors are applied to the average day demands to calculate the sprinkler demand on the potable system – particularly for the single-system peak demand. There is no potable demand calculated for the sprinkler demand, however, because the sprinkler demand is assumed to be a nonpotable use. Thus, the nonpotable peaking factor is used to calculate the nonpotable sprinkler peak hour demand. The results are provided in the table below.

Table 20. Sprinkler Single System and Nonpotable System Demands

Demand Scenario	Peaking Factor	Demand (MGD)
Sprinkler Average Day	1.00	0.56
Sprinkler Maximum Month	1.49	0.83
Sprinkler Maximum Day	1.83	1.02
Sprinkler Peak Hour (Single System)	3.63	2.03
Sprinkler Peak Hour (Nonpotable)	5.00	2.80

Water Loss

The categorical demand percentages equal approximately 97 percent of the total supply, leaving 3 percent of the water unaccounted for, i.e., a three percent water loss. According to NUA records, water loss accounts for five to seven percent of the overall demand (City of Norman, 2014). Assuming an ADD of 12.56 MGD, this equates to a loss of 0.63 MGD to 0.88 MGD. Three percent of the 12.56 ADD equals an average of 0.40 MGD. While three percent does not accurately reflect what is happening in the Norman water distribution system based on the NUA records, the categorical demands have some water loss incorporated into them because of the estimation methods, and there are some rounding and approximation errors inherent in the process. As a result, the three percent water loss will be used for all demand scenarios and all distribution systems, including the potable and nonpotable.

Future System Demand

A major component of this research project involves evaluating the state of the Norman water distribution system in 2062; thus, the 2062 demands must be estimated. There are a variety of models for projecting and estimating future demands, but one of the most common is using a per capita demand that is assumed to be constant over the study period. This research project uses this method and verifies it with previous studies of Norman.

2012 Average Per Capita Demand

The average per capita demand method calculates an average day per capita demand based on the system-wide average day demand. The calculated per capita demand is then applied to a future population projection to estimate the future water demand. The per capita demands can be adjusted to account for a range of factors reflected by decreasing per capita demands, stable per capita demands, or increasing per capita demands. As stated in the literature review, the indoor residential per capita demand has been decreasing in the United States due to more water-efficient fixtures (DeOreo and Mayer, 2012). The numbers for Norman support this, as will be demonstrated soon, however, a conservative estimate is to keep the per capita demand stable or even increase it a little. This will account for potential industrial and commercial growth or water scarcity. For this research, the average per capita demand will be calculated based on historical records and applied to the 2012 service population.

From 2001 through 2012, the average per capita demand was 135 gallons per capita day (gpcd), with a range of 127 gpcd in 2007 to 152 in 2006. These values are comparable to other Oklahoma towns and distribution service areas in the Central Region, which typically have average per capita demands ranging from approximately 60 gpcd to approximately 250 gpcd (OWRB, 2012). The yearly average per capita demands are shown in Figure 15.

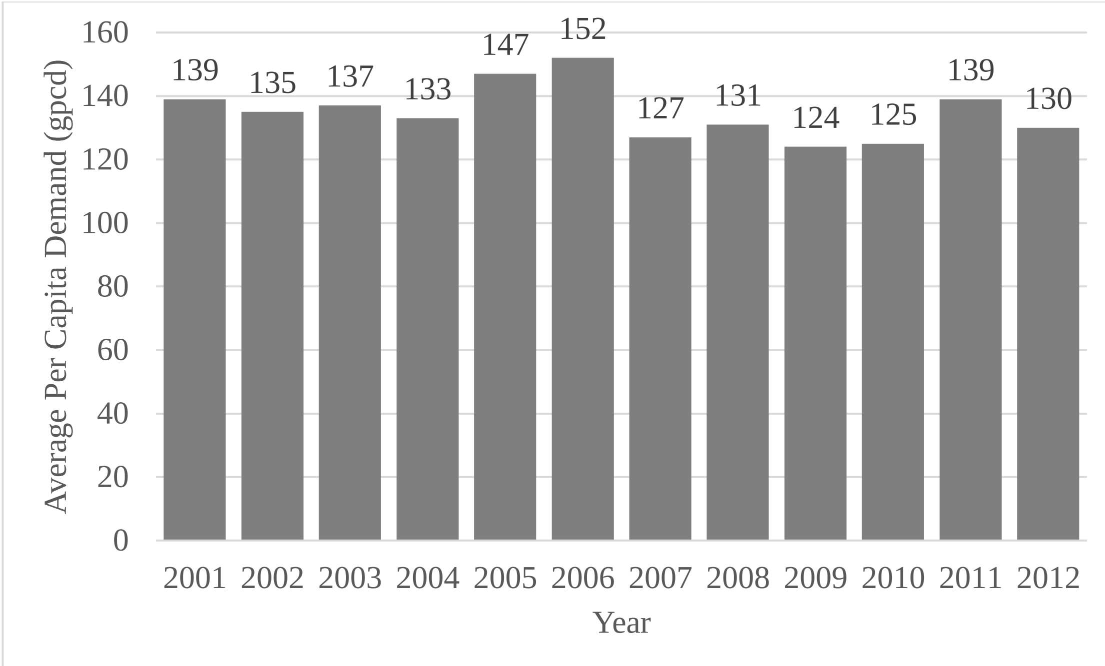

Figure 15. Average Per Capita Demands from 2001-2012

The overall trend is slightly downward, with a few notable exceptions during low-precipitation years. Even still, the per capita demand in 2011 was noticeably lower than it was in 2005 and 2006, even though there was a similar amount of precipitation during those three years. While the general trend decreases and is likely to continue falling as water conservation measures are increased and water fixtures become more efficient, the average per capita demand of 135 gpcd will be used to predict future demands in order to provide a more conservative value.

Projected 2062 Demands

Using the estimated population values from above, the future water demands were estimated through 2065 using the average per capita demand of 135 gpcd and the projected populations, assuming an 88 percent service population. The results can be seen in Figure 16.

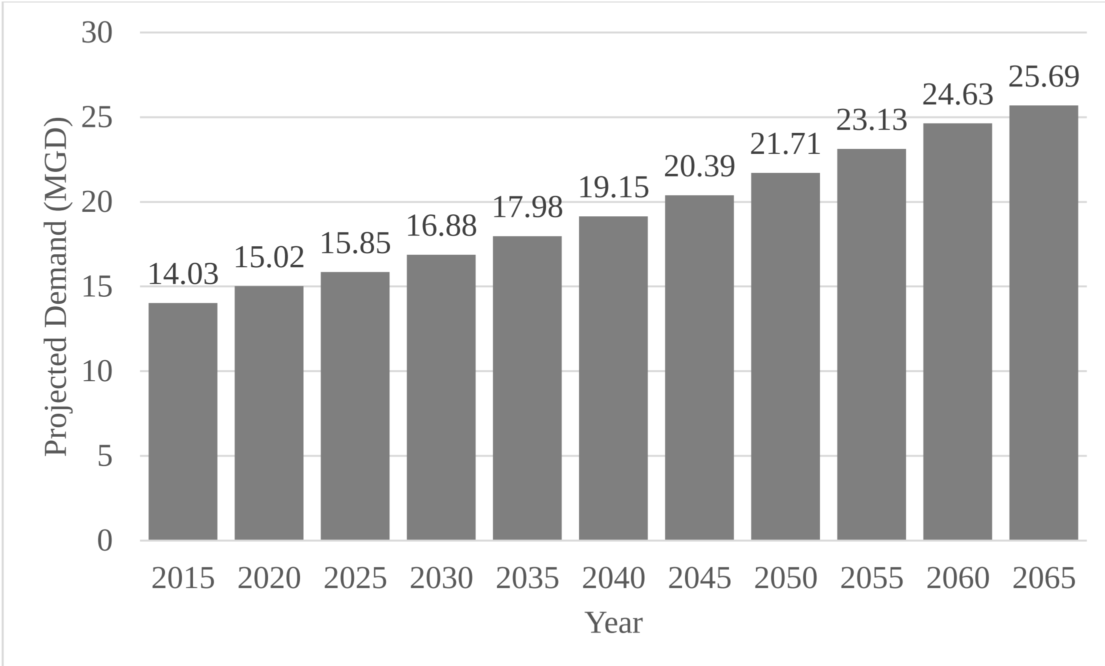

Figure 16. Projected System-Wide Demands from 2015-2065

The 2062 water demand was linearly interpolated between 2060 and 2065 to be approximately 25.1 MGD. The combined capacity of the water treatment plant capacity and the well production is 18.0 MGD, which would be reached in 2035.

The categorical water demand percentages (i.e., residential, commercial, industrial, institutional, OU, and sprinkler), as well as the baseline's fraction of the ADD, were assumed to remain constant between 2012 and 2062. The projected 2062 categorical demands are provided in the table below.

Table 21. 2062 Predicted Demands

Use Category	2062 Baseline Demand (MGD)	2062 ADD (MGD)	2062 MMD (MGD)	2062 MDD (MGD)	2062 PHD (MGD)	2062 Potable PHD (MGD)	2062 NP PHD (MGD)
System	18.95	25.05	37.32	45.84	90.93	68.78	33.20
Residential	14.00	18.51	27.58	33.87	67.18	50.82	22.55
Commercial	2.55	3.37	5.02	6.17	12.23	9.25	4.11
Industrial	0.55	0.55	0.82	1.01	2.00	2.00	-
Institutional	0.22	0.22	0.33	0.40	0.79	0.79	-
OU	0.48	0.48	0.71	0.88	1.74	1.74	-
Sprinkler	-	1.12	1.66	2.04	4.05	-	5.58
Water Loss	0.61	0.81	1.20	1.48	2.93	2.22	0.97

Fire Flow

A quick note is required about fire flow demand. A fire flow demand is not calculated herein because a fire flow analysis is calculated using Bentley® WaterGEMS automated Fire Flow Analysis tool. As the Bentley® WaterGEMS User's Manual describes:

> *Fire flows are computed at user-specified locations by iteratively assigning demands and computing system pressures. The program calculates a steady-state analysis for each node in the Fire Flow Alternative. At each node, it begins by running a Steady-State analysis to ensure that the fire flow constraints that have been set can be met without withdrawing Fire Flow from any of the nodes. If the constraints are met in this initial run, the program then begins iteratively assigning the Needed Fire Flow demands at each of the nodes, and checking to ensure that the constraints are met. The program then runs another set of Steady State analyses, this time adding the Maximum Fire Flow (as set in the Fire Flow Upper Limit input box of the Fire Flow Alternative) to whatever normal demands are required at that node, or replacing the normal demands. In either case, the program checks the residual pressure at that node, the Minimum Zone Pressure, and, if applicable, the Minimum System Pressure. If the Fire Flow Upper Limit can be delivered while maintaining the various pressure constraints, that node will satisfy the Fire Flow constraints. If one of more of the pressure constraints is not met while attempting to withdraw the Fire Flow*

> *Upper Limit, the program will iteratively assign lesser demands until it finds the maximum flow that can be provided while maintaining the pressure constraints. If a node is not providing the Fire Flow Upper Limit, it is because the Residual Pressure at that node, the Minimum Zone Pressure, or the Minimum System Pressure constraints are not met while attempting to withdraw the Fire Flow Upper Limit…If a node completely fails to meet the Fire Flow constraints, it is because the network is unable to deliver the Needed Fire Flow while still meeting the pressure constraints. (Bentley, 2013).*

In short, the Fire Flow Analysis determines how much fire flow is available at any given junction, while maintaining a set minimum pressure in the system. This way, the fire flow is evaluated at each selected junction, subject to constraints that are provided in Chapter 5. For this research, the fire flow analysis is applied to each hydrant in the potable distribution systems, both the single distribution system and the potable distribution system.

Summary of Water Demands

The water demands are determined for the entire system for 2012 and 2062 during average day demands, maximum month demands, maximum day demands, and peak hour demands. These demands are shown in the figure below.

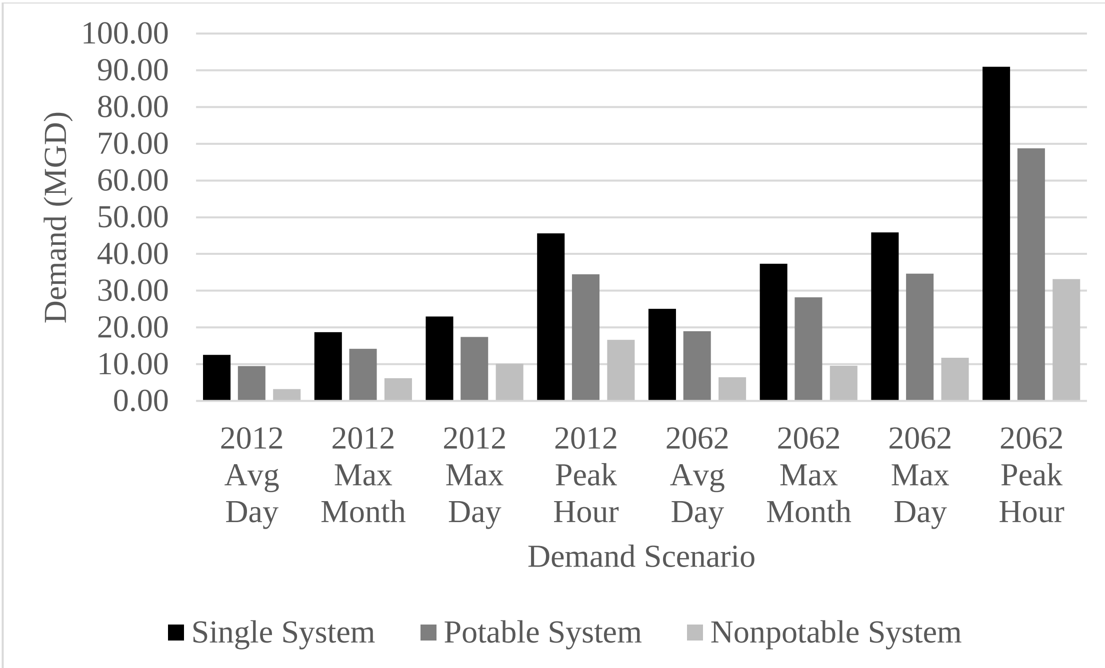

Figure 17. Summary of 2012 and 2062 Demands

The categorical demands were determined using meter billing data and calculated as a percentage of the total demand. These categorical demands are summarized in the figure below.

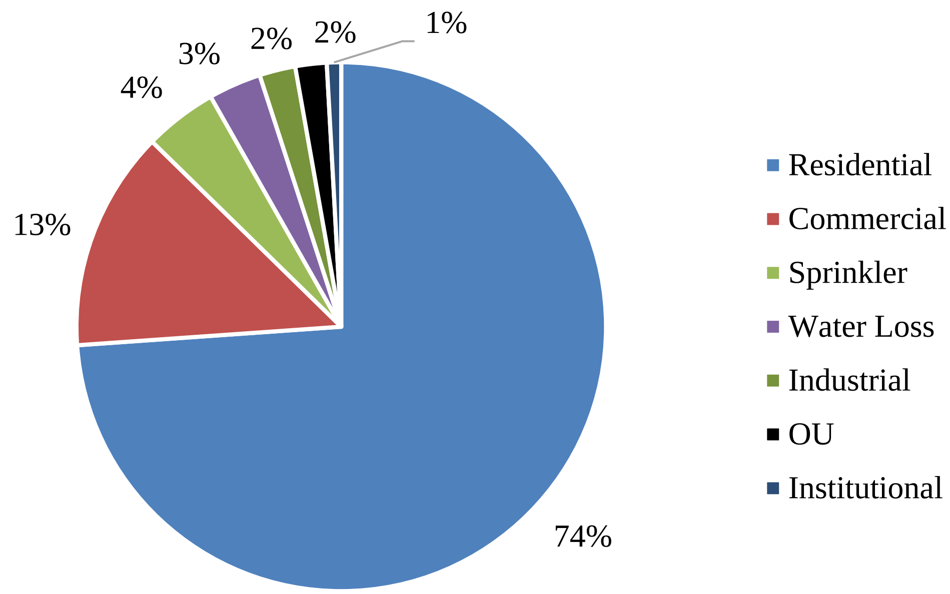

Figure 18. Summary of Categorical Demand Percentages

The 2012 system-wide and categorical demands are summarized in the table below. Note is that the numbers do not necessarily add up, which is caused by some rounding error. The actual numbers used in the spreadsheets are specific down to the hundred gallons, while the numbers reported below are specific to the ten thousand gallons. As a result, this may cause some rounding errors in the column totals. The various demands are presented in Table 22.

Table 22. 2012 System-Wide and Categorical Demands

Demand Type	Average Day Demand (MGD)	Baseline Demand (MGD)	Max Month Demand (MGD)	Max Day Demand (MGD)	Single System Peak Hour Demand (MGD)	Nonpotable Peak Hour Demand (MGD)
System-Wide	12.56	9.05	18.73	23.00	45.62	13.13
Residential Demand	9.30	7.05	13.86	17.02	33.76	11.24
Commercial Demand	1.69	1.32	2.52	3.10	6.15	1.89
Industrial Demand	0.28	0.28	0.41	0.51	1.01	-
Institutional Demand	0.11	0.11	0.16	0.20	0.40	-
OU Demand	0.52	0.52	0.77	0.95	1.88	-
Sprinkler Demand	0.56	-	0.83	1.02	2.03	2.80
Water Loss	0.40	0.31	0.60	0.74	1.47	0.48

The projected 2062 demands are summarized in the table below.

Table 23. 2062 System-Wide and Categorical Demands

Demand Type	Average Day Demand (MGD)	Baseline Demand (MGD)	Max Month Demand (MGD)	Max Day Demand (MGD)	Single System Peak Hour Demand (MGD)	Nonpotable Peak Hour Demand (MGD)
System-Wide	25.05	37.32	45.84	90.93	68.78	33.20
Residential Demand	18.51	27.58	33.87	67.18	50.82	22.55
Commercial Demand	3.37	5.02	6.17	12.23	9.25	4.11
Industrial Demand	0.55	0.82	1.01	2.00	2.00	-
Institutional Demand	0.22	0.33	0.40	0.79	0.79	-
OU Demand	0.48	0.71	0.88	1.74	1.74	-
Sprinkler Demand	1.12	1.66	2.04	4.05	-	5.58
Water Loss	0.81	1.20	1.48	2.93	2.22	0.97

Chapter 4: Allocating Demands and Preparing the Hydraulic Models

Introduction

Chapter 4 covers how the demands are allocated throughout the distribution systems and preparing the hydraulic models for analysis. The demands are allocated spatially using a top-down approach. The top-down approach uses the system-wide and categorical demands to calculate a per-parcel demand, instead of using the individual meter records for each location and calculating a system-wide and categorical demand from the meter records. The most detailed model would have demands placed at each household and business with a corresponding junction to create the most realistic model possible; however, this level of detail is too much. Instead, household and business water usages were estimated for each parcel and then applied to the nearest junction.

In this chapter, first the categorical demands are spatially allocated to each parcel served by the Norman distribution system. Then, the demands are loaded into the hydraulic model of the existing single distribution system, the model is calibrated, and finally verified. Once the 2012 single distribution system is calibrated, the potable and nonpotable distribution systems are created based on the existing distribution system. The 2012 potable distribution system uses the existing single distribution system with only the potable demands and fire flow. The 2012 nonpotable distribution system uses the existing single distribution system as the initial basis, but adapts the water sources and water line diameters to supply the 2012 nonpotable demands to the entire distribution system. Then, the 2062 single distribution system is created by adding new water lines along square miles and a new elevated storage water tank to provide additional storage and help maintain pressures in the distribution system. The 2062 potable distribution system is the same as the 2062 single distribution system, but with the reduced demands on it. Finally, two nonpotable distribution systems are proposed for the 2062 simulations for a service area with a maximum day demand of 3.8 MGD, which is the capacity of the nonpotable sources as discussed in Chapter 3.

All hydraulic models are created in the Bentley® WaterGEMS software, which is an industry-standard software for hydraulic analysis. All simulations are performed as steady-state analyses and the Hazen-Williams equation is used to calculate the friction losses.

Allocating 2012 Demands

The first step in preparing the hydraulic model determined where to allocate the 2012 demands for the various categorical demands based on parcel-level data from the City of Norman. The nodal demands for the water model were calculated based on a top-down approach (Walksi et al., 2001). The categorical demands estimated in Chapter 3 are divided between the property parcels served by the Norman water distribution system. Each property parcel served by the distribution system had two demands – one for potable demand and a second for nonpotable demands. The potable demands are based on either population for residential parcels or on the footprint size of the buildings located on the parcel. The nonpotable demands are based on each parcel's pervious area. The specific methods for determining these demands are described in the below sections.

Parcel Processing

The City of Norman has an online interactive GIS map with a variety of layers that can be downloaded by anyone. For this research, I could access a wide variety of shapefiles that I then manipulated to calculate and extract needed information to spatially allocate demands and then load the hydraulic model. The hydraulic models were created using one set of shapefiles from the City of Norman and the demands were spatially allocated using another set of shapefiles, though there is inherently some overlap between the two groups of shapefiles. This section discusses the shapefiles used to spatially allocate the demands. The next section on creating the hydraulic models discusses the shapefiles used to create the hydraulic model. The shapefiles, as originally downloaded, were separate from each other and needed processing before the demands for each parcel can be calculated. The original shapefiles included a few fields and some shapefiles had more information than others. The various shapefiles and their included fields relevant to allocating demands are provided in the table below.

Table 24. Original Shapefiles Used to Spatially Allocate the Categorical Demands

Shapefile	Relevant Included Fields	Original Number of Elements
Buildings	Area in square feet	70,441
2010 Census Tracts	Total Population Number of Households Average Household Size	29
Parcels	Addresses Area in square feet	42,000
Parking	Parking type (e.g., paved, island, or unpaved) Area in square feet	5,104
Pavement	Pavement code (e.g., patio or driveway) Area in square feet	91,164
Pools	Area in square feet	3,709
Schools	Area in square feet	24
Water Mains	Line Type Diameter	24,216
Zones	Zone code	4,025

First, a spatial join between the Zones shapefile, Schools shapefile, and the Parcel shapefile assigned a use code to each parcel. This identified each parcel as residential, commercial, industrial, or another classification (see Appendix B for the full list of classifications). The spatial distribution of zones is shown in Figure 17 below.

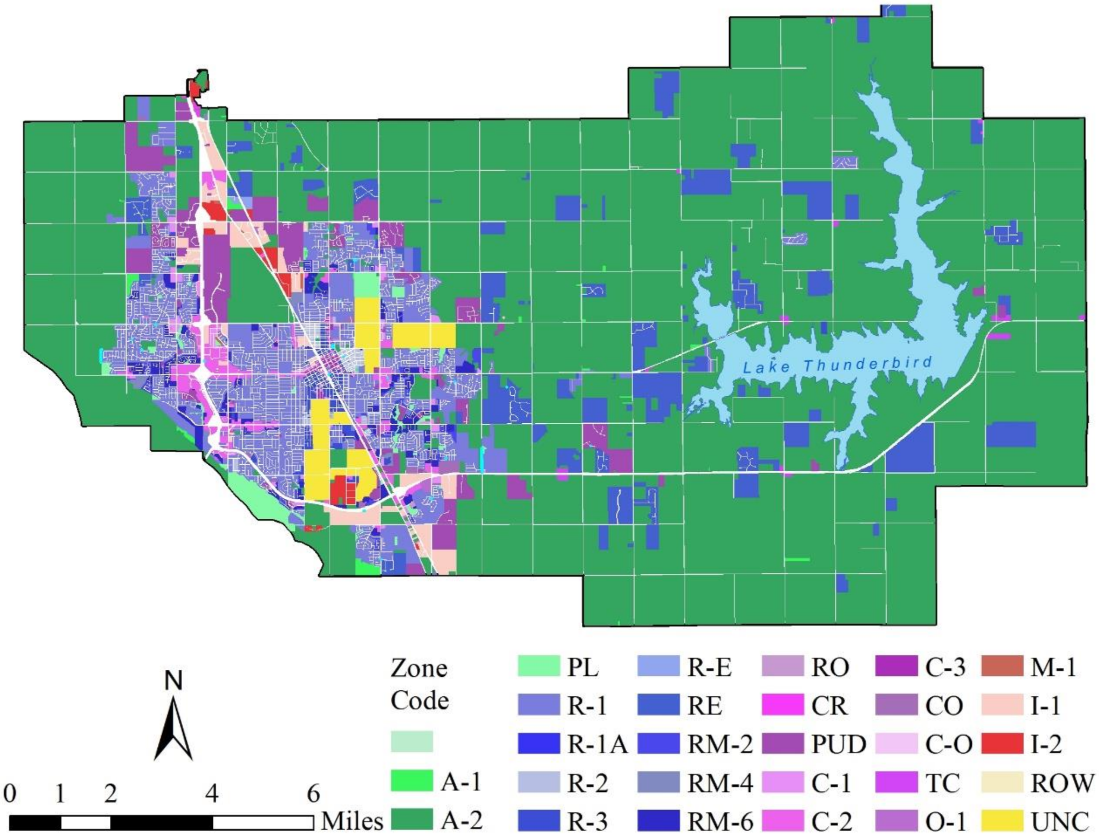

Figure 19. Norman Zoning Map

Briefly, the agricultural and park parcels are color-coded in shades of green; residential parcels are color-coded in blue; commercial parcels are color-coded in purple; industrial parcels are color-coded in reds; and right-of-way and uncategorized are color-coded in yellow. The map shows that the majority of Norman is zoned agricultural, though most of central Norman is zoned residential, with some commercial. OU is coded as uncategorized, as are some other parcels in Hall Park.

Next, the parcels were spatially joined with the Buildings, Parking, Pavement, and Pool shapefiles with a summary of the numeric attributes of the elements that were located in an individual parcel. The join summarized the number of elements per parcel, the sum of the areas of the elements per parcel, the minimum element area per parcel, and the maximum element area per parcel. This was performed in four separate joins, each one building on the previous join. For the Buildings shapefile, the number of elements per parcel and the maximum element area were used to identify which building might be a main building (e.g., house or business) and which building(s) might be outbuildings. Some elements overlapped parcels, particularly the buildings shapefile. Because many of the residential parcels reported having more than one building on them, they were manually back-checked and the attributes were corrected to reflect the actual number of buildings on that parcel and the area of the buildings.

Once the joins were complete, fields were added to the Parcel shapefile to calculate the impervious area and the pervious area. The impervious area was calculated as the sum of the building footprints, the pavement area, the parking area, and the pool area (if there were a pool). Next, the impervious area was calculated as a percentage of the parcel total area. In some cases, the impervious area was reasonable in that it ranged from 0 to 100 percent. In many cases, the impervious area was greater than 100 percent, which indicated that one or more element overlapped one or more parcels, which assigned the areas to each parcel. This problem had two fixes. First, the buildings or pavement sometimes overlapped two normal-sized parcels, so the particular building or pavement element was correctly assigned to only parcel. Second, the parcels were subdivided and the problem was fixed by merging one or more parcels. Whenever there was doubt about whether or not two or more parcels had the same owner, the parcels owners were verified on the Cleveland County Assessor website. If two or more parcels were joined, then resulting area and associated attributes (e.g., building count, building area, and pavement area) were recalculated to ensure that they were as correct as possible. Once the impervious area was calculated, the pervious area was calculated by subtracting the impervious area from the total parcel area. Figure 20 shows two residential parcels with the buildings and pavements, and the calculated areas.

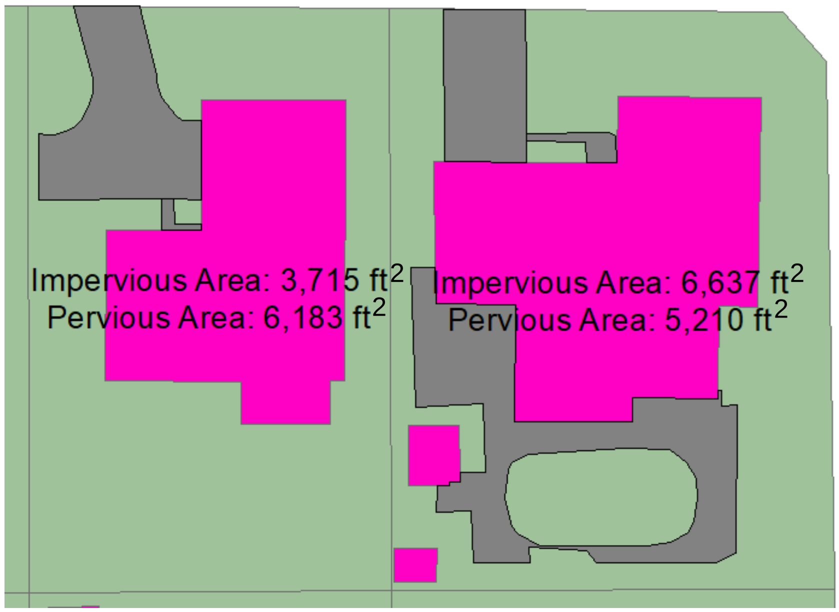

Figure 20. Example of Parcel Processing Results

Finally, each parcel within approximately 200 feet of a water main was selected as a property in the distribution system and exported as a separate shapefile. From the water distribution parcels, the various categories were exported as separate shapefiles:

- Residential Parcels (29,377 parcels)
- Commercial Parcels (1,242 parcels)
- Industrial Parcels (460 parcels)
- Mobile Home Parcels (5 parcels)
- Apartment Parcels (200 parcels)

- School Parcels (19 parcels)
- OU Parcels (45 parcels)
- Park Parcels (86 parcels)

The parcels that were not coded with a zone code (such as right-of-way), medians, and a few parking islands were not exported. While some parcels were unintentionally omitted this way, the method correctly identified the vast majority of parcels.

Once individual shapefiles were created for the various demand categories and a few subcategories, the census tract shapefile was spatially joined with the Residential, Mobile Home, and Apartment Parcels shapefiles, with each parcel receiving average number of people per household. The number of residential parcels per census tract was checked against the number of households in each census tract to verify that the residential parcels were coded more or less correctly.

In summary, the following parameters were determined for each parcel:

- Total area (square feet)
- Individual building footprint areas (square feet)
- Area of paving (square feet)
- Area of pool (if one exists on the property)
- Total area of building footprints (square feet)
- Total impervious area (square feet)
- Total pervious area (square feet)
- Average population per household (for Residential, Mobile Home, and Apartment Shapefiles)

The demand for each parcel is calculated based on the information calculated above. Once the demands were assigned to each parcel, the total demand from the GIS parcels was checked against the calculated total demand to ensure that the demands were correctly allocated between the calculations and the GIS parcels.

The final parcel processing step assigned the parcel demands to specific junctions in the hydraulic model. Once the specific demands per parcel were calculated, the junctions from the hydraulic model were imported into GIS. Each parcel with a demand was assigned to the nearest pipe and then to one of the junctions on either end of the pipe, with checks to ensure that parcels were assigned to a pipe and a junction on the correct street. Most of the parcels were correctly assigned, though there may be some error involved due to the high number of parcels. The individual parcels were then combined based on the assigned junctions to create “service parcels”. This allowed the demands to be imported into the hydraulic model much faster than if the individual parcels were imported.

Residential Parcel Demands

Based on processing the parcels as described above, Norman has approximately 30,400 single-family residential parcels in the distribution system service area. Norman is mostly composed of residential parcels, which spread across the city and particularly along the entire eastern side and central part of the distribution system, as shown in the figure below.

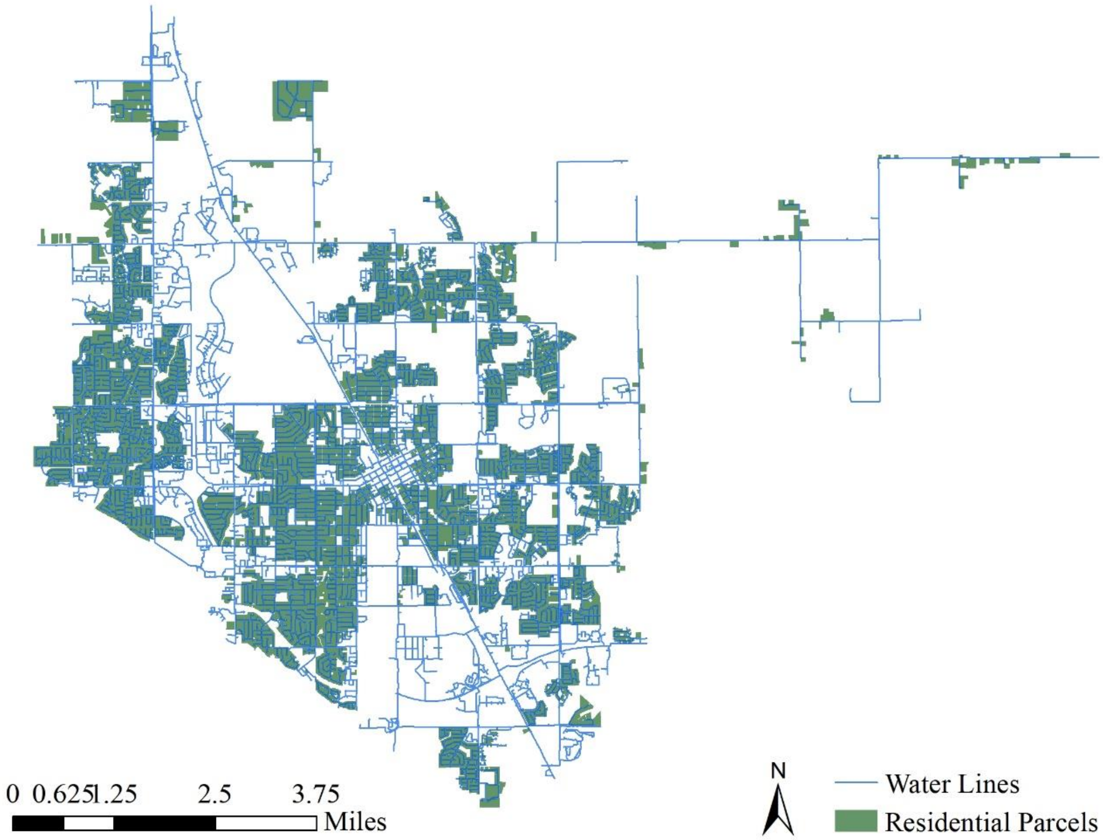

Figure 21. Locations of Residential Parcels

Each parcel represents one household with two separate demands: a potable baseline demand based on population and a nonpotable demand based on pervious area. The potable demands are based on per capita demand using census data to estimate the number of persons per household and the nonpotable demands are based on the pervious area per parcel. All indoor household demands were categorized as potable, excluding toilet flushing.

In the GIS data, there are approximately 30,400 single-family homes in the city of Norman. Some error is associated with this because some lots were omitted due to the building footprints being smaller than 700 square feet (sf) or there were no buildings located on the property. Additionally, some lots may have been mislabeled as commercial lots instead of residential due to how the Zones shapefile aligned with the individual parcels when doing the spatial join. It is important to note that, although the number of single homes determined from GIS is less than the number of residential accounts, there are apartment complex meters that are billed as residential accounts instead of commercial. By including the number of residential apartments with the number of single-family homes, the number of residences determined with GIS is comparable to the number of residential accounts. This was further verified by comparing the number of residential parcels to the approximate number of households per census tract.

Residential Baseline Demands

Population determines the residential baseline demand for each residential parcel. To break this down, there are three steps:

1) Census data is used to estimate the population living at each residential parcel, apartment parcel, and mobile home park.
2) The residential per capita demand is calculated based on the residential baseline demand and the residential population in residential parcels instead of in apartments or mobile home parcels.
3) The residential per capita demand is applied to each residential parcel, multiplied by the average number of people per household.

For the first step, the residential population must be separated from the "commercialized population", that is, the population living on parcels with commercial water meters. This includes anybody living in large apartment complexes, group housing (such as retirement homes), and mobile home parks. According to the 2010 Census, the total population of Norman was 110,925 and the estimated service population was 97,614 people. With a 1.3 percent growth rate, the 2012 population is estimated at 101,695.

The commercialized population is estimated first. The mobile home population is calculated by applying the average household size for the census tract by the number of buildings in mobile home parcels. According to the GIS data, there are 981 units in the five mobile home parks, each located in a different census tract. The average household size for the census tracts ranges from 2.1 people per household to 2.56 people per household. When the average household sizes are multiplied by the number of buildings in the mobile home parks, this equals a total population of 2,238 people in mobile home parks.

The census tract data also includes information about the population living in group quarters, the number of group quarters, and the number of multiunit structures. This information is useful for estimating the apartment population. According to Census Quick Facts, there were 48,284 households in Norman in 2012 and 15.2 percent of them were multiunit structures. This equates to 7,339 multiunit structures in Norman. There are 49 large apartment parcels in the GIS data, with a combined 731 buildings on them. Assuming that there are approximately four units per apartment building (a rough assumption from personal observation visiting apartment complexes in Norman), each with an average of 2.23 people per apartment (US Census, 2013), the large apartment complexes have an estimated 6,521 people living in 2,924 apartments. According to the census data, there are 4,415 remaining apartments on smaller apartment parcels.

Finally, there are 6,860 more people living in group quarters, according to the census data. This includes people living in retirement homes, institutions, and the

Veterans Affair Center in Norman. Combined, the commercialized population equals 15,619 people. This leaves 86,076 people living on residential parcels in Norman in 2012. Briefly, the various populations are summarized in the table below.

Table 25. Calculated 2012 Residential and Commercialized Populations

Population Category	Population
Commercialized Population	
Mobile Home Parks	2,238
Large Apartment Complexes	6,521
Group Housing Population	6,860
Total Commercialized Population	15,619
Residential Population	86,076
Total Service Population	**101,695**

The second step for the residential baseline demands calculates the residential per capita demand using the residential baseline demand of 7.51 MGD (as calculated in Chapter 3). When 7.51 MGD is divided by the residential population of 86,076, the baseline residential per capita demand is 83.9 gpcd. This value is in line with the United States Geologic Survey (USGS) 2010 survey results, which found that Oklahoma has an average per capita demand of 85 gpcd at residences (Maupin et al., 2014)

The final step applies this number to each residential parcel in GIS, multiplying the baseline residential per capita demand by the average household size.

Residential Nonpotable Demands

The method to calculate the residential nonpotable demands is much simpler than that for the baseline demands. For the residential nonpotable demands, the calculated irrigation demands were divided by the total pervious area in the residential parcels to calculate an irrigation demand by square foot of pervious area. The results are provided in Table 26.

Table 26. Residential Nonpotable Demands by Pervious Area Unit

Demand Scenario	Residential Nonpotable Demand (MGD)	Demand per Square Foot (gpd)
Average Day	1.78	0.0076
Maximum Month	6.33	0.0272
Maximum Day	8.9	0.0382
Peak Hour	26.2	0.1126

According to the GIS data, 95 percent of all residential parcels are less than 0.75 acre, with an average size of 0.22 acres or approximately 9,600 square feet. For these parcels, the frequency distribution in Figure 22 shows that the majority of residential parcels are between 0.20 and 0.25 acres

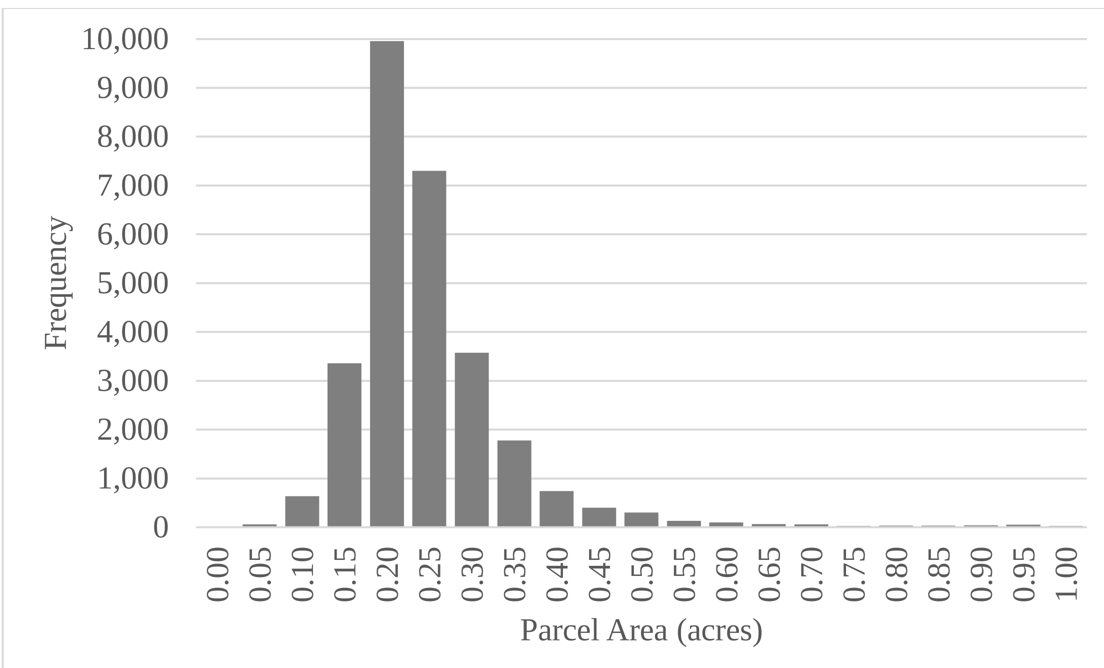

Figure 22. Frequency Distribution of Residential Parcel Sizes

To verify that the demands are reasonable, the average yard irrigation is analyzed for the average day and maximum day demand scenarios. From the GIS data, the average pervious area for 95 percent of all residential parcels is 0.13 acres, or approximately 5,600 square feet. Based on this, the average day irrigation would be approximately 42.5 gpd. Because most people do not irrigate every single day, this value makes sense. During the maximum day demand scenario, the average yard would receive approximately 214 gpd. If the homeowner were to irrigate with a hose that can supply approximately 5 gpm, then the yard would be watered for nearly 45 minutes, which seems reasonable. Thus, the residential irrigation values are accepted.

Commercial Demands

After the parcel processing, there are approximately 1,450 commercial parcels in the GIS database, though 323 of these parcels have no buildings on them. Additionally, mobile home parks and many apartment complexes in Norman are categorized as commercial meters. So, the mobile home parks and apartment complexes are included under the commercial demand. The majority of the commercial parcels are centralized along both sides of Interstate 35 on the western side of the distribution system, along Main Street in downtown Norman, and then along Classen Boulevard on the southeastern side of the distribution system. The mobile home parks and apartment complexes are spread across the distribution system, though many of the largest apartment complexes are located near OU. The spatial distribution of the commercial, apartment complexes, and mobile home parks is shown in Figure 23 below.

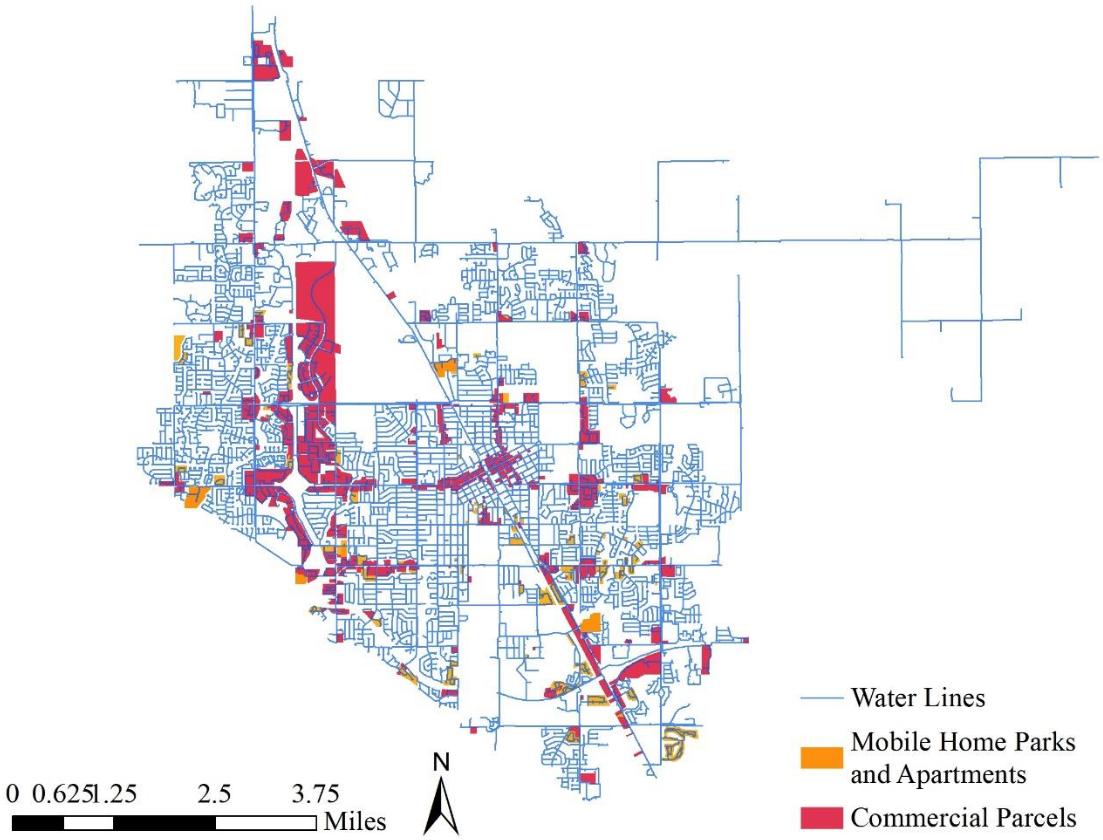

Figure 23. Locations of Commercial Parcels

Commercial Baseline Demands

The commercial baseline demands per parcel are based on the footprint area of the main building (if there is more than one building). Without specific meter demands, one method of approximating the baseline demand involves using the heated building space (Morales et al., 2013). However, neither the heated building space nor the total building space were available, so the footprint area is used instead.

The commercial baseline demand of approximately 1.3 MGD was then divided by the entire square footage of the commercial buildings, which is 13,205,645 square feet. This resulted in a commercial demand per square foot of approximately 0.10 gpd/sf. This demand was then multiplied by the size of the largest building for each commercial parcel.

Commercial Irrigation Demands

The commercial irrigation demands were calculated by dividing the various demand scenarios by the total pervious areas for all commercial parcels to estimate an irrigation demand per pervious square foot. The total pervious area for commercial parcels is estimated at approximately 1,082 acres. Table 27 provides the calculated commercial demand per square foot.

Table 27. Commercial Nonpotable Demands by Pervious Area Unit

Demand Scenario	Categorical Demand (MGD)	Demand per Square Foot (gpd)
Average Day	0.37	0.0071
Maximum Month	1.2	0.0232
Maximum Day	1.78	0.0344
Peak Hour	1.85	0.0357

Industrial Demands

The industrial demands were calculated in nearly the same method as the commercial properties, except that the entire industrial demand was divided by building square footage, with no consideration given to the pervious area on the industrial parcels. According to the GIS data, there are 407 industrial parcels, which are mainly located on the northern and southern sides of the distribution system as shown in Figure 24 below.

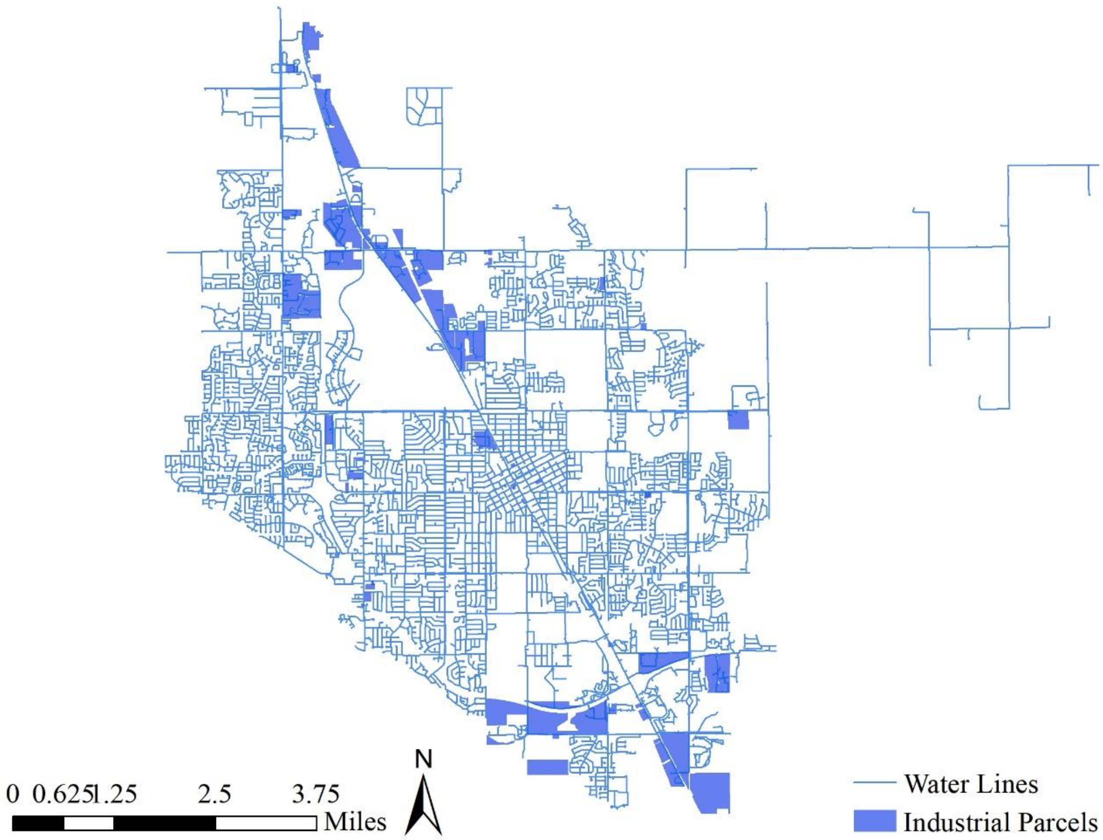

Figure 24. Locations of Industrial Parcels

The entire building space for the industrial buildings equals nearly 6,500,000 square feet. The demand per square foot is calculated by dividing the various demand scenarios by the total building footprint and is provided in Table 28 for the four demand scenarios.

Table 28. Industrial Demands by Building Area Unit

Demand Scenario	Categorical Demand (MGD)	Demand per Square Foot (gpd)
Average Day	0.28	0.0427
Maximum Month	0.41	0.0637
Maximum Day	0.51	0.0782
Peak Hour	1.00	0.1552

Institutional Demands

According to the GIS data, there are 219 institutional parcels and 19 school parcels in Norman. The largest concentration of institutional parcels is at Griffin Memorial Hospital and the Oklahoma Veterans Center, which are on the eastern side of the distribution system. The schools, which includes elementary, middle, and high schools, are spread out across the distribution system, as shown in Figure 25.

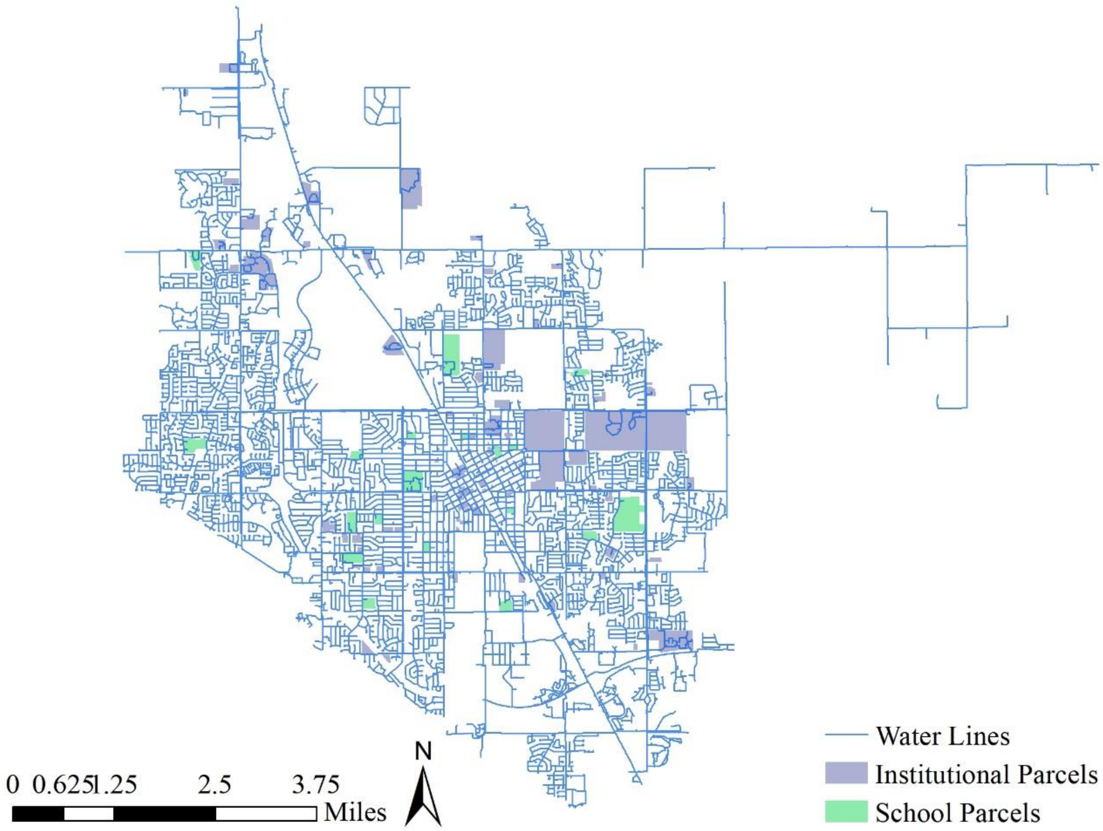

Figure 25. Locations of Institutional and School Parcels

The total building footprint area for the institutional and school parcels is approximately 6,675,000 square feet. The demand per square foot is calculated by dividing the various demand scenarios by this area, with the results provided in the table below.

Table 29. Institutional Demands by Building Area Unit

Demand Scenario	Categorical Demand (MGD)	Demand per Square Foot (gpd)
Average Day	0.11	0.0165
Maximum Month	0.16	0.0245
Maximum Day	0.20	0.0301
Peak Hour	0.40	0.0597

OU Demand

After the parcel processing, there are 45 OU parcels. These are all centered at the OU campus in south central Norman, as shown in Figure 26 below.

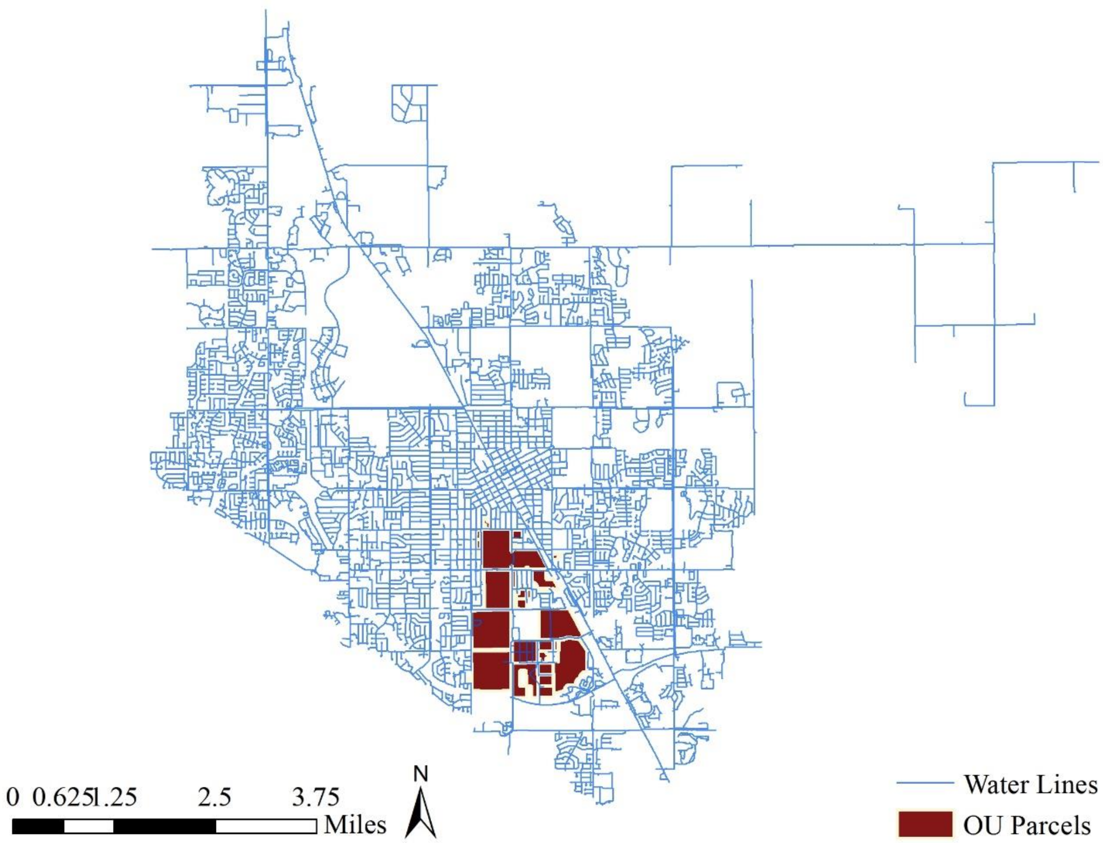

Figure 26. Location of OU Parcels

Total footprint area for buildings on OU parcels is approximately 4,000,000 square feet. This number is divided into the total demand for OU to estimate the unit demands by building footprint area. The results are provided in Table 30 below.

Table 30. OU Demands by Building Area Unit

Demand Scenario	Categorical Demand (MGD)	Demand per Square Foot (gpd)
Average Day	0.24	0.0591
Maximum Month	0.36	0.0880
Maximum Day	0.44	0.1081
Peak Hour	0.87	0.2145

Sprinkler Demand

The sprinkler meters are spread out across Norman, but the exact locations are unknown, so without addresses or some way to geolocate them, their locations must be based on some other method. So, the sprinkler demands are applied to the park parcels throughout Norman. According the GIS data, there are 86 park parcels in Norman, with a total calculated pervious area of 931 acres. The parks dot Norman, with larger parks at Westwood Golf Course, the ball fields at Griffin Memorial Park, Reaves Park, and the Sutton Wilderness Park, as shown in Figure 27 below.

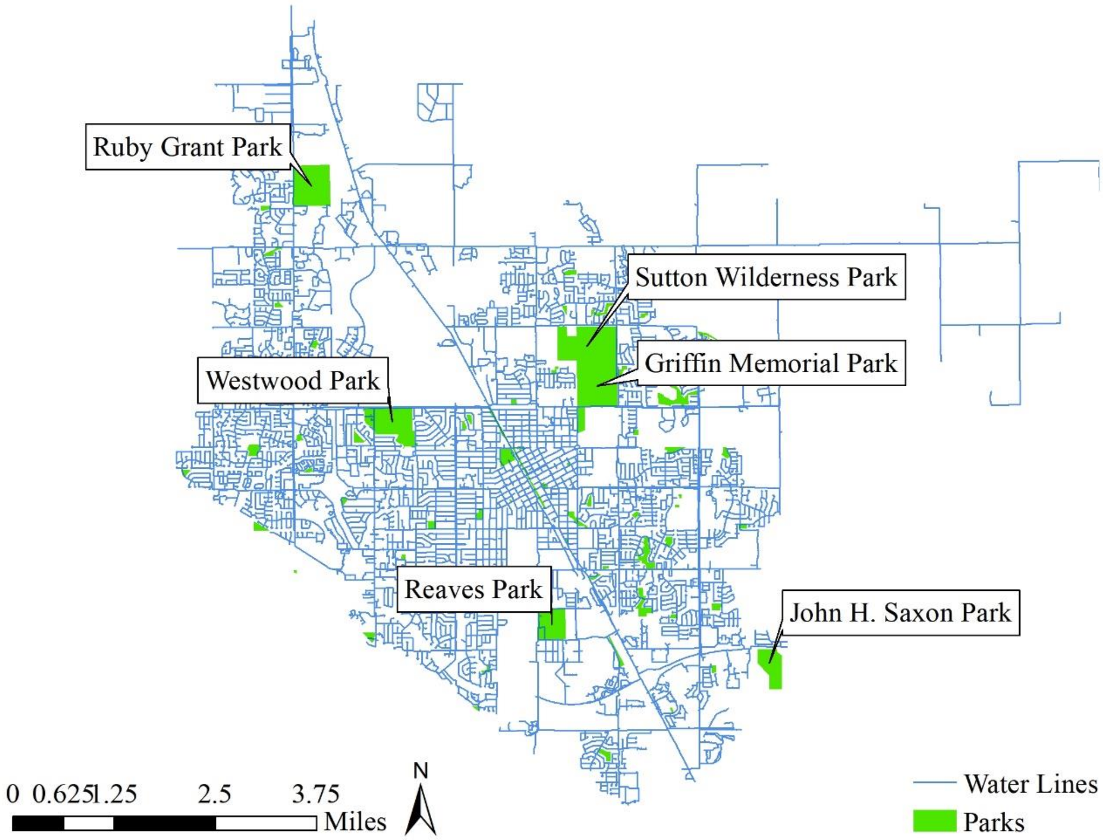

Figure 27. Locations of Park Parcels

The parks cover approximately 931 acres within the Norman distribution system. As with the other parcel types, the categorical demands are divided by the total pervious areas to estimate the irrigation demands, though Sutton Wilderness Park and Griffin Park are not included because Sutton Wilderness Park is not irrigated and Griffin Park has its own irrigation supply. The results for the remaining parks are shown in the table below.

Table 31. Sprinkler Demands by Pervious Area Unit

Demand Scenario	Categorical Demand (MGD)	Demand per Acre (gpd)
Average Day	0.56	602
Maximum Month	0.83	896
Maximum Day	1.02	1,101
Peak Hour	2.03	2,183

Water Loss

The water loss demand component differs from the other demands in that it is applied over the entire distribution system. The hydraulic model of the single distribution system has 17,116 junctions. The water loss is evenly distributed to every junction in the distribution system. Of note, many junctions are located very close together, but the error associated with this is potentially negated because the total number of junctions would decrease if some of these junctions were merged together, but then the water loss per junction would increase commensurately. Thus, the water loss is applied to every junction across the distribution system and the specific value for the demand scenarios is provided in Table 32.

Table 32. Water Loss by Junction

Demand Scenario	Categorical Demand (MGD)	Water Loss per Junction (gpd)
Average Day	0.4049	23.7
Maximum Month	0.6033	35.3
Maximum Day	0.7410	43.3
Peak Hour	1.4699	85.9

Allocating 2062 Demands

By 2062, the population is expected to increase by nearly 100,000 people. While some of this growth will occur inside the current distribution system, large portions of Norman remain undeveloped and have been designated for future development.[4] The designated growth areas were developed into a future land use map for the Norman 2025 Plan, recreated in the figure below.

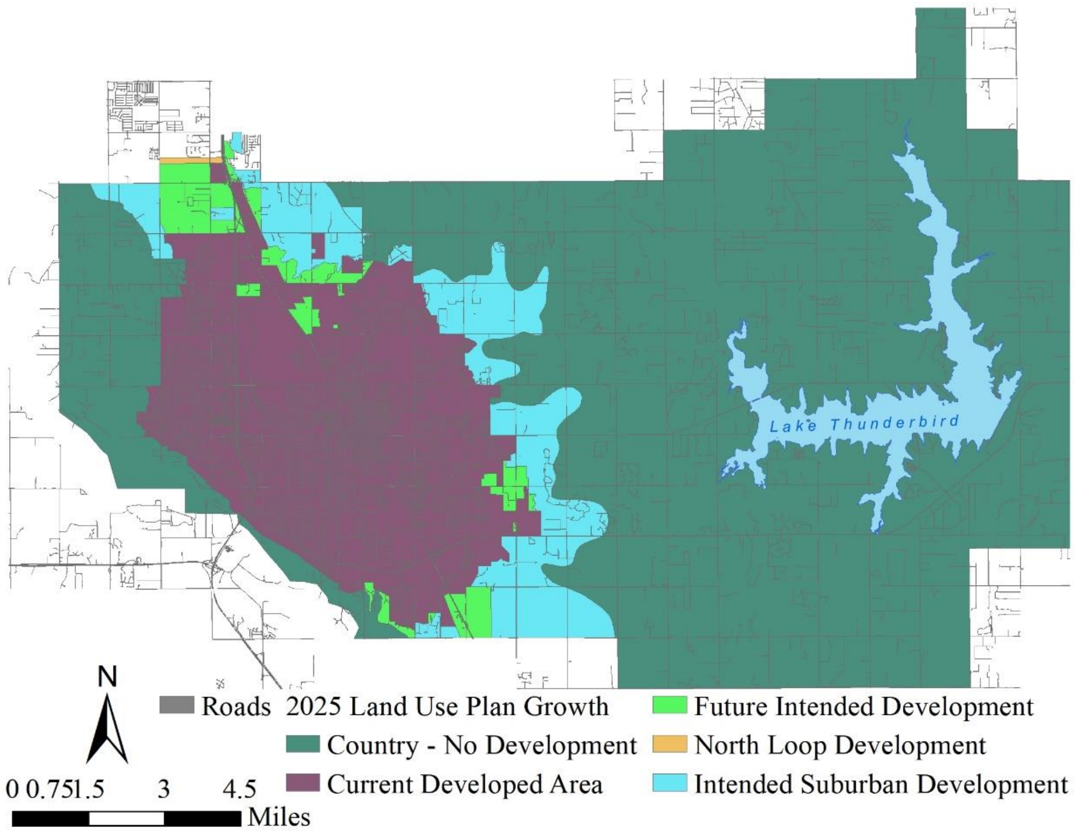

Figure 28. Norman 2025 Future Land Use (Clarion Associates Team 2004)

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[4] In fact, between 2012 (the year with the most data) and 2017 (when this is research is being finished), the Norman population has already begun both in-filling and growing outwards. Many of the smaller houses near the OU campus have been replaced with small 4-unit apartment complexes and there have been several new neighborhoods have been developed on the edges of the main part of Norman.

The growth areas identified in the land use map will be used to determine where the future water lines will go for the distribution area. Individual neighborhoods are not identified, as that is beyond the scope of this project and the details are better suited for urban planning instead of a civil engineering thesis. Moreover, experience shows that skeletonizing the system does not affect the overall model fidelity. For simplicity, it is assumed that all demand types except for residential and commercial will remain within the current distribution area – whether or not this is accurate. Thus, approximate areas will be only identified for future residential and commercial growth.

As Norman continues to grow, three main assumptions affect the future demand in the distribution system for this research project. The first assumes that the service population remains at approximately 88 percent of the total population. This assumption means that development in Norman's rural areas grows consistently with the distribution system. The second assumption is that the 2012 per capita demand will remain consistent at 135 gpcd. As discussed in Chapter 3, the total population of Norman is estimated to increase by approximately 83 percent and the service population will increase by nearly 86,500 people. When the per capita demand is multiplied by the 2012 and 2062 service populations, the calculated 2012 ADD is 13.7 MGD and the 2062 ADD is 26.7 MGD. The population increases and future demands are summarized in Table 33 below, with the differences between the two years.

Table 33. Population and ADD Differences between 2012 and 2062

Year	Total Population	Service Population	Per Capita Demand (gpcd)	Average Daily Demand (MGD)
2012	115,562	101,695	135	13.7
2062	212,840	187,299	135	25.3
Difference	97,278	85,605	-	11.6

For this research, the expected area needed to accommodate the future residential and commercial demand is calculated two ways. The first way assumes a constant demand per area and estimates the future area based on the amount of land needed to accommodate the future demand. The second way assumes a constant population density and estimates the future land area based on the amount of land needed to accommodate the future population.

To perform the first method, the 2012 residential and commercial demands per area must be calculated. In 2012, the distribution area covered approximately 40 square miles. With a residential average day demand of 9.28 MGD, the residential demand per square mile is approximately 235,500 gpd. With a commercial average day demand of 1.69 MGD, the commercial demand per square mile is approximately 42,900. These two demands were summed together, for a total demand per square mile of approximately 278,400 gpd.

The 2062 residential and commercial demands are 18.51 MGD and 3.37 MGD, respectively. With the increased demand, the distribution system would have to double in size to 72 square miles – a 39-square mile increase that would entirely fill the Norman city limits.

Method two approximates the increased land area needed based on population. In 2012, the service population was approximately 101,695 people. With a service distribution area of almost 40 square miles, this equates to 2,581 people per square mile. By 2062, the service population is expected to grow to 185,699 people. Assuming a constant population density, this equates to a total area of 72 square miles – a 33-square mile increase.

The demand method has a more conservative growth area, so the proposed growth area will be approximately 38 square miles. Knowing this area, the additional residential and commercial demands will be evenly allocated across the expanded distribution system based on the final number of new junctions in the hydraulic model needed to accommodate this growth area. The projected growth area used for this research follows the projected growth for the Norman 2025 Land Use Analysis and is shown in Figure 29 below.

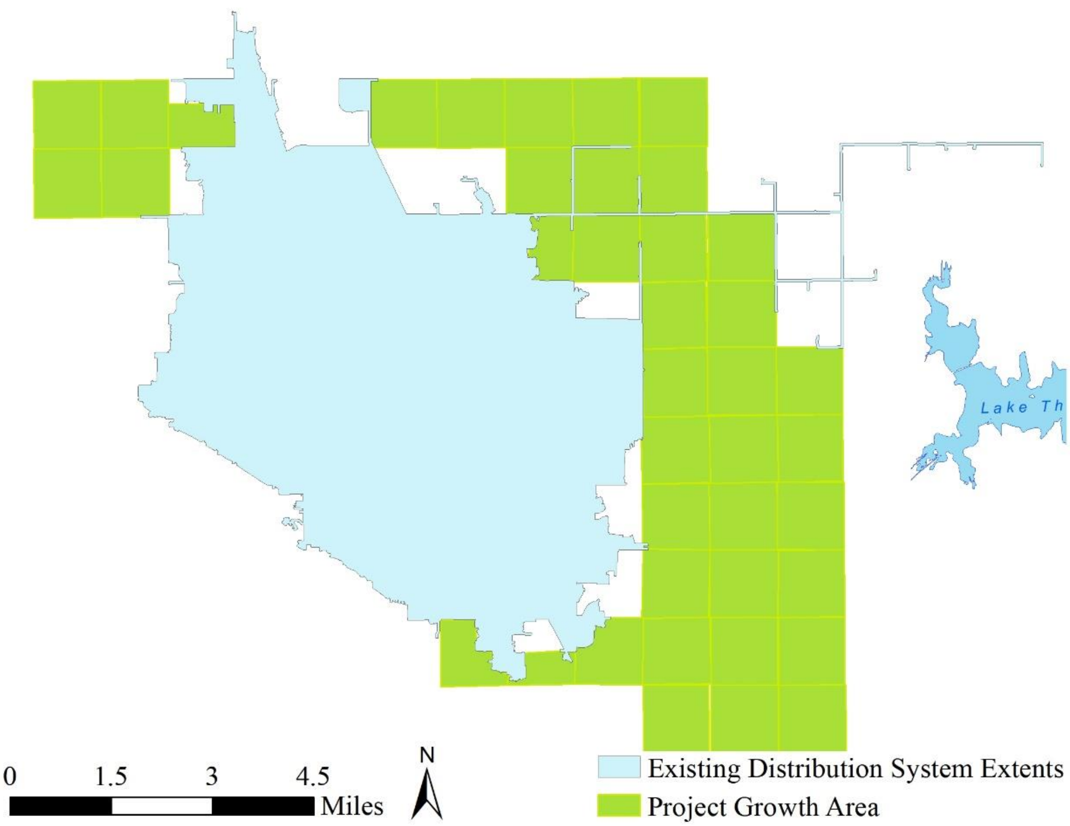

Figure 29. Norman 2062 Distribution System Growth Area

Creating the 2012 Single Distribution System Hydraulic Model

The 2012 hydraulic model was created in Bentley® WaterGEMS using shapefiles from the Norman interactive GIS map (City of Norman 2013). Shapefiles available for download included:

- Water Lines
- Hydrants
- Storage
- Wells

Fittings and valves would have added an unnecessary level of complexity. In WaterGEMS, minor loss coefficients for the pipes account for the various fittings; however, this research did not specifically include minor loss coefficients because the scale was so large. Instead, the minor loss coefficients were accounted for by lower friction coefficients on the water lines. Based on personal work experience, this is standard practice for large water models because it would take much more effort to estimate the minor losses for every pipe. The Meters shapefile was omitted because there were only 137 meters in the shapefile with very limited information about each meter, so the shapefile did not provide enough useful information to be included. The Pumps shapefile had only one pump, located at Westwood Park, but no other information besides the location. Finally, there were several attempts to load the Valves shapefile into WaterGEMS, however a friction loss curve is required for each valve and there were unknown for all the various valves in the distribution system. As a result, the potential minor loss associated with the valves was included in the friction coefficients on the water lines.

The useful shapefiles were analyzed in ArcMap and then brought into WaterGEMS using ModelBuilder, a tool in WaterGEMS that uses existing GIS data to create a new hydraulic model or update an existing model (Bentley 2013). Elevation data was obtained from OKMaps Oklahoma, an interactive GIS map that provides a wide variety of GIS data for the entire state of Oklahoma. This research used the LiDAR elevation data for NRCS 2m Digital Elevation Model (DEM) Bare Earth Contours to assign the ground elevations to the junctions, fire hydrants, wells, and water towers. WaterGEMS used T-Rex, a tool that interpolates specific elevations based on an elevation raster file. Ground elevations are used instead of the actual junction elevations for two main reasons. First, the specific depth of each junction is unknown, so the depths would have to be assumed. Minimum ground cover in Oklahoma is three feet below the surface (ODEQ 2012), but the exact depths are unknown because fittings could be installed much deeper than that to avoid other buried pipeline or to get around an obstacle. Second, there is little pressure difference with a three-foot elevation difference; in fact, "an elevation accuracy of 5 ft. is adequate for most nodes" (Bentley 2013). Third, the water must be brought to the surface for use, so this indicates water pressure near the point of use.

Fire Hydrants

The fire hydrants shapefile shows 5,228 fire hydrants spread across the distribution system, as shown in the figure below.

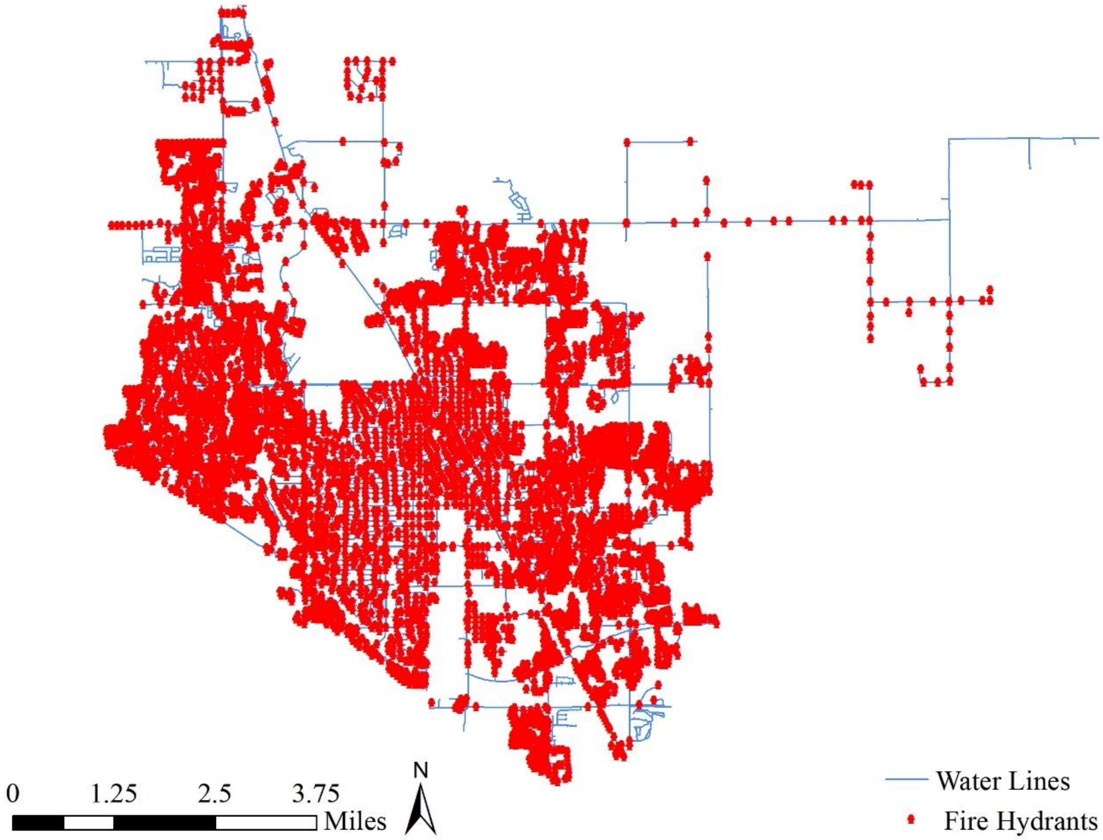

Figure 30. Fire Hydrants in Distribution System

The shapefile included attributes for:

- Date Installed
- Status (active or abandoned)

Only the active hydrants were imported into the hydraulic model and no attributes were assigned, such as elevation or demand. Because the shapefile did not include elevation data, the two-meter contours were used to assign ground elevations to the hydrants with the TRex tool. The fire hydrants were connected to the distribution system after the water lines were imported, as will be discussed in the section about the water lines.

The fire hydrants are also used for the fire flow analysis. In the fire flow alternative, the minimum pressure for the system is allowed to be 25.0 psi, with a minimum required fire flow of 1,000 gpm. A maximum required fire flow of 5,000 gpm is applied, which ends the fire flow analysis at that particular hydrant.

Water Towers

The water storage shapefiles included five water towers with the name and installation year. The water towers are spread across the distribution system, as shown in Figure 31.

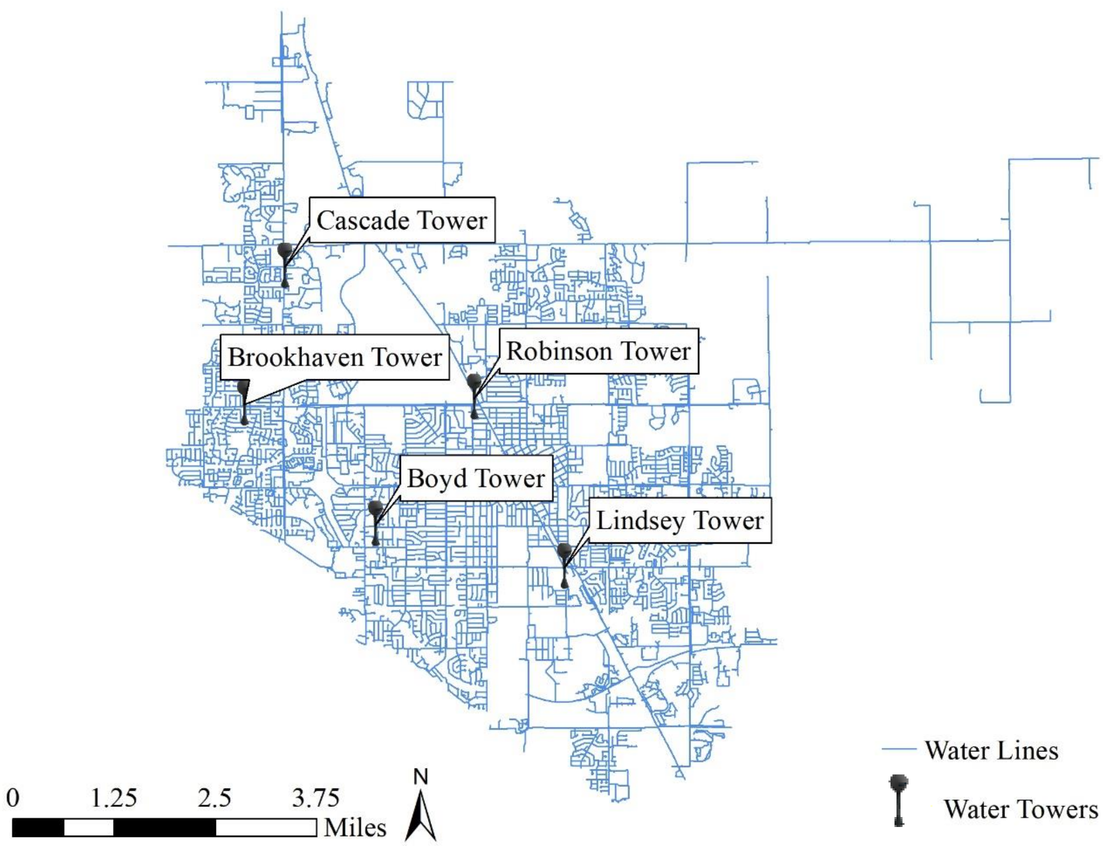

Figure 31. Water Tower Locations

In addition to the shapefile, the NUA provided more specific, operational information for the five towers, provided in Table 35.

Table 34. Water Tower Operational Information

Parameter	Robinson	Cascade	Brookhaven	Boyd	Lindsey
Capacity (MG)	0.5	2.0	1.5	0.5	0.5
Bowl Depth (feet)	39.66	50	42.5	40	48.2
Diameter (feet)	50	82.5	77.5	46.1	42.0
Overflow Elevation (feet MSL)	1315.02	1315	1315.1	1320.21	1312.01
Ground Elevation (feet MSL)	1190.19	1189.5	1191.02	1160.21	1153.22
Height above ground (ft to overflow)	124.86	125.5	124.08	160	158.79
Gauge reading at overflow (psi)	54.1	54.3	53.7	69.3	68.8
Height above ground (overall)	135.27	130.01	136.03	162	-
Gallons per foot capacity	12,607	46,710	35,294	12,500	10,373
Top elevation	1325.46	1319.51	1327.05	1322.21	1153.22
Calculated Volume (gal)	500000	2335500	1500000	500000	500000
Top elevation (Calculated)	1354.68	1365	1357.6	1360.21	1360.21

ModelBuilder imported the water towers into the hydraulic model as tanks and the following parameters for each water tower were set:

- Label
- Is Active? True
- Operating Range: Elevation
- Elevation (Base)
- Elevation (Minimum)
- Elevation (Initial)
- Elevation (Maximum)
- Ground Elevation
- Inactive Volume

- Section Type
- Diameter

All the above information except for the initial elevation was determined by either the shapefile data, the operational data table, or the elevation contour data. The initial elevations for all the towers was set at 1315.00, which is just below the overflow elevations for the water towers, except for the Lindsey Tower which is not currently in use. The overflow elevations also indicate that the hydraulic grade line (HGL) for the distribution system is 1315 ft. The water towers were connected to the distribution system after the water lines were imported, as will be discussed in the section about the water lines.

Wells

The Wells shapefile contained 47 wells, with one abandoned well, 15 inactive wells, and 31 active wells, as shown in Figure 7. Only the active wells are included for the existing single distribution system and the potable distribution system, which are shown in Figure 32.

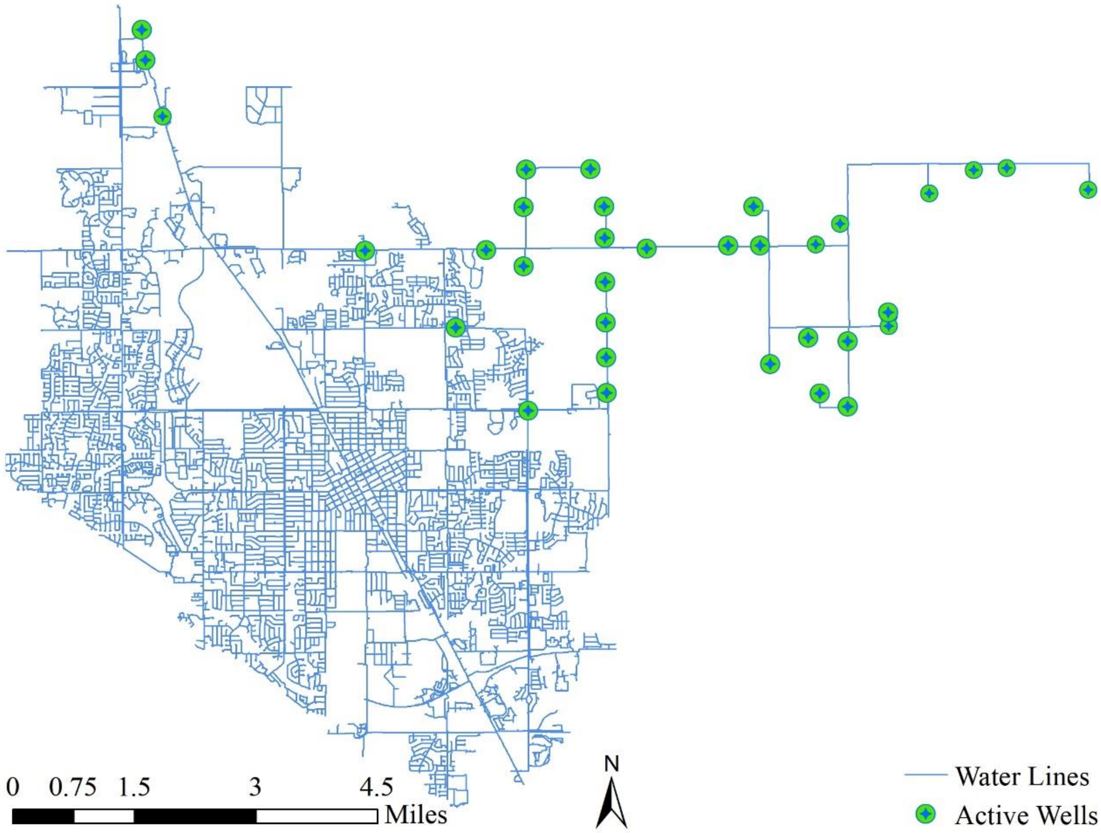

Figure 32. Well Locations

The wells were imported with ModelBuilder as reservoirs. There were several unknowns for the wells, including the depth of the intakes, the drawdown curves, or the pump curves to withdraw water. The one known for the wells were the average flow rates – the active well flow rates were provided in the 2060 Strategic Water Supply Plan (Carollo, 2014) and by the City of Norman (personal email from Chris Mattingly, October 28, 2013). To approximate the actual operating conditions, the elevations were set to 1315 feet to maintain the HGL in the distribution systems and flow control valves were placed on the discharge water lines. The flow control valves artificially limited the flows from the wells to realistic flows; otherwise, the flow from the wells would have provided a limitless water supply to the distribution system. The elevations for the flow control valves were set to a relatively arbitrary elevation that was between 1315 feet and the elevation of the junction at the other end of the water line, often an elevation that WaterGEMS linearly interpolated between these two boundary conditions. The flow control valves were set to the flow rates for each of the wells, as shown in Table 35.

Table 35. Flow Rates for the Wells

Well Number	Status	Flow (gpm)
1	Active	161
2	Active	224
3	Active	121
5	Active	146
6	Active	290
8	Active	225
19	Active	191
20	Active	144
33	Active	214
34	Active	162
35	Active	142
36	Active	82
37	Active	120
38	Active	189
39	Active	197
40	Active	168
41	Active	179
43	Active	173
44	Active	167
45	Active	146
46	Active	216
47	Active	179
48	Active	145
49	Active	202

Well Number	Status	Flow (gpm)
51	Active	150
54	Active	117
55	Active	150
56	Active	120
57	Active	167
58	Active	150
59	Active	325
60	Active	240
61	Active	200
4	Inactive	249
10	Inactive	134
11	Inactive	112
12	Inactive	164
13	Inactive	190
14	Inactive	177
15	Inactive	215
16	Inactive	143
18	Inactive	136
21	Inactive	144
23	Inactive	160
25	Inactive	125
31	Inactive	159
32	Inactive	182
HP3	Inactive	160

The wells were connected to the distribution system after the water lines were imported, as will be discussed in the section about the water lines.

Water Lines

The water lines constitute the backbone of the entire water distribution system and connect all the various elements, including wells, water towers, fire hydrants, and the end users. The original shapefile contained 24,216 water lines, as shown in the figure below.

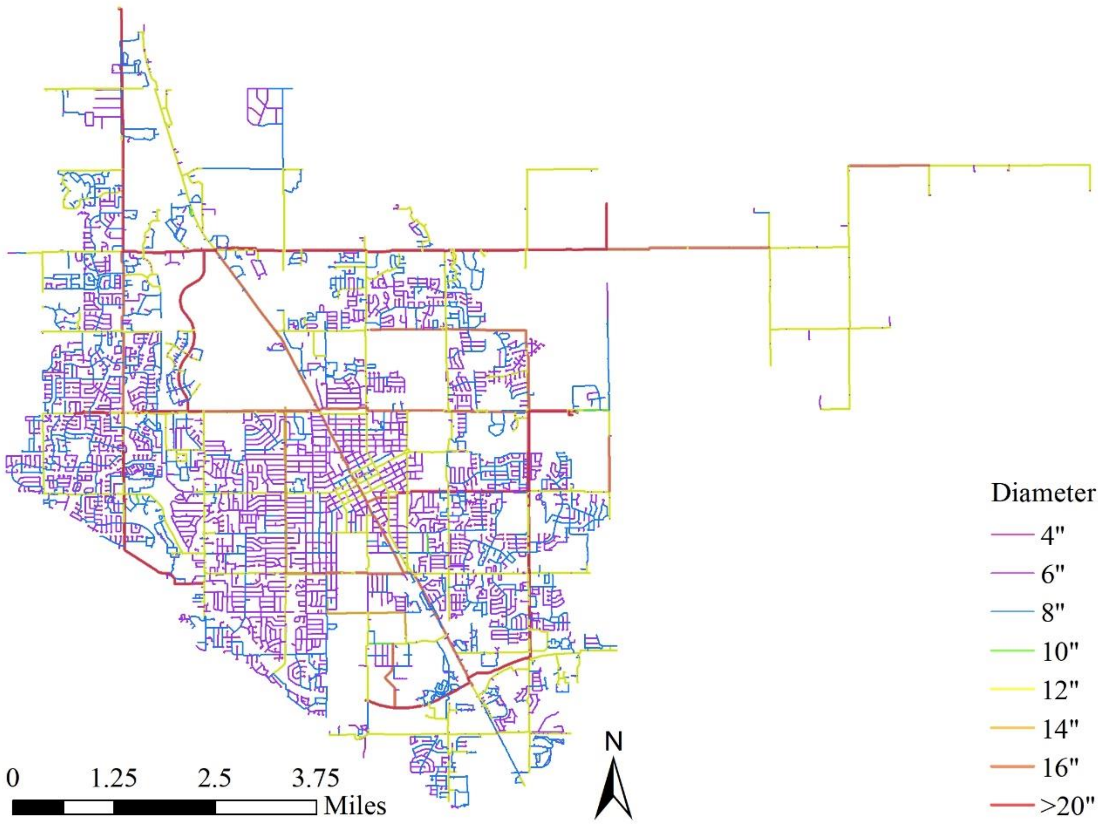

Figure 33. Water Lines

The water lines were imported into the hydraulic model last because ModelBuilder determines connectivity based on the elements in the distribution system. In order to connect the fire hydrants, water towers, and wells to the distribution system, they had to be imported prior to the water lines so that ModelBuilder would connect the water lines to these elements.

The attributes for the water lines included:

- Diameter (inches)
- Length (feet)
- Date Installed (date)
- Material

The diameters ranged from 0 inches[5] to 30 inches, though the vast majority of water lines were either six or eight inches. The 0-inch diameter were lines were examined to determine if they could be deleted or if the diameters of connecting water lines could be applied to them. In most cases, the diameters of connecting water lines were used instead of the 0 inch diameters. The frequency distribution of the original water line diameters are shown in the figure below.

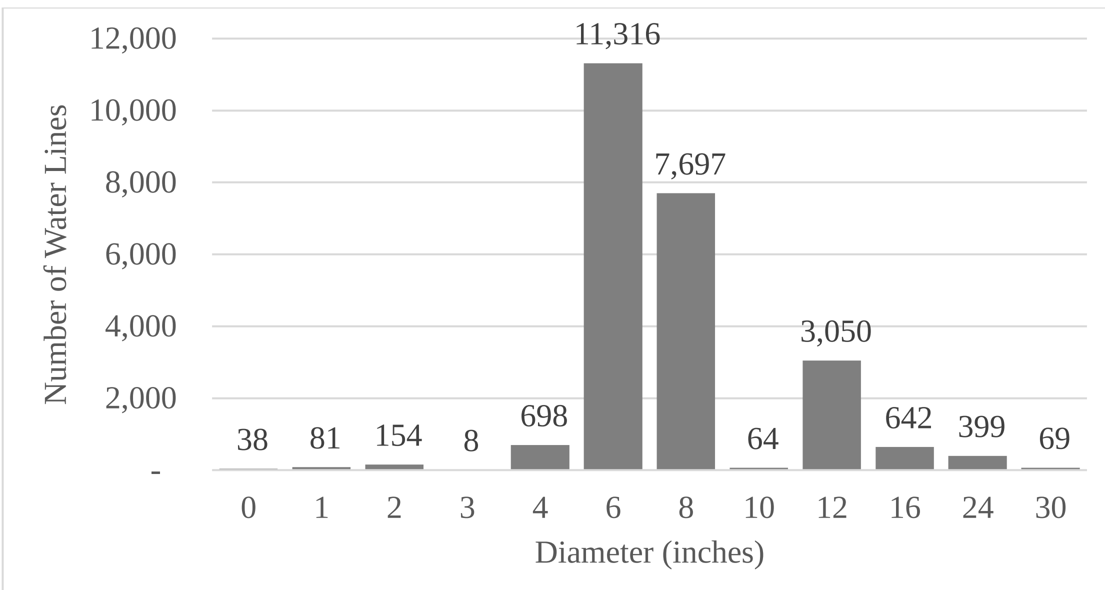

Figure 34. Frequency Distribution of Water Line Diameters

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[5] I could find no explanation for why there were 0-inch diameter water lines in the GIS data.

After the water lines were imported into WaterGEMS, the model was skeletonized to remove all water lines that were four inches and smaller using a built-in tool in WaterGEMS. The smallest water lines were imported into the model and then removed so that the correct connectivity could be preserved in the model. The smallest diameter lines were removed from the model for two primary reasons. First, many of the smallest diameter water lines appeared to be service lines for some users and this research is intended to look at the distribution to the point where the service lines connect to the distribution system instead of where the service lines provide service. Second, the hydraulic models are intended to be an all-pipes reduced model, which "is similar to an all-pipes model with the exception that intermediate hydraulically insignificant nodes along pipe reaches of similar diameter, material, and construction date are dissolved (reduced)" (AWWA, 2012). In other words, smallest diameter pipes provided too great a level of detail for the purposes of this research, so the because the smaller diameter water lines provided too much detail for the purposes of the project.

The installation dates were not initially used to create the hydraulic model; instead, they were used to calibrate the model. Out of the 24,216 water lines, almost a third did not have an installation date. The other 17,252 water lines had installation dates from as early as 1961, with many water lines installed during the late 1970s and early 1980s, and since the mid-1990s, as shown in the chart below.

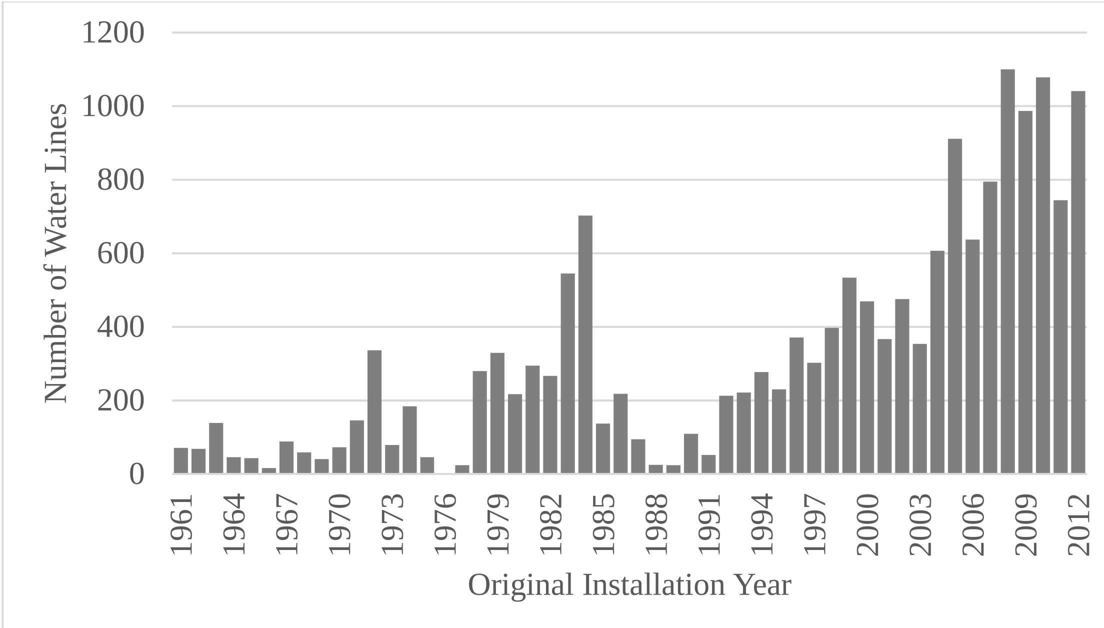

Figure 35. Number of Water Lines by Installation Year

The materials included nine different materials, though the predominant material was polyvinyl chloride (PVC) pipe and 752 pipes had no pre-assigned material. The nine materials and the frequency distribution are shown in Figure 36 below.

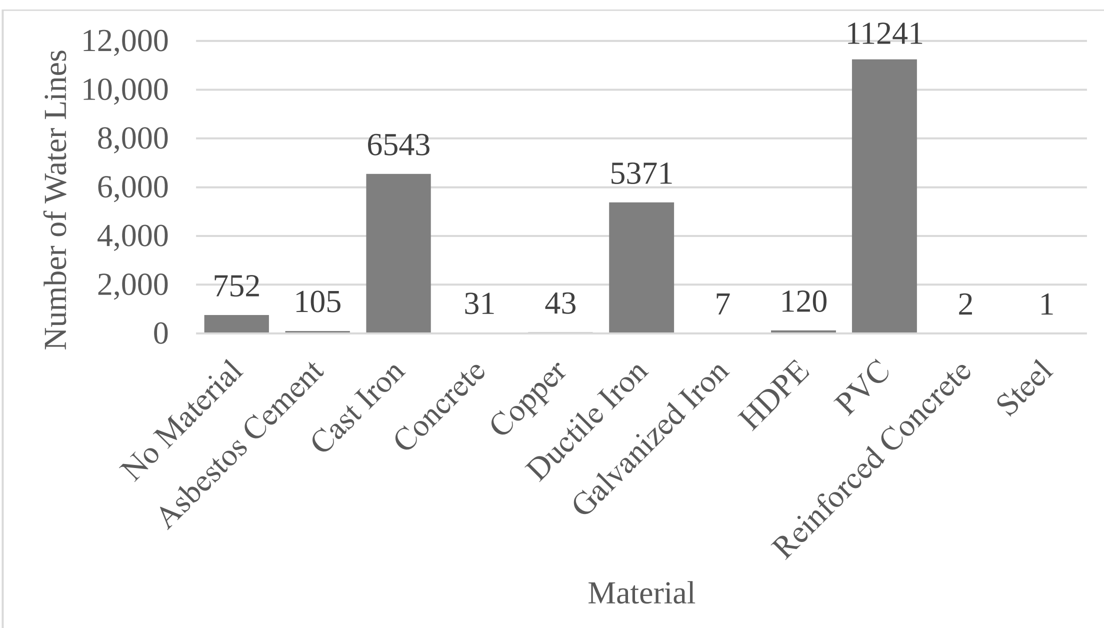

Figure 36. Frequency Distribution of Water Line Materials

WaterGEMS has default friction coefficients for each material, which were applied to all the water lines with specified materials. Some of the water lines did not have a specified material, so these water lines were analyzed similar to the water lines with no diameter, and the materials were manually changed to match the materials of the connecting water lines. The Hazen-Williams calculation method was used for all demand simulations, so the default Hazen-Williams friction coefficients were applied to the water lines, as listed in Table 36.

Table 36. WaterGEMS Hazen-Williams Friction Coefficients for Select Materials (Bentley 2013)

Material	Default Hazen-Williams Coefficient
Asbestos Cement	140
Cast Iron	130
Concrete	110
Copper	135
Ductile Iron	130
Galvanized Iron	120
HDPE	150
PVC	150
Reinforced Concrete	110
Steel	140

Junctions

The junctions in the hydraulic model were not obtained directly from the Norman shapefiles, but were instead created by WaterGEMS when the water lines were imported with ModelBuilder. In the second step, ModelBuilder asks how the user would like to handle missing data, with the option to check a box to “Create nodes if none found at pipe endpoint”. This box was checked when the water lines were loaded using ModelBuilder, which put a junction at the end of every pipe that did not already have an element, such as a fire hydrant, water tower, or well. The junctions created this way did not have elevation data, so TRex interpolated the elevations using the two-meter contours files obtained from OKMaps Oklahoma.

WTP

The Norman WTP did not have its own shapefile, so it was manually created in the hydraulic model. The location is shown in the figure below.

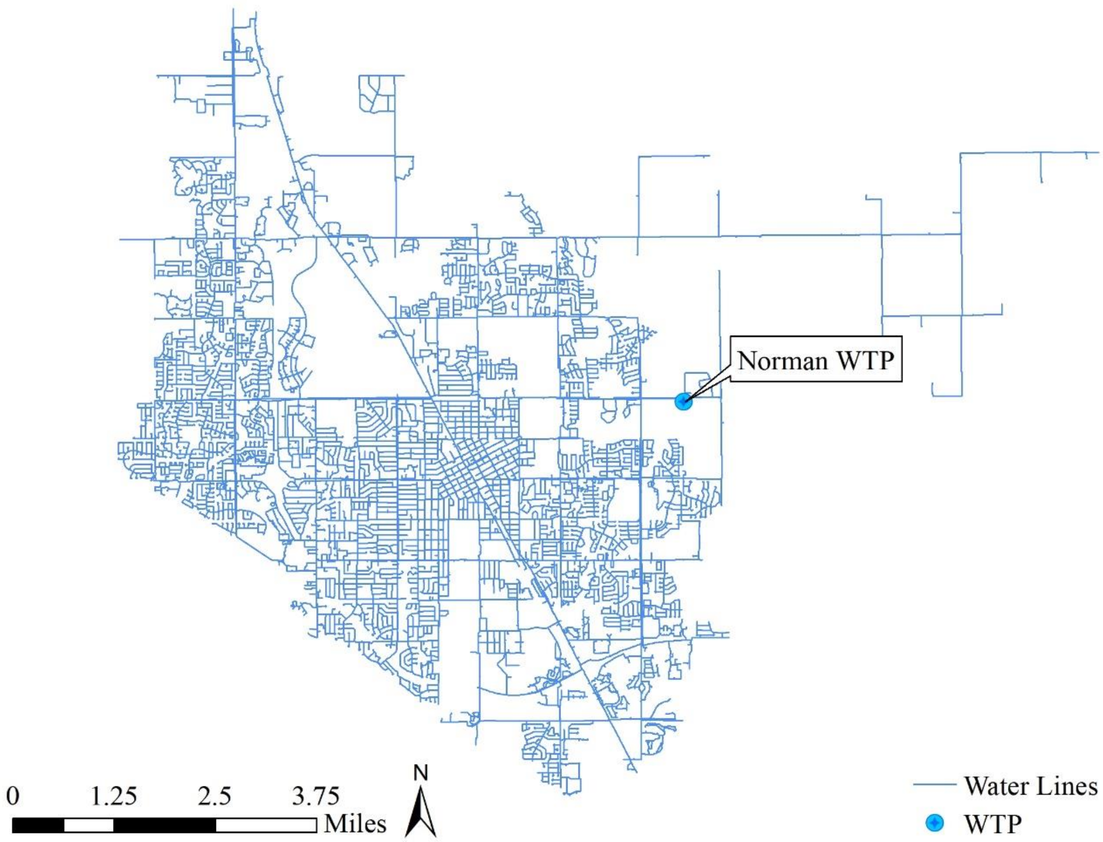

Figure 37. WTP Location in Hydraulic Model

The WTP provides water similar to a reservoir in that it provides water as needed, only limited by the treatment capacity. So, the WTP was simulated as a reservoir, similar to the wells with an elevation of 1315 feet to maintain and preserve the HGL set by the water towers.

Loading Demands

To load the hydraulic model, the demands from Allocating 2012 Demands were entered into the model using LoadBuilder, a built-in tool in WaterGEMS to distribute the demands as intended by the user. LoadBuilder offers multiple methods to load a hydraulic model and, after some trial and error, the method used for this project was Load Estimation by Population. The Load Estimation by Population method assigns a demand per area with a given population density – the load density field. This method uses the service parcels, created by dissolving the individual parcels by a single junction, to identify the demand load for each junction.

Each shapefile was loaded separately, and each subsequent demand was created as a sub-alternative to the previous demand. This was done so that the demands could be kept separate in the hydraulic model, yet still be able to be used together if needed. By keeping the various demands in separate child alternatives, each demand could be checked for accuracy and redone if needed before loading the next demand and without messing up the previously loaded demands.

The residential baseline demands were loaded first using this method with the average number of people per acre (LoadBuilder would internally calculate the total number of people in that service parcel) and applying the 83.9 gpcd rate. The commercial baseline demands were calculated directly in ArcMap in a Commercial Demands shapefile and then loaded second into the model, with a load density demand of 1.0 gpcd, which ensured that LoadBuilder would assign the pre-determined commercial demand without a multiplier. The remaining demands – residential irrigation, commercial irrigation, industrial, institutional, OU, and sprinkler – were all loaded the same way that the commercial baseline demands were loaded.

Steady-State Analysis

A steady-state analysis is used for all of the hydraulic simulations in this research project. The steady-state analyses assess system pressures, HGLs, headloss gradients, and velocities for specific demands at a single instant in time. The single distribution system and the dual distribution system are both simulated with given demand scenarios:

- Average Day Demand
- Maximum Month Demand
- Maximum Day Demand
- Peak Hour Demand

Calibrating and Verifying the Existing Single Distribution System Hydraulic Model

One reason for creating a hydraulic model of the existing distribution system is to ensure that the model behaves as the system does in reality. To do this, the model is calibrated based on hydraulic tests and measurements. Fire flow tests provide an instantaneous snapshot of the distribution system. The static and residual pressures are measured at one hydrant and a nearby hydrant is opened and the flow is measured. For this research, the model is calibrated using historical fire flow test data provided by the NUA. The NUA provided 136 fire flow test results at fire hydrants around the city of Norman from December 2009 through May 2014.

To ensure that the model accurately reflects the real-world system, the model was calibrated by adjusting the pipes' Hazen-Williams roughness coefficients (roughness coefficients). The calibration was performed by adjusting the roughness coefficients based on material and age and then testing the pressures based on flow tests. When the pipes were first imported into the hydraulic model, the roughness coefficients were uniformly set to 130. This did not accurately reflect the conditions, because the materials and ages of the pipes varied. A C_H value of 130 is typical for new cast iron pipes, and a C_H value of 80 is typical for old cast iron pipes (Chin, 2013). It is also lower than the typical values for PVC water lines, which often has a range of 135 to 150 (Chin, 2013). As discussed earlier, the City of Norman mainly used cast iron, PVC, and ductile iron. In the 1960s through the 1980s, the majority of pipes were made of cast iron. Starting in the 1980s, the City of Norman began to switch to PVC.

To test the accuracy of the calibration, historical flow test data was used. The Norman Utilities Authority provided the data, which included the test date, the flow hydrant, the pressure test hydrant, the static pressure readings, the residual pressure, and the flow. The fire flow tests were conducted around the city. The hydrant numbers in the flow test data were matched with the hydrants in the GIS based on identification numbers to determine which ones could be used in the hydraulic model, because some of the flow test hydrants did not have an identification number or address. The matching hydrants are shown in Figure 38 below.

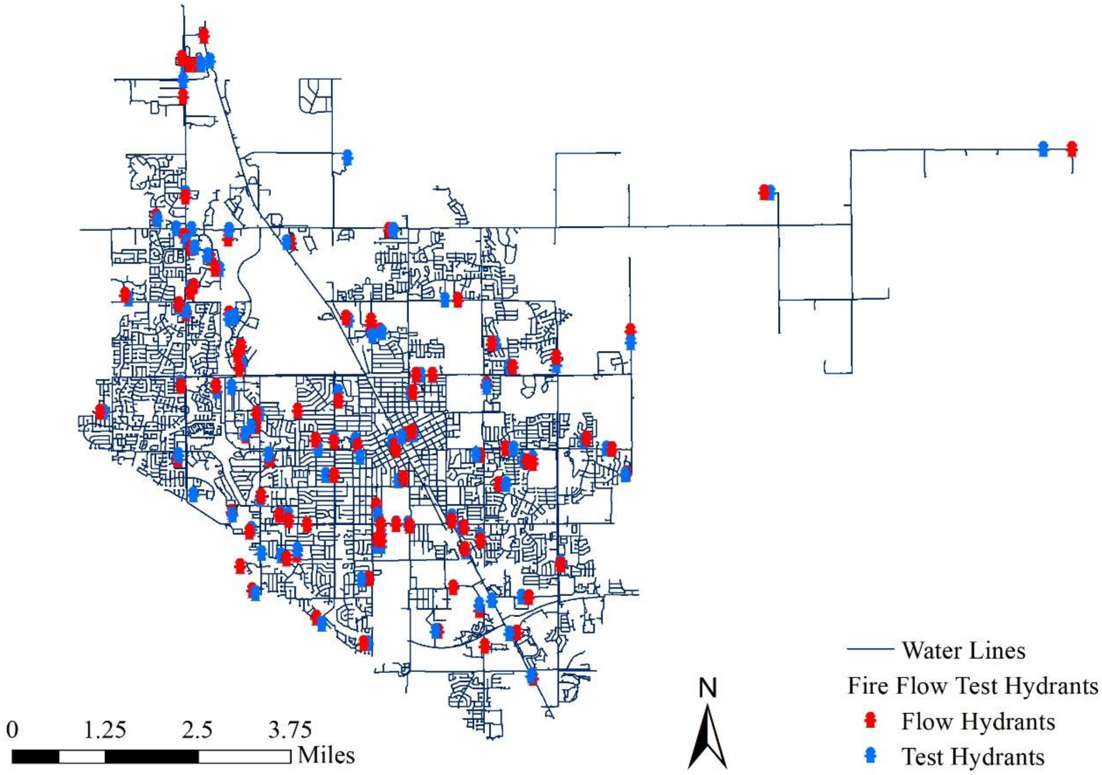

Figure 38. Location of Flow Tests

Once the hydrants were matched to the GIS data, 22 flow test results during March, April, May, and October[6] from 2009 to 2013 were selected to calibrate the roughness coefficients in the hydraulic model. This simulated flow tests during periods of historically average demands. The flow test results were simulated in the WaterGEMs hydraulic model as discrete scenarios and alternatives for each flow test using the 2012 average day demands.

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[6] Flow tests conducted during September and November would have been included, except there were no flow tests from these months that matched with the hydrants in the GIS data.

The initial calibration adjusted the roughness coefficients based on age and material. The roughness coefficients for the older pipes – pre-1980 – were lowered by subtracting 15 from the initial value to more closely simulate the older pipes. The calibration results were fairly close to the flow test results, but not quite close enough to be accepted. The second calibration attempt lowered the roughness coefficients for all the water lines in small increments for all pipes to account for minor loss coefficients because the minor loss coefficients were not included otherwise. This meant that the minor loss coefficients for all fittings and valves were assumed to be included in the roughness coefficient. A lower roughness coefficient also reflects aging of the pipes post-1980. The final calibration determined that the best match for including the minor loss coefficients lowered the roughness coefficients by 15 for all water lines. To evaluate the calibration accuracy, the modeled pressures from the hydraulic model are plotted against the observed pressures from the field with a one-to-one to visualize the results. An accurate model has modeled pressures equal to the observed pressures and all the results are along the one-to-one line. This is very difficult to achieve, so the aim is to have the results close to the one-to one line. The results from the second calibration attempt show that the model typically has slightly higher pressures than what were observed in the field, shown by the dots below the one-to-one line in the figure below.

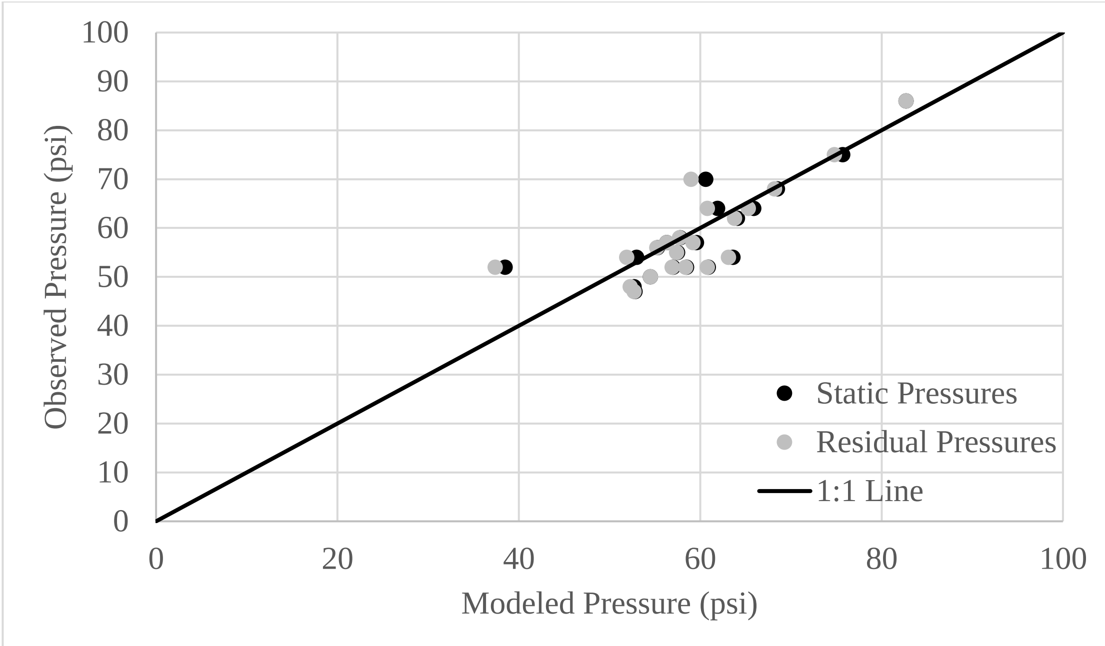

Figure 39. Static and Residual Pressure Results from 2nd Calibration

The results from the 2nd calibration attempt are all clustered near the one-to-one line and have an R^2 value of 0.66. This indicates a decent correlation between the observed pressures and the modeled pressures. Once the model was calibrated, it was validated with data from eight flow tests that were conducted in July and August between 2010 and 2013. The validation results were also plotted with the model results against the observed results and they are shown in Figure 40 below.

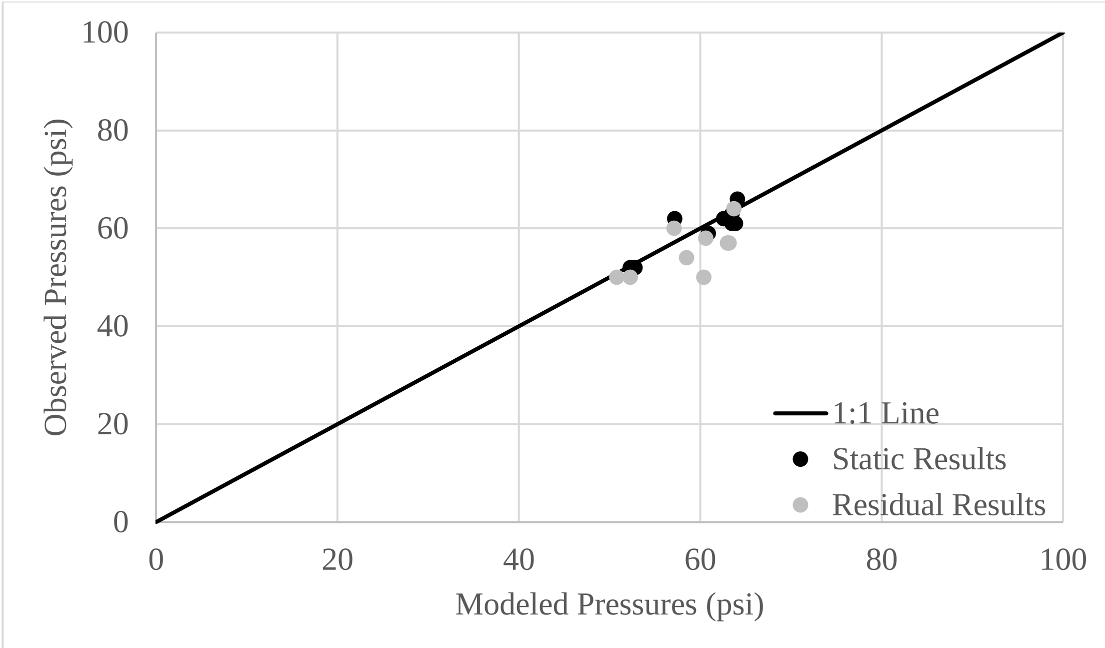

Figure 40. Static and Residual Pressure Results from Calibration Validation

The R^2 value for the static results is higher than for the residual results at 0.77 versus 0.68, respectively. This indicates a better correlation for the hydraulic model when no fire flows are present and both of these numbers are even closer than the calibration results. These numbers show that the hydraulic model performs similarly enough to the existing distribution system. The results from both the calibration and validation are accurate enough to consider the hydraulic model calibrated.

Creating the 2012 Nonpotable Hydraulic Model

A dual distribution system, by default, has two distribution systems: a potable system and a nonpotable system. Communities with a dual distribution system have typically used their existing single distribution system to continue distributing potable water and create a new nonpotable distribution system. For this research, the existing single distribution system will be used as the potable distribution system. Additionally, the existing single distribution system will be the model for the proposed nonpotable distribution system.

Designing new distribution systems inherently contains many choices. First off, the structure for the new distribution system must be chosen. There are three main types of distribution systems: grid, loop, and tree. The grid and loop systems provide redundancy and allow for water to follow multiple paths to any point in the system. The tree system is not recommended due to the lack of reliability and the high potential of stagnant zones in a system (Asano et al., 2007). However, the grid and loop systems are often more expensive to construct then a tree system because more water line is needed to loop or grid the system. Second, the optimal hydraulic design is not always the most practical in practice, such as larger diameters and larger water lines. Thus, there is a balance between accommodating better pressures, more flow, and avoiding unnecessary headloss. Finally, the demands must be divided between the potable and nonpotable distribution systems.

For the nonpotable distribution system for this research, the nonpotable system will closely follow the existing potable distribution system. In fact, the same distribution system will be used for the 2012 nonpotable distribution system to simulate serving all the urban irrigation demand with nonpotable water. Because the nonpotable system will be based on the existing potable distribution system, this means that the nonpotable system will be gridded and looped as much as seems feasible, but there will inherently be some dead-end branches. Second, design criteria will be used to determine approximately the optimal diameters to deliver the required flow at adequate pressures. Third, the only nonpotable demands that will be put on the nonpotable distribution system will be the irrigation demands calculated in Chapter 3.

Nonpotable demands consist of water demands that will not be used for anything that will come into prolonged contact with skin or will be ingested. Examples of nonpotable demands include irrigation, fire flow, toilet flushing, and some industrial uses. As discussed in the Literature Review, irrigation and fire flow are common demands on nonpotable systems. In many distribution systems, fire flow controls the design for the system because the fire flow is so much greater than the municipal demand. For a nonpotable distribution system, fire flow demand would likely be greater than the irrigation demand – just as it often is in single distribution systems. A dual distribution system would also need storage if the system is to be used most effectively. The storage would offset periods of peak supply to coordinate with periods of peak demand.

A dual distribution system must be sustainable in some way and must account for the constant reuse supply, lack of peak supply, potential seasonal demands, high demand peaking factors, and storage. The system operators must decide whether to provide a seasonal distribution system or year-round distribution system. Regardless, the supply oftentimes does not match demand in that the supply is a constant flow all day, every day, with few peaks, whereas the demand is often intermittent with high peaks. This necessitates a balance between the supply and demand, which often includes some sort of storage or demand schedule to ensure that the demands do not exceed the supply. For example, the Fort Sill Army Base in Oklahoma instituted a schedule to accommodate as many demands as possible while not exceeding the total flow.

This section about designing the 2012 Nonpotable Distribution System first discusses key assumptions about the design and operation of the system, and the available nonpotable water sources and how much of the demand they can realistically supply. Then, various design criteria and the decisions about the distribution system that are made with regards to those design criteria, including minimum pressures, water main material, maximum velocity, water line sizing, and storage.

Assumptions

Before discussing design details for the nonpotable distribution system, it must be stated that this research project is not a specific design for the City of Norman; it is a conceptual project to evaluate the feasibility of implementing a dual water distribution system that is aimed at serving residential irrigation customers. Thus, it is assumed that all nonpotable water lines will be placed an adequate distance from potable lines and other utilities. It is also assumed that all water lines – both potable and nonpotable – will be located within easements and right-of-ways, such that only the general location of water lines is known and that the actual paths will not be determined. An actual design would consider vertical and horizontal spacing between potable and nonpotable lines to prevent the nonpotable system from contaminating the potable system, similar to how the spacing between sanitary sewer lines and water lines are regulated. An actual design would also provide more detailed information about the proposed systems. This research project is purely conceptual and could be considered only a 10 percent design in industry-standard terms, so the specific design details will not be determined.

Another assumption is that all precautions are taken to prevent cross connections between the potable and nonpotable distribution system. This could potentially be a large undertaking and would undoubtedly be necessary if a nonpotable distribution system were implemented. For the purposes of this research study though, it is assumed that there are no cross connections between the potable and nonpotable distribution systems.

Water Sources

The water sources for the nonpotable distribution system are effluent from the Norman WRF and the inactive wells (high in arsenic). In Chapter 3, it was determined that the WRF can provide a maximum of 1.7 MGD. The inactive wells have an estimated maximum capacity of 2.1 MGD. The combined total of 3.8 MGD can supply the 2012 average day nonpotable demand of 3.23 MGD, but the daily average cannot supply the nonpotable demands greater than 3.8 MGD. In order to accommodate the other nonpotable demand conditions, storage will be needed to save the excess water for the higher demand times. The nonpotable water sources are shown in the figure below.

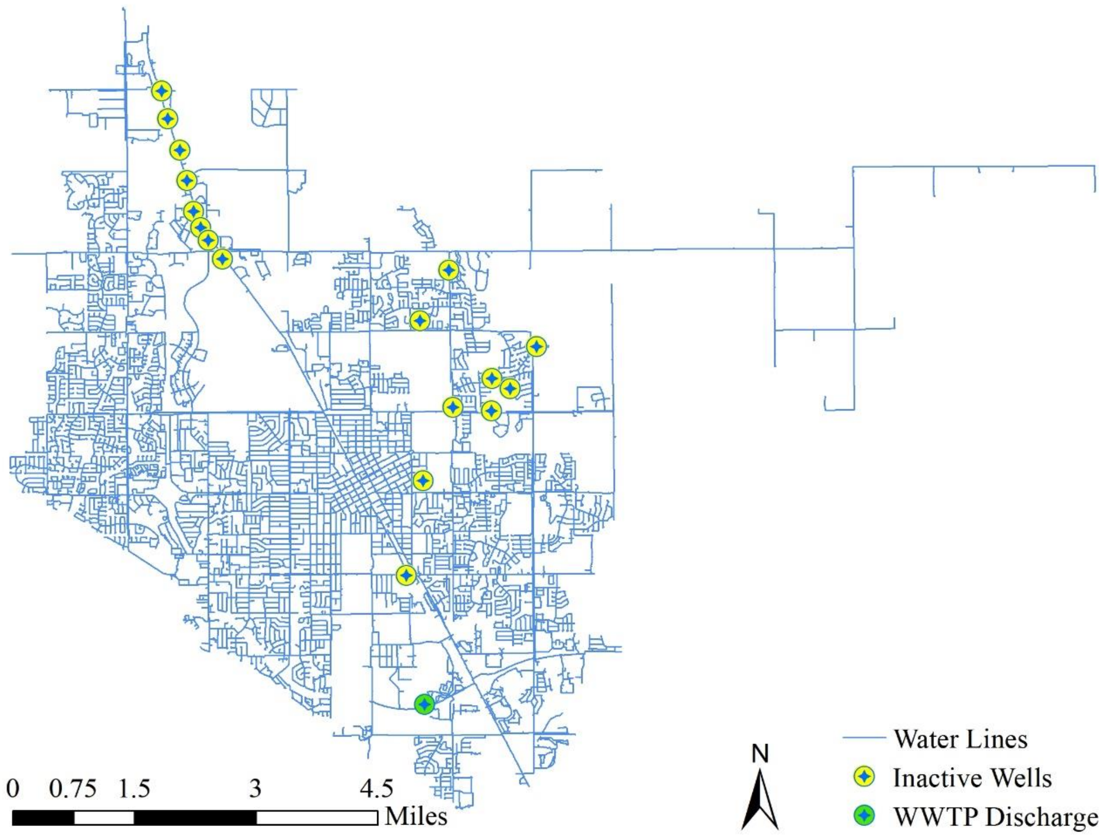

Figure 41. Nonpotable Water Sources

A rough analysis of the nonpotable source capacity and the 2012 nonpotable demands reveals that the total yearly production from both the WRF and the inactive wells is approximately 1,368 MG and the 2012 nonpotable demand is approximately 1,162 MG. This shows that the WRF and wells can supply all the estimated 2012 nonpotable demands as long as there is sufficient storage in the system, primarily for the three summer months when there is an estimated deficient of approximately 78 MG/month. During the majority of the year, there is enough nonpotable water to supply the nonpotable demand, assuming that there is negligible nonpotable demand during the winter months. If this water can be stored properly to preserve water quality, then there will be an excess of approximately 205 MG in a year.

In 2062, the nonpotable supply will not be able to accommodate the nonpotable demands by approximately 950 MG/year. There is a chance that the nonpotable supply will increase in the coming years as the population increases. This is not reliable, however, as the WRF effluent volume stayed relatively consistent over approximately 10 years, even as the population increased. Thus, it is not assumed that the WRF effluent will increase very much, so the combined nonpotable supply will remain the same for this research. Because the capacity will remain the same, there are two options for the 2062 nonpotable demands:

1) Find additional nonpotable water supplies.
2) Limit the number of nonpotable connections to a sustainable number.

For Design 1, the primary new nonpotable water supply would be to drill additional wells. For Design 2, the number of nonpotable connections could be limited to either new housing additions or locations near to nonpotable water sources, or both. The city of St. Petersburg faced this same problem and decided to limit the number of nonpotable connections because one yard required the flow from three houses (St. Petersburg, 2012). For this research, Design 2 will be used for the 2062 nonpotable demands and the number of nonpotable connections with a total of 3.8 MGD during maximum month demands.

Junction Pressure

Distribution systems must provide adequate pressures for the end users, so junction pressures are particularly important in designing distribution systems. First starting with state regulations, Oklahoma regulations for nonpotable distribution systems do not specify a minimum required pressure; however Oklahoma regulations for potable distribution systems require a minimum pressure of 25 pounds per square inch (psi) of pressure in the lines (ODEQ, 2012). This potable minimum pressure has been deemed to provide adequate pressures at household fixtures. Second, the nonpotable distribution system will have to supply water for irrigation and irrigation sprinkler heads typically require a minimum of 20 psi for proper operation (Asano et al., 2007; Rain Bird Corporation, 2017). While a nonpotable system is not required to provide a minimum pressure, this research will aim to provide a minimum pressure of 30 psi in the nonpotable system to account for elevation changes, headloss, and sprinkler heads requiring more pressure (Asano et al., 2007). ODEQ does not have a maximum pressure limit, but the maximum recommended pressure for irrigation systems is 80 psi (Asano et al., 2007).

Water Line Material

The City of Norman has installed primarily PVC water lines since the 1980s, likely because PVC is relatively corrosion-resistant, easy to install, lightweight, and relatively cheap compared to other pipe materials. Thus, PVC is the only pipe material considered for the nonpotable distribution system with Hazen-Williams coefficient of 140 to account for minor losses (Chin, 2013).

Water Line Velocities

Water line velocities are not regulated by Oklahoma, but some municipalities set a maximum allowable velocity at three to six feet per second (fps) and the AWWA recommends a maximum velocity of five feet per second during average day demands (Chin, 2013). This helps minimize potential friction loss and hydraulic transients, which in turn leads to higher pressures and lower pumping costs (Chin, 2013). For this research, the velocities during average day demand will typically be limited to five feet per second, in accordance with the AWWA.

Water Line Sizing

The pipes and junctions in a distribution system must supply adequate pressure, adequate velocity, and not experience excessive headloss. Potable distribution systems must be sized for the larger flow of either the maximum day demand with fire flow or the peak hour demand (Chin, 2013). The nonpotable distribution system for this research however will only provide irrigation flows, so the water lines will be sized for these flows. The largest water line will come from the WRF, because it has the single largest flow of the nonpotable water sources. The wells combined have a larger flow than the WRF, but the individual wells have less flow than the WRF does. Thus, the largest flow will come from the WRF, with approximately 1.8 MGD.

Initially, the nonpotable water lines had the same diameters as the potable distribution system water lines. Six additional alternatives were created to compare the maximum flow and maximum velocity in the water lines, with four alternatives using the original diameters from the existing distribution system while limiting the maximum diameter size and two alternatives incrementally reduced all the water line diameters by two inches:

1) Maximum diameter of 16 inches
2) Maximum diameter of 14 inches
3) Maximum diameter of 12 inches
4) Maximum diameter of 10 inches

5) Reduce all diameters that are 6 inches and larger by 2 inches or to the next standard pipe size
6) Reduce all diameters that are 6 inches and larger by 4 inches or to the next standard pipe size

The results from these alternatives reveal that the maximum velocity stays below 6.0 fps for all the diameter alternatives, though it approaches 6.0 fps when the maximum diameter in the system is 10 inches, as shown in the table below. Additionally, the pressures throughout the system remained at approximately 32 psi during all four alternatives. The results are provided in Table 37.

Table 37. Maximum Velocities During Nonpotable ADD with Diameter Alternatives

Alternative	Maximum Velocity (fps)	Flow in Pipe with Maximum Velocity (gpm)	Diameter of Pipe with Maximum Velocity (gpm)	Minimum Pressure in System (psi)
Original Diameters	3.85	151.0	4"	32.6
16" Max Diameter	4.30	168.5	4"	32.6
14" Max Diameter	4.48	175.5	4"	32.6
12" Max Diameter	4.73	185.1	4"	32.5
10" Max Diameter	5.81	227.5	4"	32.4
2" Reduction	3.67	143.6	4"	32.7
4" Reduction	4.65	182.0	10"	31.6

After completing the analysis on average day demands, the other three nonpotable demand scenarios were run with the 10-inch Maximum Diameter, the 12-inch Maximum Diameter, the 2-inch Reduction, and the 4-inch Reduction alternatives to evaluate the velocities and pressures in the nonpotable distribution system. The limiting demand scenario for each alternative was the peak hour demand, which resulted in negative pressures throughout the system with the 10-inch and 12-inch Maximum Diameter alternatives and the 4-inch Reduction alternative, with minimum pressures of -88.8 psi, -24.2 psi, and -93.3 psi respectively. The headlosses and velocities were also extremely high for both of these alternatives, while they were almost negligible during much smaller demand scenarios. This was likely caused by high flows going to the edges of the distribution system. The pressures were lowest at the higher elevations and at the edges of the distribution system farthest from the water sources. Only the 2-inch Reduction alternative had positive pressures through the entire system, though the minimum pressure was still extremely low at 1.3 psi. This occurred at the highest point in the distribution system. Because the 2-inch Reduction alternative performed the best of the six alternatives, it was used for the hydraulic analysis.

One final step further refined the distribution system. The water lines connected to the wells and WRF were increased to 12-inch diameters and several water lines serve potable sources were downsized to 8-inch and 10-inch diameters because they did not need to transport the same volume of water as with the single or potable systems. The figure below shows the final diameters across the nonpotable distribution system.

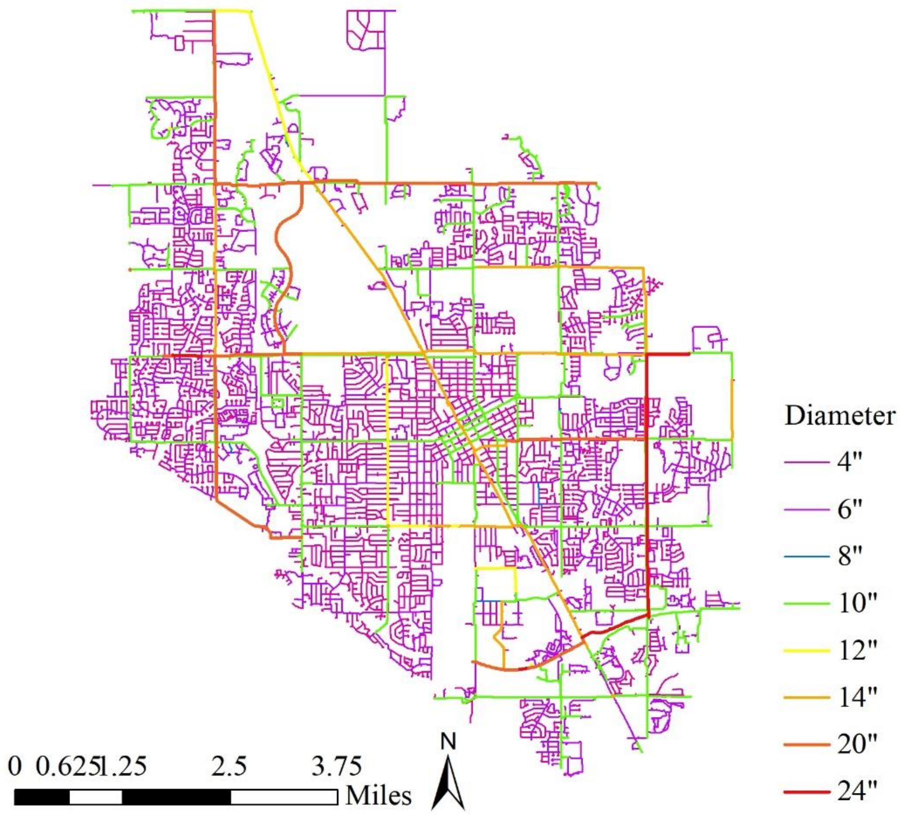

Figure 42. 2012 Nonpotable Distribution System Diameters

Storage

All systems need storage to accommodate peak demands, whether it is the morning and evening peaks for potable demand, nighttime peak for public irrigation, or fire flows (Asano et al., 2007). In nonpotable distribution systems, storage accounts for diurnal and seasonal variations in nonpotable water demand (Asano et al., 2007). The irrigation demand is often highest during the summer months and decreases during the fall to a minimum or nonexistent demand during the winter. The WRF, however, continues to produce effluent at a relatively consistent rate.

As it was discussed previously, water storage would be required to meet peak demands because the nonpotable sources have a maximum supply of 3.8 MGD, which is less than the peak nonpotable demands. Thus, water storage is required to store the recycled water that is not needed during periods of low demand and to supplement the effluent source during the high demand months.

There are two types of storage for nonpotable water – short-term and long-term (Asano et al., 2007). First, short-term storage typically includes tanks, such as ground storage and elevated storage tanks. Besides providing storage, elevated storage provides an additional benefit in that it helps to maintain pressures in the distribution system if the storage is elevated. There are simulated pumps at the wells and the WRF, however, the HGL will decrease as the water gets farther from the source. Because the initial nonpotable distribution system will serve the entire service area, it is likely that the HGL and pressures will decrease to below 20 psi in the farthest locations.

Second, long-term storage includes reservoirs, which would likely be necessary for the nonpotable distribution system in this research. For 2012, at least 230 MG would need to be stored long-term to cover the maximum month nonpotable water demands. This volume was determined by calculating the monthly total volume provided by the nonpotable sources and demanded from the nonpotable users. The nonpotable demand is assumed to be zero during December, January, and February since these are winter months and no irrigation would be needed. Average day demands are assumed for spring (i.e., March, April, and May) and fall (i.e., September, October, and November), and maximum month demands during the summer (i.e., June, July, and August). The demands differ from month to month because the daily demands were multiplied by the number of days in the months. The monthly differences between the supply and demand are presented in the table below.

Table 38. Nonpotable Supply versus Demand for Storage

Month	Supply Capacity (MG/mo)	2012 Demand (MG/mo)	Excess Supply (MG/mo)
January	118	0	118
February	106	0	106
March	118	100	18
April	114	97	17
May	118	100	18
June	114	186	-72
July	118	193	-75
August	118	193	-75
September	114	97	17
October	118	100	18
November	114	97	17
December	118	0	118
Total	**1,388**	**1,163**	**225**

The table shows that the supply from the nonpotable sources exceeds the nonpotable demand by a yearly total of 225 MG. This is the total volume of water that could be stored to accommodate peak demands, which aligns wells with the deficit volume during June, July, and August. During June, July, and August, the demands exceed the supply by 222 MG. These two numbers match well and show that the sources could likely supply the nonpotable demands. This value is used as the basis for determining the nonpotable storage needed to meet the nonpotable demands.

Even more would need to be stored to cover the maximum day and peak hour nonpotable water demands. However, the nonpotable sources cannot accommodate this. Thus, the nonpotable storage would have to be limited to approximately 225 MG. Storing 225 MG would likely require a reservoir, which equates to approximately 690 acre-feet. If the reservoir were 10 feet deep, the surface area would be at least 69 acres. While this would not occupy much land, this reservoir would likely be impractical for several reasons. First, it would require some sort of mixing to prevent the water from becoming stagnant during the months when the demand is less than the supply. Second, it would require pumps to withdraw water from the reservoir, which adds to the cost and power needed. Third, a long-term storage reservoir would likely cost more money than the citizens of Norman would like to spend for nonpotable water, though this specific reason is evaluated in more detail in Chapter 6.

If the WRF can reliably discharge more effluent, then the total storage volume could decrease and the nonpotable distribution system could supply more of the nonpotable demand on a daily basis. For the 2012 hydraulic model of the nonpotable distribution system, it will be assumed that storage is provided at the WRF and the supply for the peak demands will come from the WRF.

2012 Nonpotable Distribution System

The 2012 nonpotable distribution system is modeled along the same lines as the 2012 single distribution system and 2012 potable distribution system. The nonpotable distribution system does not extend quite as far as the wells on the eastern side of Norman because there are few water users along those water lines. This system is the basis for designing the 2012 and 2062 nonpotable distribution systems. The 2012 nonpotable distribution system is shown in Figure 43.

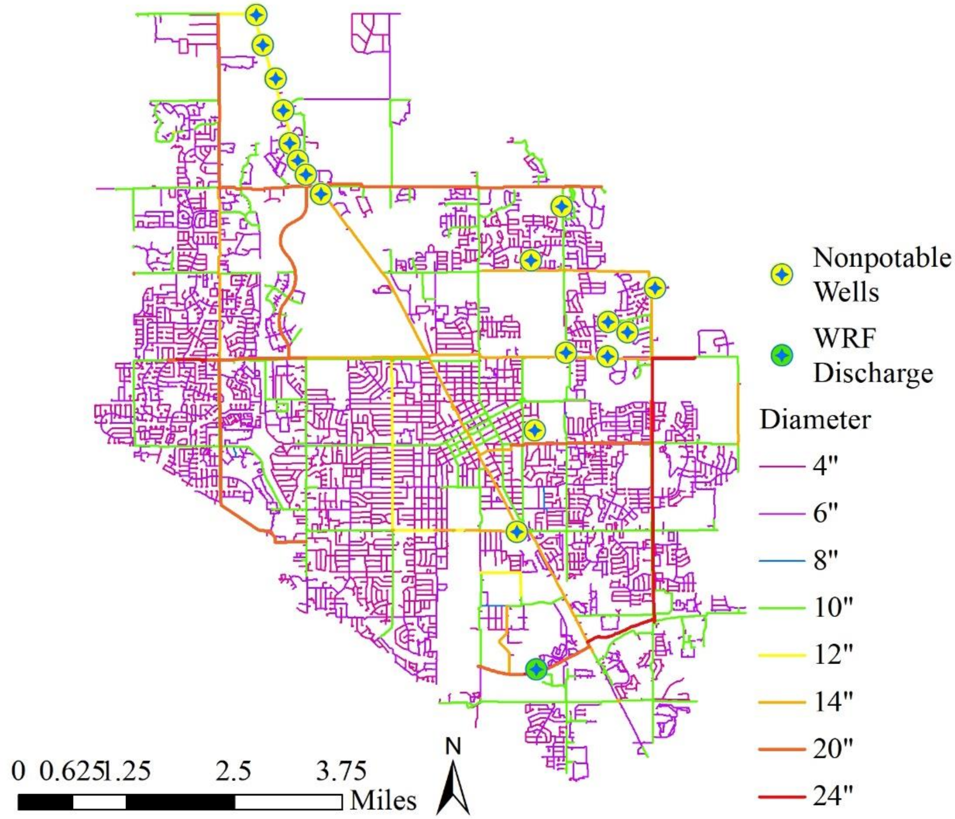

Figure 43. 2012 Nonpotable Distribution System

Creating the 2062 Single and Potable Distribution Systems

To model the distribution systems in 2062, they need to be expanded to accommodate the increased demands and growing population. The 2062 demands and growth areas were determined in Chapter 3 and earlier in Chapter 4, respectively. The new water lines for the 2062 distribution systems will follow the square mile section lines in the growth areas determined earlier.

Adding New Water Lines

The new water lines are drawn in one-mile sections using the projected growth area from the City of Norman. The projected growth area encompasses nearly 38 square miles, which is more than adequate to accommodate the growing population and demand.

The figure below shows the expanded distribution system with an additional 38 square miles.

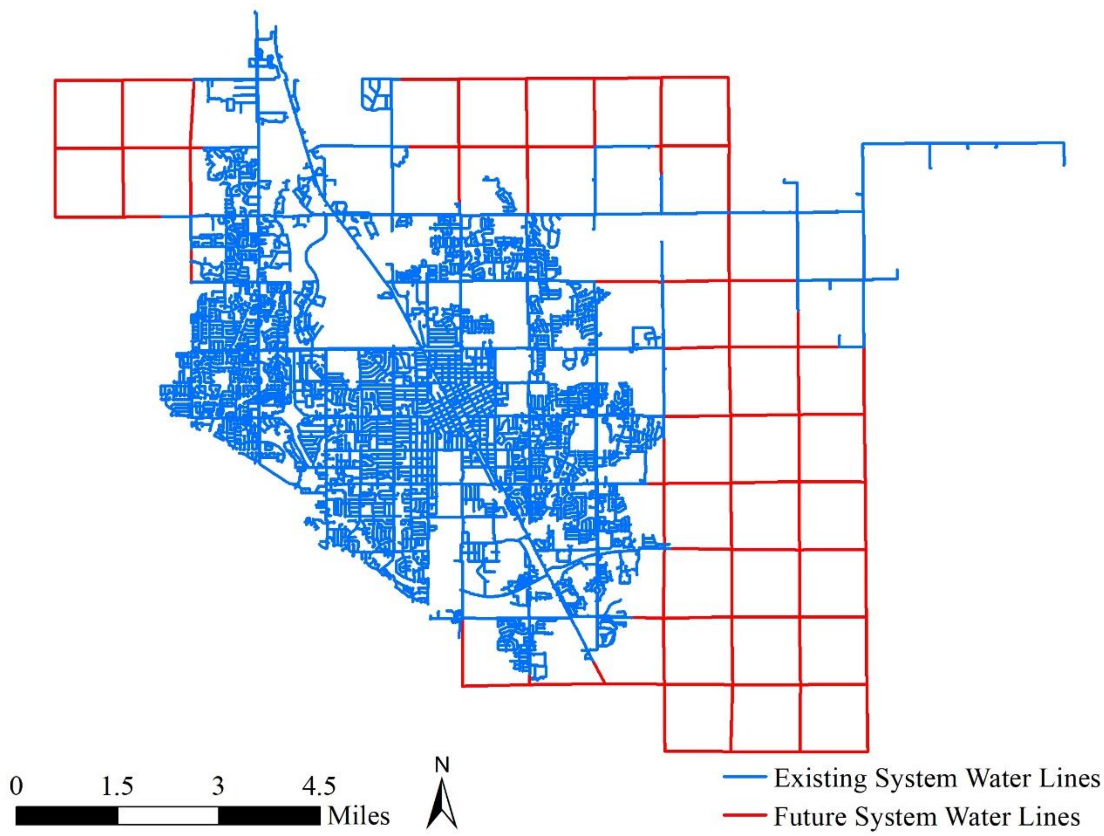

Figure 44. New Water Lines for 2062 Distribution System

In the above figure, 78 new junctions were added for the additional space. The increased demand is placed on these 78 new junctions, with the demands calculated as the components for the base demand, estimated average irrigation demand, and estimated maximum irrigation demand. As stated above in the Assumption Section for Allocating 2062 Demands, the property sizes are approximately the same for each of the additional square miles and that the additional commercial and residential demands are equally distributed across the growth area. This will be the design for both the 2062 single distribution system and the 2062 potable distribution system, because the population must still be served by potable water.

New Water Line Sizes

More than 90 percent of the water lines in the City of Norman are 12 inches or smaller and most of the water lines along section roads are 12 inches, though there are some main water lines that are 16, 24, and 30 inches. To accommodate the population growth, the new water lines are initially modeled at 12 inches, though a new 16 inch water main is simulated along the eastern edge of the distribution system, shown in Figure 45.

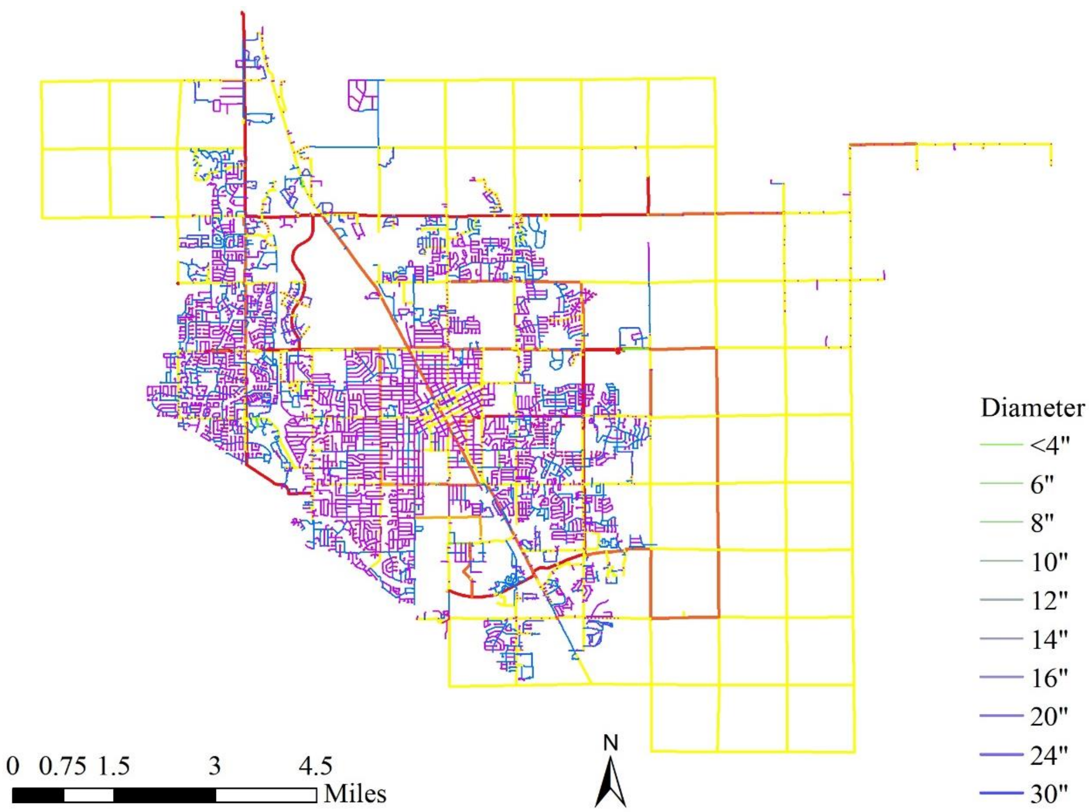

Figure 45. 2062 Single Distribution System Proposed Diameters

New Elevated Storage

During the initial hydraulic evaluations, some of the pressures in the expanded area of the distribution system were negative, partly due to high elevations in the southeastern-most part of the expanded distribution system. Additionally, elevated storage will likely be needed to supply the peak demands and provide fire flow storage. As such, a 0.5 MG water tower is simulated in the southeastern part of the distribution in the location shown in the figure below. The volume was chosen to be consistent with the volumes of the other water towers in the distribution system. The location was chosen because it is relatively close to the existing distribution system while being relatively central to the southeastern corner of the expanded distribution system, as shown in the figure below.

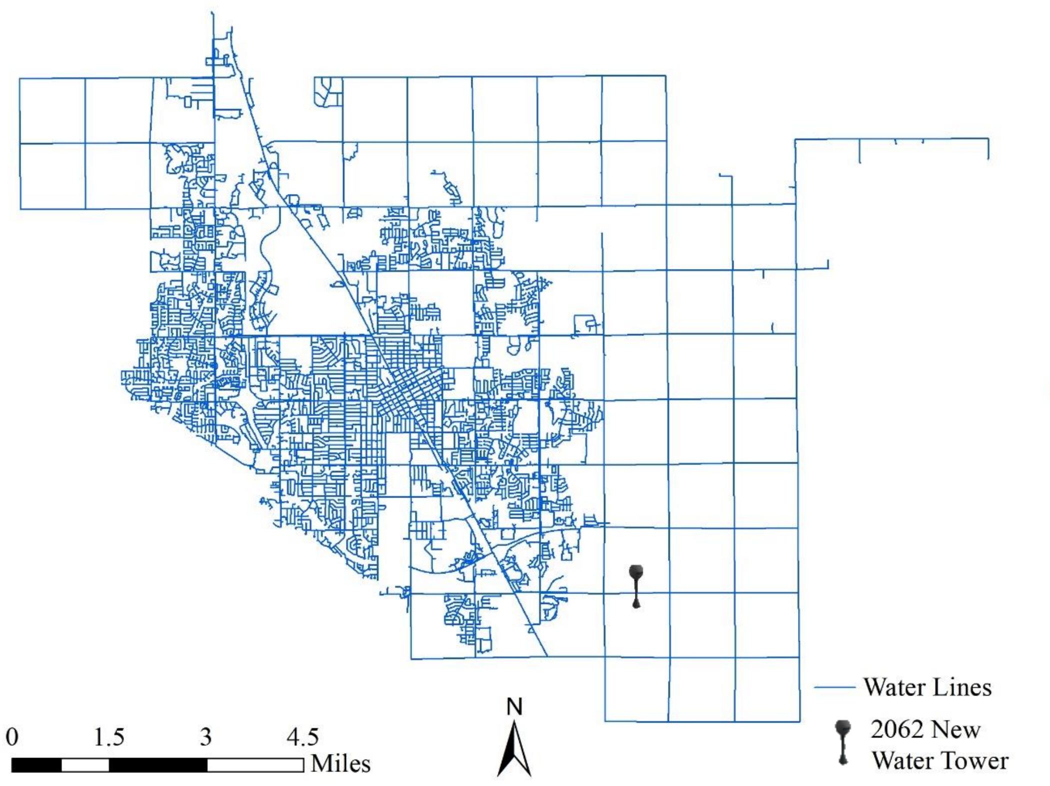

Figure 46. Location of Proposed Water Tower for 2062 Distribution System

Creating the 2062 Nonpotable Distribution System

The nonpotable distribution system must also be modeled in 2062 to evaluate how well it will do and to help determine the service area that could more realistically be served by the nonpotable sources. As it was discussed previously, the current nonpotable water sources will not be able to supply the 2062 nonpotable demands. As a result, the 2062 nonpotable distribution system will not be modeled to the same spatial extent as the potable distribution system. Instead, it will be limited to only the number of connections with a total maximum month demand of 3.8 MGD. This equates to a little more than half of the 2012 nonpotable distribution system, which has a maximum month demand of 6.2 MGD. The 2012 distribution system aimed to serve the entire 2012 service area, which was done to evaluate how feasible a large nonpotable distribution system would be. The 2062 nonpotable distribution system aims to selectively serve approximately 50 percent of the service area, particularly the areas that are closest to the nonpotable distribution sources or proposed for 2062.

This approach is more realistic to what would actually happen if a secondary nonpotable distribution system were to be installed in Norman. Modeling the nonpotable distribution system in 2012 is more an exercise to see how well the system would do with existing demand. Modeling the nonpotable distribution system in 2062 is closer to looking at what the system would be if it were to be installed.

The HGL for all models is set at 1310 feet, which is slightly lower than the HGL for the potable system, by setting the elevations for the wells at 1310 feet in the hydraulic models. In general, this will help to keep the pressures in the nonpotable system lower than the pressures in the potable system to help prevent potential contamination in the event of a leak in either system.

Evaluation Criteria

The evaluation criteria are the same for the 2062 nonpotable distribution system as for the 2012 nonpotable distribution system. This means that the minimum pressure is 30 psi and the maximum velocity during average day demands is 6.0 fps.

Water Line Design

The water lines are all simulated to be PVC water lines with a Hazen-Williams coefficient of 140, which is slightly lower than the new coefficient value of 150 to account for minor losses in the distribution system. The water line diameters from the 2-inch Reduction alternative for the 2012 nonpotable system were used as the basis for the 2062 nonpotable system design.

2062 Nonpotable Service Area

To determine the optimal nonpotable service area for 2062, the service area is limited to the customers with a cumulative maximum day demand of 3.8 MGD, which is the estimated capacity of the nonpotable water sources. For this research, two service area options were considered:

1. Primarily serve the areas closest to the nonpotable sources, whether those areas are in existing distribution system or in new areas; and,
2. Primarily serve new areas of the distribution system, with some service to existing areas.

The service area for Design 1 attempts to supply nonpotable water to the areas closest to the sources, so some existing neighborhoods and commercial areas are simulated with retrofits near the eight wells that are in the center of Norman. Seven other wells on the north side of Norman are connected with a water main to send the water to the north and south to serve the existing neighborhoods and commercial areas to the west and south of these wells. The WRF has the largest service area, because it has the single largest supply capacity. The total service area is approximately 26 square miles with approximately 194 miles of total water line. The resulting service area and water line diameters are shown in the figure below.

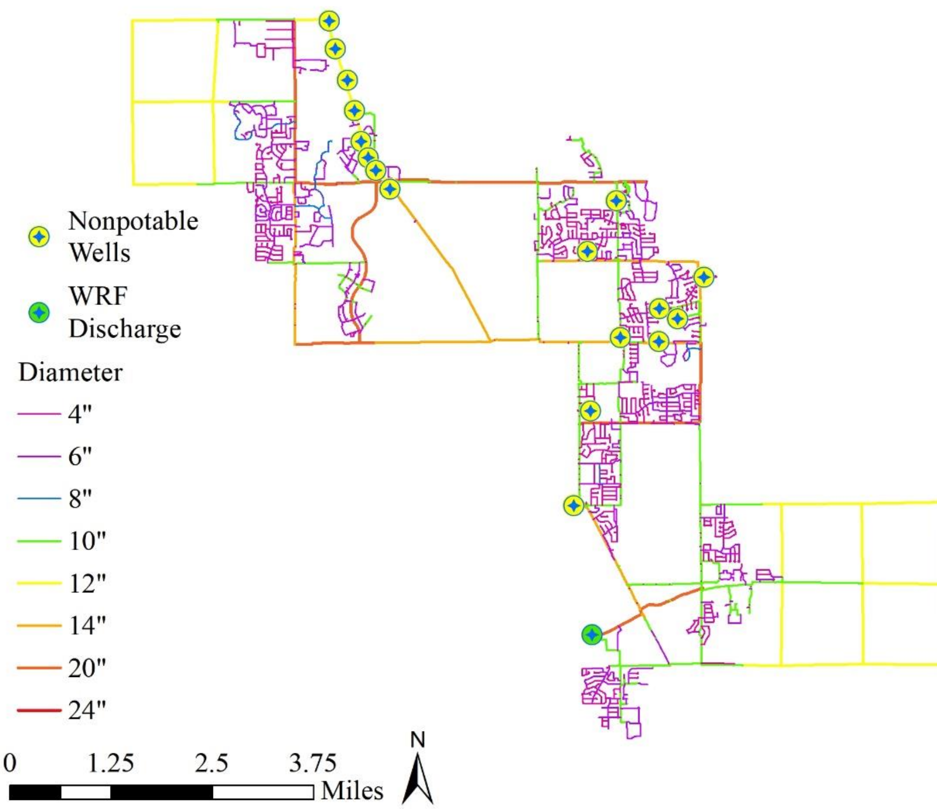

Figure 47. Service Area and Diameters of 2062 Nonpotable Distribution System – Design 1

The water line diameters were adjusted from the 2012 2-inch Reduction alternative to reduce the diameters while maintaining adequate pressures in the system. While the peak hour pressures within the distribution system do not meet the requirements as discussed in Chapter 5, the adjusted water line diameters are smaller than they were initially, which results in a cheaper distribution system, as discussed in Chapter 6. The majority of water lines have either 4-inch or 6-inch diameters and these water lines are located in the neighborhoods. A few of the water lines in the neighborhoods have 8-inch diameters. All the water lines that connect nonpotable sources to the distribution system have 10-inch diameters, and all the water lines along major roads or section lines have at least 10-inch diameters, with a few 12-inch, 14-inch, and 18-inch diameters. The distribution of water lines for Design 1 of the 2062 nonpotable distribution system are provided in Figure 48 below.

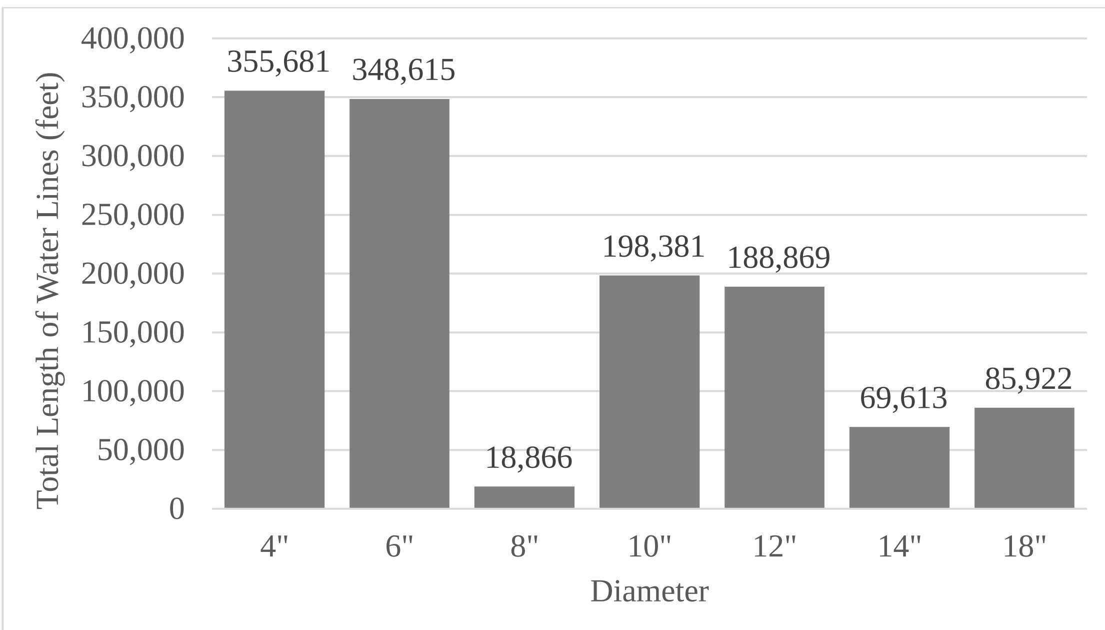

Figure 48. Proposed Lengths of Water Lines by Diameter for Design 1

The service area for Design 2 only includes the expanded area, which has an estimated maximum day demand of 4.42 MGD. This is greater than the available supply from the nonpotable sources, so a few junctions are omitted to keep the maximum day nonpotable demand to 3.8 MGD. The water lines within the existing distribution system are only there to connect the nonpotable water sources to the new service area. Additionally, the water line diameters were further refined so that the water line diameters were all similar in size, e.g., a 10-inch water line is increased to 12 inches if it is between two 12-inch water lines or decreased to 8 inches if it is between two 8-inch water lines. Other than the water lines to the wells, the resulting design is entirely looped, though it does not include the detailed water lines to serve the new customers.[7] The resulting service area is approximately 34 square miles, which is shown with the water line diameters in the figure below.

[7] Note that there is no water line across the northern side of the distribution side to connect the northern nonpotable wells with the northeast section of the expanded distribution system. This was intentional because a railroad line goes through this area and, based on work experience interacting with several railroad companies, it is challenging to obtain a permit from the railroads to cross under them. Thus, the proposed water lines for Design 2 for the 2062 nonpotable distribution system follow the paths for existing water lines.

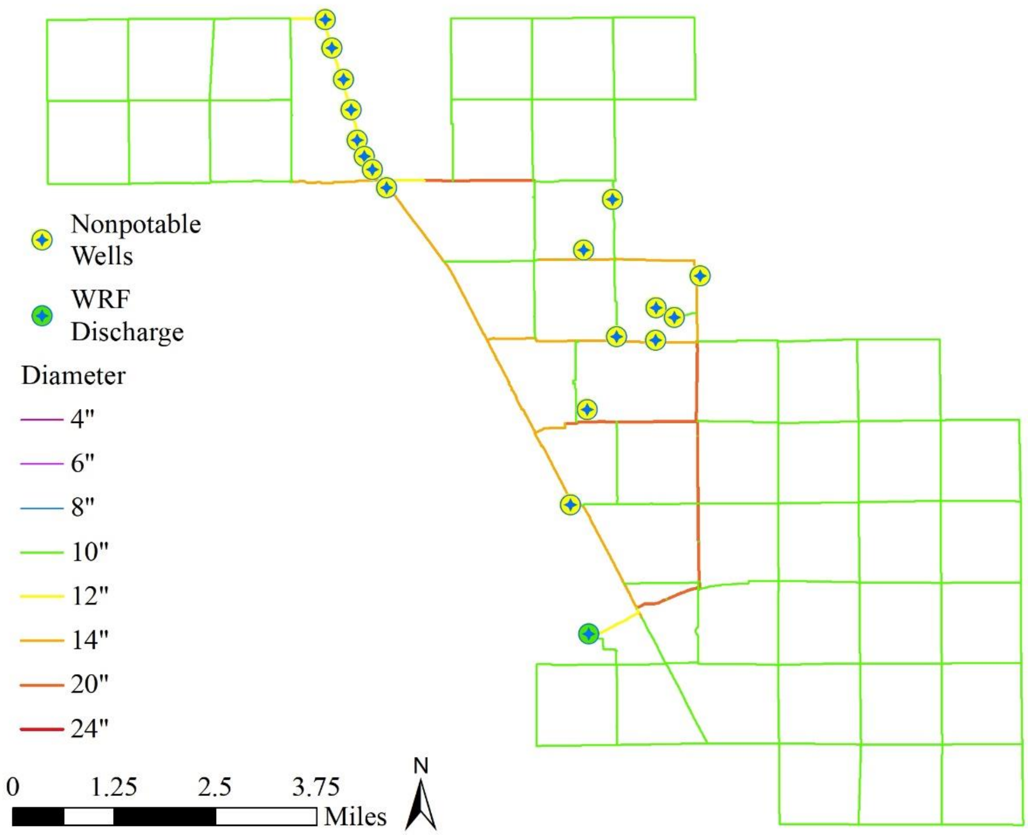

Figure 49. Service Area and Diameters of 2062 Nonpotable Distribution System – Design 2

In the figure above, there are approximately 124 miles of nonpotable water line. Approximately 82 percent of the water lines in Design 2 are 10-inch diameter water lines, 3 percent are 12-inch diameter, 10 percent are 14-inch diameter, and 5 percent are 18-inch diameter water lines. These water lines were selected to provide adequate pressures at the junctions during maximum day demand, as shown in Chapter 5.

In some regards, the design for Design 1 is more detailed than the design for Design 2 because it has more water lines to serve neighborhoods, whereas the design for Design 2 is mainly along square mile sections and assumes that developers will provide the more detailed designs to serve the customers. Conversely, the diameters for Design 2 are more carefully selected because there are fewer of them. The hydraulic results from both designs are presented in Chapter 5.

Storage

The 2062 nonpotable distribution system has two designs to serve the future nonpotable service area, with maximum day demands of 3.8 MGD. Storage would be necessary to meet the peak hour demands, since the maximum day demands are equal to the nonpotable water supply. Over a year, the proposed nonpotable system is estimated to need approximately 725 MG less than what the WRF and wells can supply; however, this volume of storage is not necessary to serve the proposed service areas. For comparison, the table below shows how much water would be needed to supply the entire 2062 service area with nonpotable water and how much water would be needed to supply limited 2062 service areas, with estimates for the amount of excess water supplied by the nonpotable sources. The table shows that the entire service area would demand almost 1,000 MG more than the nonpotable sources can supply whereas the limited service area has an excess supply of 725 MG over a year.

Table 39. Summary of Supply and Demands for 2062 Nonpotable System

Month	Supply Capacity (MG/mo)	2062 Total Nonpotable Demand (MG/mo)	Excess Supply with Total Demand (MG/mo)	2062 Limited Service Area Demand (MG/mo)	Excess Supply with Limited Demand (MG/mo)
January	118	0	118	0	118
February	106	0	106	0	106
March	118	200	-82	64	54
April	114	194	-80	62	52
May	118	200	-82	64	54
June	114	371	-257	93	21
July	118	384	-266	96	22
August	118	384	-266	96	22
September	114	194	-80	62	52
October	118	200	-82	64	54
November	114	194	-80	62	52
December	118	0	118	0	118
Total	**1,388**	**2,321**	**-933**	**663**	**725**

The limited service area is designed so that the distribution system can meet the maximum month demands, so no storage would be needed to meet the maximum month or maximum day demands. Instead, storage would be needed to meet peak hour demands. Since the nonpotable sources are estimated to have an excess capacity of 725 MG during the year, there should be adequate supply to meet the peak hour demands. The estimated peak hour demand needed for the limited service are is approximately 10.5 MGD, which is 6.7 MGD more than the nonpotable sources can supply. If the peak hour demand were required for several consecutive days (e.g., 20 days) for a limited time each day, then the distribution system would need approximately 134 MG during one month. If this example is expanded to cover the three summer months (assuming the peak hour demand is demanded for at least 20 days each month during a limited time each day), then approximately 400 MG of storage might be needed. This equates to approximately 1,230 acre-feet. The nonpotable sources could supply this estimated storage volume because it provides an estimated excess 725 MG over the year. For the nonpotable distribution system in 2062, it is assumed that the WRF has enough land space to provide a storage reservoir large enough to accommodate 400 MG. In reality, the hydraulics would probably be better if two storage reservoirs were provided – one on the northern side of the distribution system close to the wells and one located near the WRF. For this research however, only one long-term storage reservoir is included in the hydraulic analyses. The potential costs are evaluated in Chapter 6.

Summary of Allocating Demands and Preparing the Hydraulic Models

Chapter 4 discussed allocating demands for 2012 and 2062, creating the 2012 single distribution system, calibrating and verifying the 2012 single distribution system, creating the 2012 nonpotable distribution system, and expanding the distribution systems for 2062. One of the challenges associated with expanding the distribution systems is that the nonpotable water sources cannot supply all the nonpotable demands, especially without large storage volumes that would be somewhat cost-prohibitive to construct. As a result, the nonpotable distribution system is scaled down to approximately 60 percent in order to be more practical, provide better service, and keep the demands within the available supply, with two designs to serve nonpotable demands in 2062. These two designs provide a choice between serving a mix of existing and potential water users for Design 1 and serving only potential water users for Design 2. Chapter 5 will evaluate the hydraulics of these distribution systems.

Chapter 5: Hydraulic Analyses

Introduction

After the hydraulic models were prepared, as discussed, in Chapter 4, they are evaluated in Chapter 5. The models are analyzed primarily for pressure because it is the main criteria for most distribution systems. The models are also analyzed for velocities and headloss to evaluate how well they perform. Because the distribution systems are looped, low velocities and pressures could cause problems in terms of system performance, water quality, and irrigation performance (Kang and Lansey, 2012).

For the single and potable distribution system scenarios, ODEQ regulations will be followed. For the nonpotable distribution system scenarios, the design criteria established in Chapter 4 will be followed. The single and potable distribution system are compared to each other because they both provide potable water to all the end users in the distribution service area. The single and potable distribution systems are not directly compared to the nonpotable distribution system because it would not be a direct comparison – they do not have the same criteria nor the same flows.

This chapter starts with a brief discussion of the governing equations for Bentley® WaterGEMS software. Then, the hydraulic results for several scenarios are presented, with visual representations of the results in each subsection below. Seven hydraulic models are evaluated:

- 2012 Single Distribution System
- 2012 Potable Distribution System
- 2012 Nonpotable Distribution System
- 2062 Single Distribution System

- 2062 Potable Distribution System
- 2062 Nonpotable Distribution System – Design 1
- 2062 Nonpotable Distribution System – Design 2

Each of the models are evaluated with their respective demands for:

- Average Day Demand
- Maximum Month Demand
- Maximum Day Demand
- Peak Hour Demand

For simplicity, only the results for average day, maximum day, and peak hour are reported because the results for maximum month and maximum day for every model were very close to each other. For each reported scenario, figures depicting the junction pressures and the pipe velocities are provided. Additionally, available fire flow results are provided for the maximum day demand scenarios for the single and potable distribution systems.

Governing Equations

The continuity principle is one of two governing equations that support the simulations in WaterGEMS; continuity requires that the sum of outflows at each junction is equal to the sum of inflows (Bentley, 2013; Chin, 2013). The boundary conditions are the demands, the tank levels, and the reservoir elevations. The steady-state continuity equation for an incompressible fluid is (Chin, 2013):

$$\int_{A_1} v_1 dA = \int_{A_2} v_2 dA$$

Where: v = velocity

A = area

This equation can be simplified to (Chin, 2013):

$$v_1 A_1 = v_2 A_2 (= Q)$$

Where: v = velocity

A = area

Q = volumetric flow rate

The total head is calculated as (Chin, 2013):

$$h = \frac{p}{\gamma} + \alpha \frac{V^2}{2g} + z$$

Where: h = total head

p = pressure

γ = specific weight of water at 20°C, (62.4 pounds·force per cubic foot)

α = kinetic energy correction factor

V = average velocity across the pipe cross section

z = elevation of the centroid of the pipe cross section

The total head equation is part of the energy equation, which helps solve the pipe network equations (Bentley, 2013):

$$\left(\frac{p_1}{\gamma} + \alpha_1 \frac{V_1^2}{2g} + z_1\right) = \left(\frac{p_2}{\gamma} + \alpha_2 \frac{V_2^2}{2g} + z_2\right) + h_L + h_s$$

Where: p = pressure

γ = specific weight of water at 20°C, (62.4 pounds·force per cubic foot)

α = kinetic energy correction factor

V = average velocity across the pipe cross section

z = elevation of the centroid of the pipe cross section

h_L = head loss

h_s = work done by fluid per unit weight

Because there are no pumps or turbines included in the hydraulic model, the h_s term is zero. The head loss is calculated by the Hazen-Williams friction equation, which estimates the energy losses as fluid flows through pipes (Bentley 2013):

$$h_l = \frac{10.44\, L\, Q^{1.85}}{C^{1.85} d^{4.8655}}$$

Where: L = length of pipe (feet)

Q = flow through pipe (gallons per minute)

C = Hazen-Williams coefficient

d = diameter (inches)

The HGL is part of the total head calculation, without the kinetic energy term. The HGL is calculated as (Chin, 2013):

$$HGL = z + \frac{p}{\gamma}$$

Where: HGL = Hydraulic Grade Line (feet)

p = pressure (pounds per square inch)

γ = specific weight of water at 20°C, (62.4 pounds·force per cubic foot)

The HGL equation can also be rearranged to calculate pressure (Bentley, 2013):

$$p = (HGL - z)\rho g$$

Where: p = pressure (pounds per square foot)

HGL = hydraulic grade line (feet)

z = elevation (feet)

ρ = density of water (slugs per cubic foot)

g = gravitational acceleration (feet per seconds squared)

Pipe networks are typically solved iteratively for the flow in each pipe by invoking continuity and energy. Once the flows in the pipes are known, the model then generates results for pressure, velocity, and available fire flow in water lines, which are presented in this chapter.

2012 Single Distribution System Hydraulic Evaluations

The single distribution scenarios are evaluated with both potable and nonpotable demands on the same distribution system. All demands are supplied by the WTP and the 26 active wells. The maximum day demand scenarios are also computed with fire flow analysis to determine the available fire flow at each hydrant in the distribution system. In general, the single system performs very well, with a minimum pressure of 27.1 psi during the peak hour demand scenario and most fire hydrants capable of meeting the minimum 1,000 gpm fire flow requirement during maximum day demands (ODEQ, 2012). This indicates that the single distribution system was effectively designed to meet Norman's demands and ODEQ requirements. There is little difference between the demand scenarios in terms of pressure and velocity results, with the minimum pressures between 27.9 psi and 31.5 psi for all four demand scenarios.

2012 Single System with Average Day Demands

The existing system is analyzed after calibration and validation (as presented in Chapter 4). During average day demands of 12.6 MGD, pressures are typically between

60 and 80 psi in the central part of Norman, between 40 and 60 psi on the north and east side of Norman, and higher than 80 psi in the lower elevations in southern Norman along the Canadian River. The minimum pressure reported is 31.5 psi, which is good in that the lowest pressure in the system is greater than the minimum allowed pressure by ODEQ, and the average pressure is 62.1 psi. The spatial distribution of pressures under average day demands is shown in Figure 50 below.

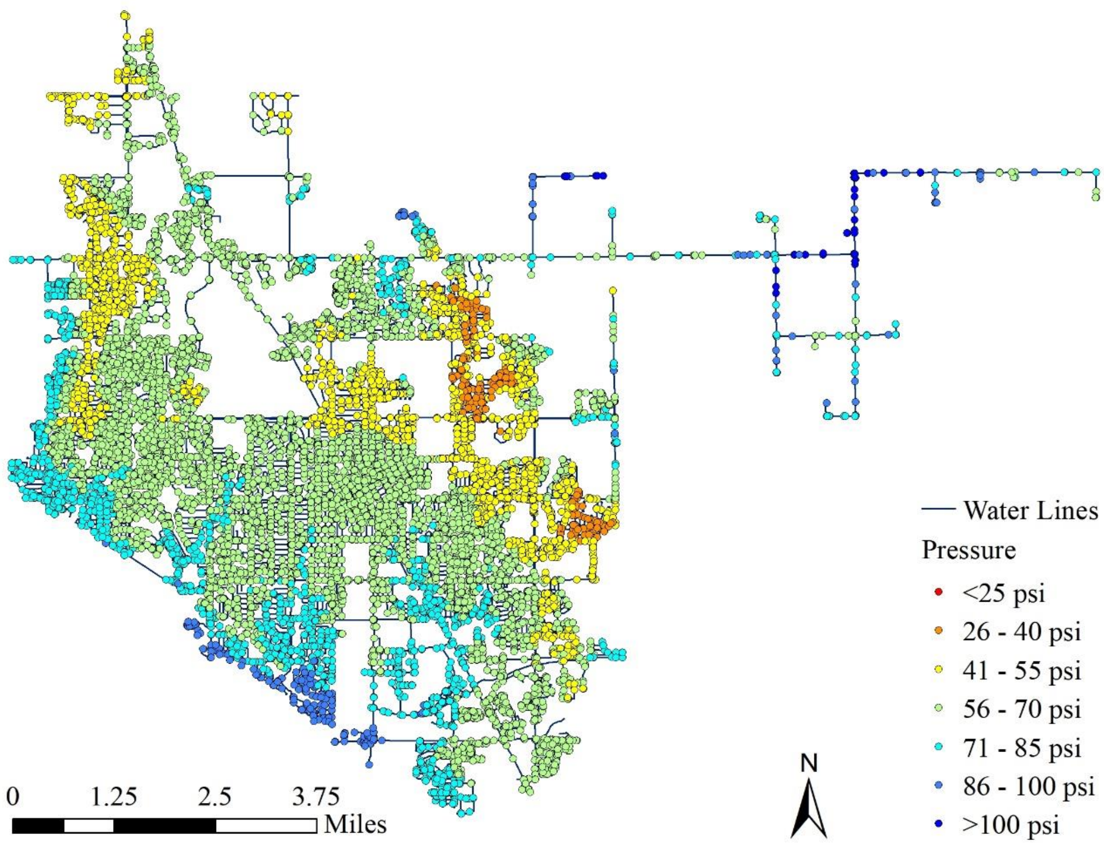

Figure 50. 2012 Single System Average Day Demands Pressure Results

There are no excessive velocities shown in the results for the Norman single distribution system with average day demands. As expected, the velocities in the neighborhoods are slower than the velocities in the mains along the major streets. The highest velocities are along water lines that provide a lot of flow. The spatial distribution is shown in Figure 51 below.

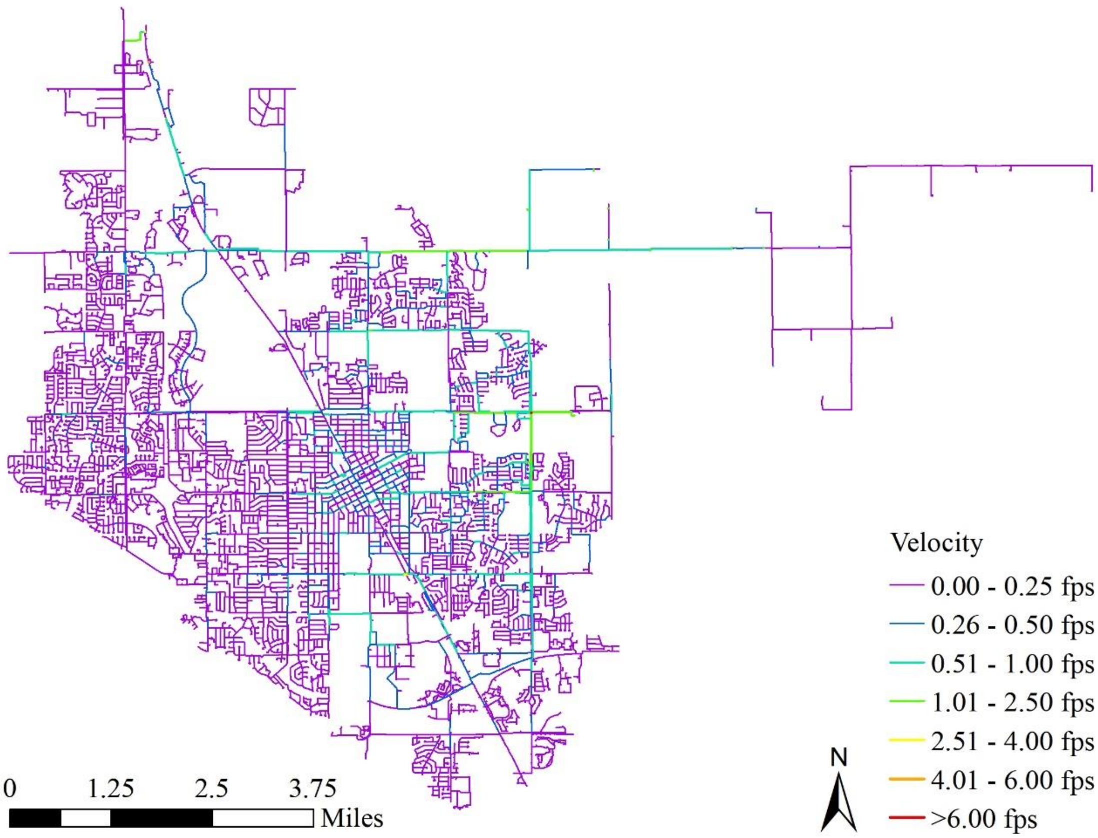

Figure 51. 2012 Single System Average Day Demands Velocity Results

2012 Single System with Maximum Day Demands

The 2012 single distribution system is next evaluated with the maximum day demands of 23.0 MGD applied to the nodes, with an added fire flow analysis. The pressures under maximum day demands have an average of 61.5 psi, with a minimum of 30.8 psi. While these numbers are slightly lower than the numbers for maximum month demands, they are still above the minimum ODEQ pressure of 25.0 psi. As expected the lowest pressures are at the highest elevations, though slightly lower pressures can be seen across the system, as shown in Figure 52.

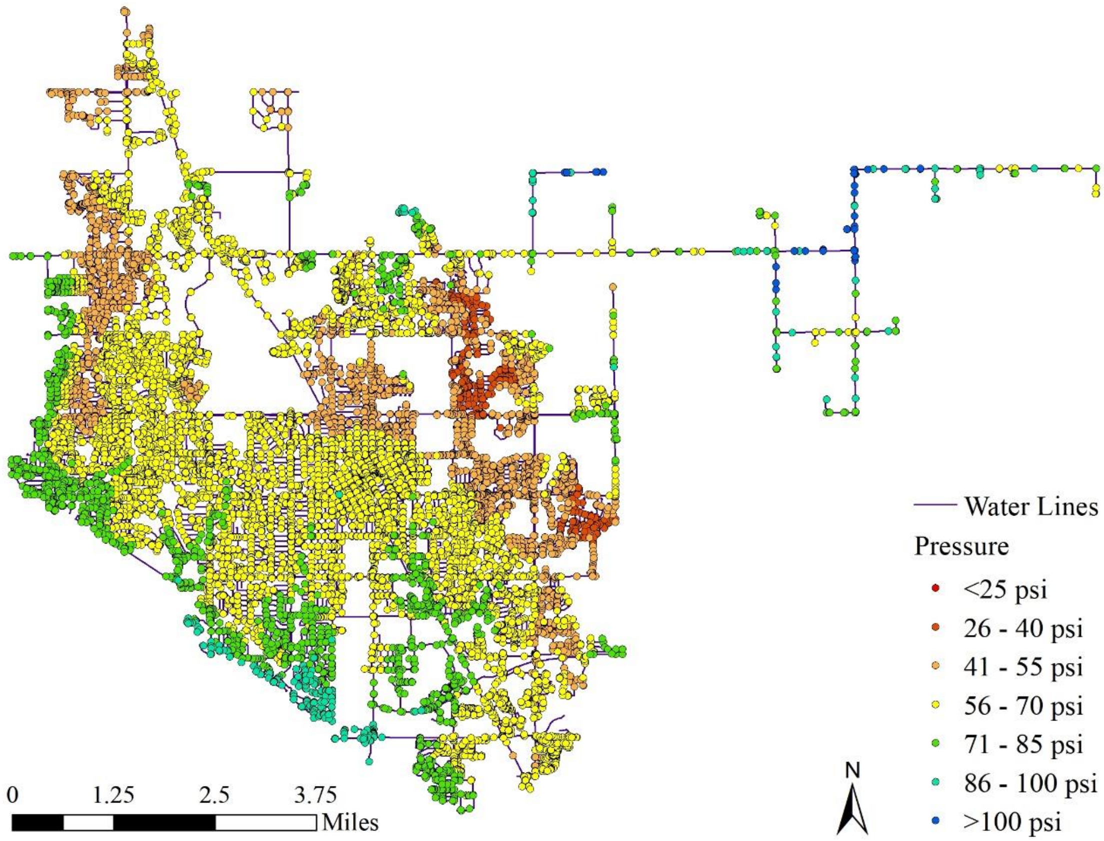

Figure 52. 2012 Single System Maximum Day Demands Pressure Results

As expected, the available fire flow during maximum day demands is slightly lower than with the maximum month demands, due to the higher demand on the system. During this scenario, there are 83 fire hydrants that cannot provide at least 1,000 gpm and 1,140 fire hydrants that can provide at least 5,000 gpm. About a quarter of the fire hydrants with simulated available fire flow less than 1,000 gpm are located in the north central part of the distribution system, where a new development is at the end of a water line. While the numbers are not as good as with maximum month demands, these numbers still indicate that the distribution system can provide adequate fire flow for most of Norman. The spatial distribution of the available fire flow is provided in the figure below.

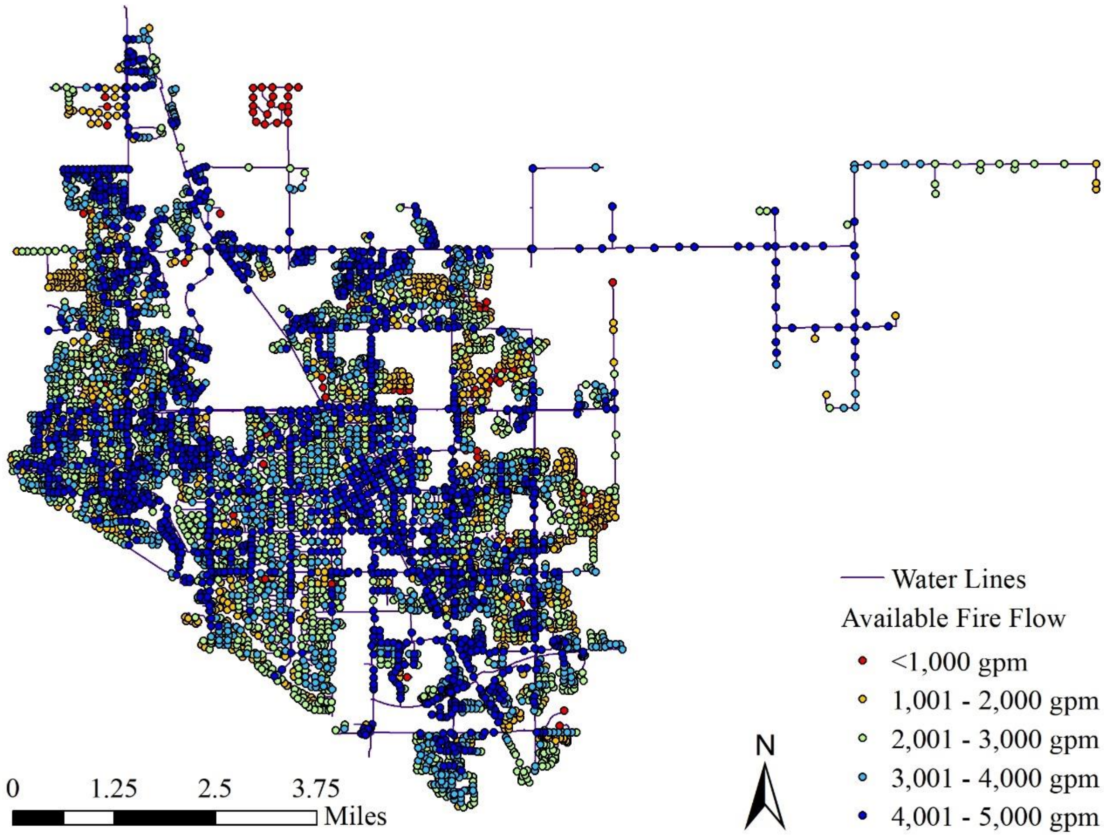

Figure 53. 2012 Single System Maximum Day Demands Available Fire Flow

The velocities under maximum day demands are slightly higher, though they are still well below the maximum velocity of 6.0 fps, as shown in Figure 54.

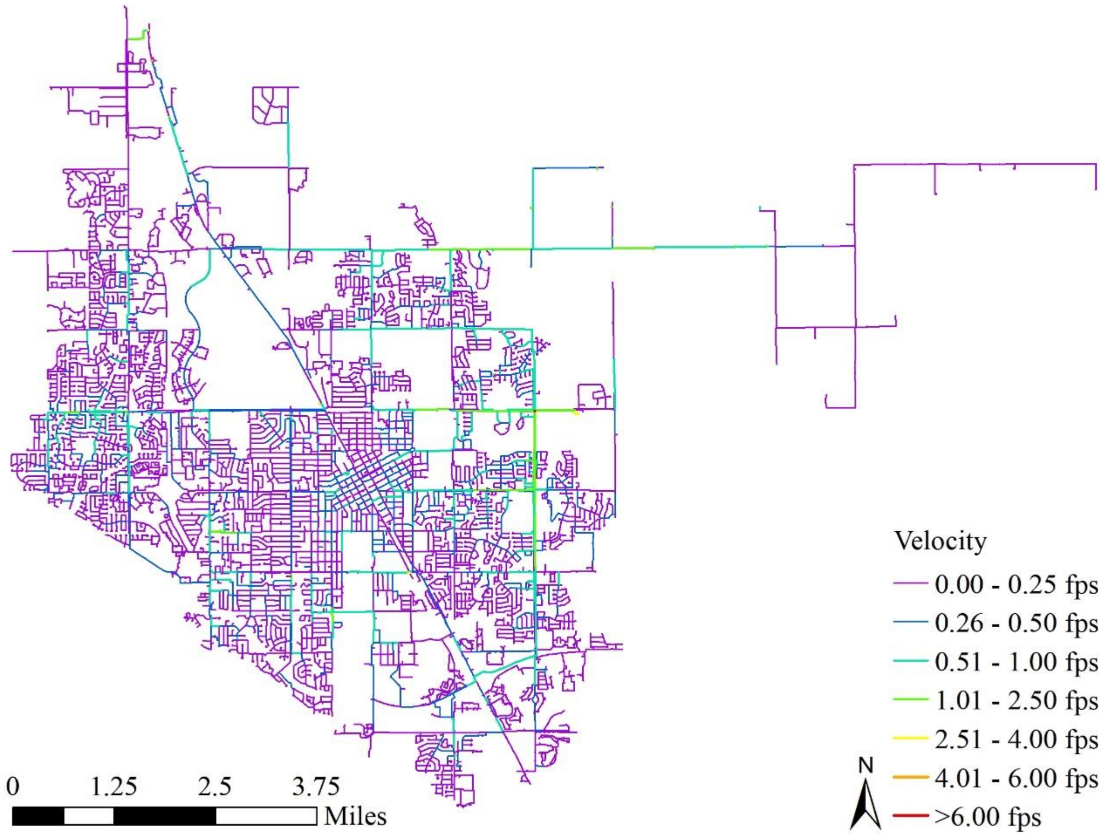

Figure 54. 2012 Single System Maximum Day Demands Velocity Results

2012 Single System with Peak Hour Demands

The third hydraulic evaluation for the 2012 single distribution system applies the peak hour demands of 45.6 MGD to the nodes in the system. The results for the peak hour demands show the lowest pressures, the lowest HGL, and the highest velocity. Still, the minimum pressure under peak hour demands is 27.9 psi, which is above the minimum of 25.0 psi, and the average pressure is 58.8 psi. Consistent with the previous demand scenarios, the lowest pressures are at the highest elevations and the highest pressures by the wells, as shown in Figure 55.

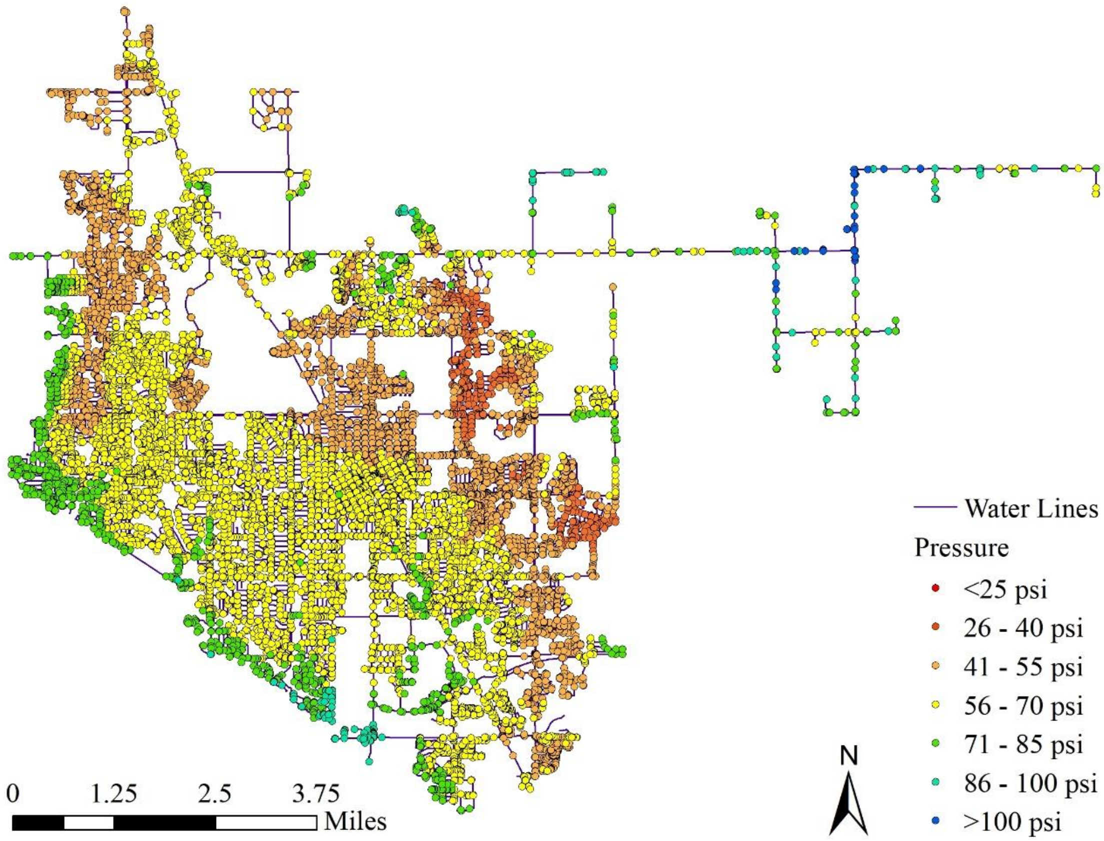

Figure 55. 2012 Single System Peak Hour Demands Pressure Results

The peak hour demand scenario generates the highest velocities of the four single system demand scenarios, with a few water lines having velocities above 6.0 fps. These water lines come from the WTP, so a higher velocity is expected because there is a higher flow from these lines. Additionally, there is one short, small pipe that acts as a bottleneck in the center of the distribution system. The rest of the distribution system experiences more appropriate velocities at less than 2.5 fps, as shown in Figure 56.

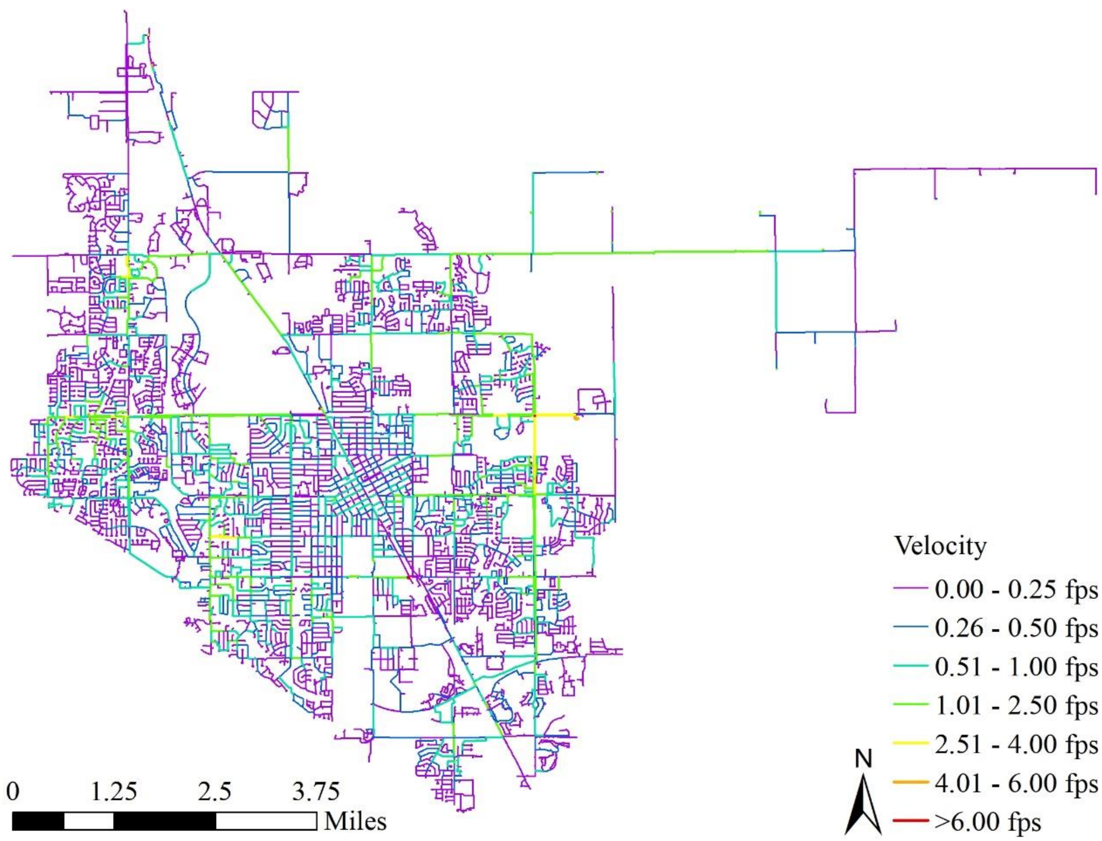

Figure 56. 2012 Single System Peak Hour Demands Velocity Results

Summary of 2012 Single Distribution System

In all, the 2012 single distribution system performed well in hydraulic simulations of average day, maximum month, maximum day, and peak hour demands. Most importantly, the pressures remained above 25.0 psi in all simulations, which indicates that the distribution system is adequately designed to meet the demands of the water distribution system in 2012.

2012 Dual Distribution System Hydraulic Evaluations

For the first dual distribution simulations, the 2012 demands are divided between potable and nonpotable demands, with the potable demands applied to the existing distribution system and the nonpotable demands applied to an imaginary secondary distribution system that closely mirrors the existing distribution system. The pressure requirements are the same for both systems as for the single distribution system, with a minimum pressure of 25.0 psi. The potable system is also evaluated for its capacity to provide adequate fire flow under maximum month and maximum day demands, with a minimum targeted fire flow of 1,000 gpm at each of the hydrants in the distribution system (ODEQ, 2012). The nonpotable distribution system is not evaluated for fire flow because the fire flow is kept on the potable system for this research.

2012 Potable Distribution System

The 2012 potable distribution system is evaluated with the same distribution system and water sources as the single distribution system. In 2012, the City of Norman did not isolate the potable demands from the nonpotable demands; instead, the dual distributions are purely theoretical. For this research, it was assumed that the existing distribution system would serve as the potable distribution system for a few reasons:

1) The potable system will be providing the fire flows;
2) The existing distribution system is already sized to provide fire flows; and
3) The existing distribution system already serves the entire customer base.

With that said, the below subsections provide the pressure and velocity results for the 2012 potable system during average day, maximum day, and peak hour demand scenarios.

2012 Potable System with Average Day Demands

The first analysis of the 2012 potable system is for average day demands, which are 9.50 MGD as determined in Chapter 3. With this demand, the 2012 potable system performs well. The minimum pressure is 31.6 psi and the average pressure is 62.2 psi.

The spatial distribution of pressures on the 2012 potable system during average day demands, as shown in Figure 57, looks very similar to the single system average day demands.

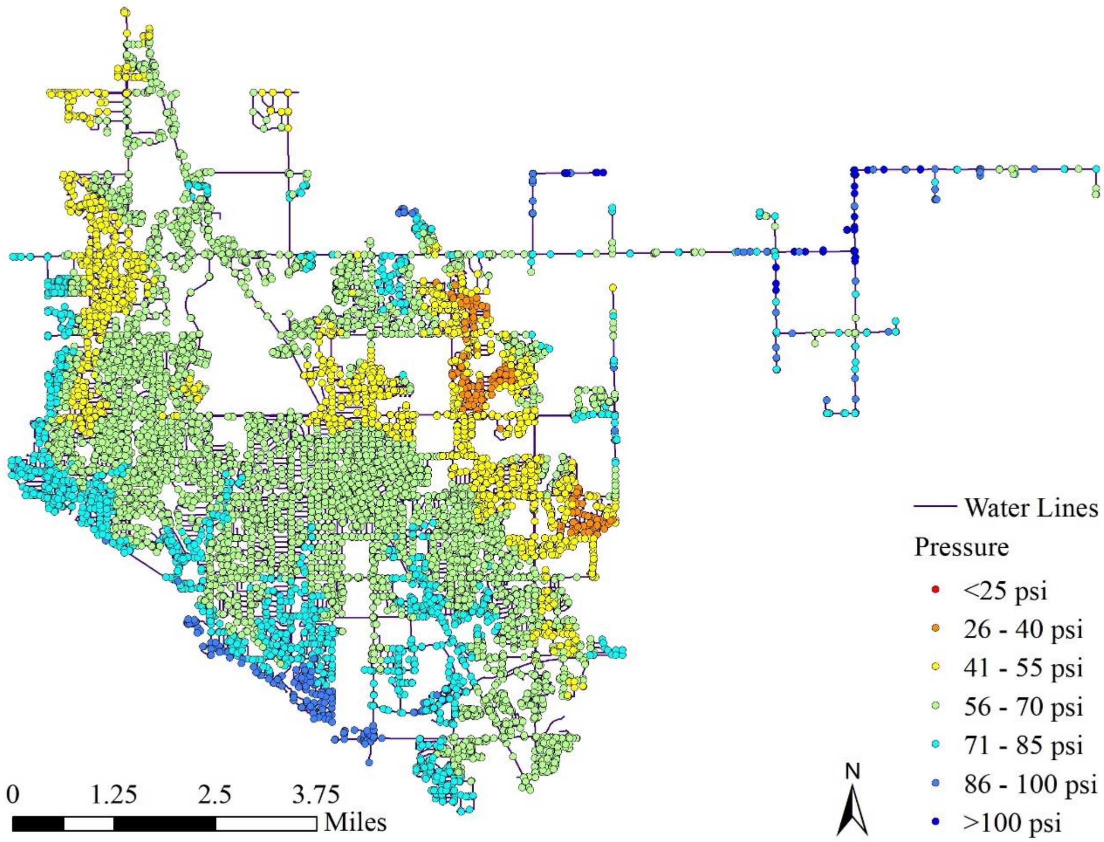

Figure 57. 2012 Potable System Average Day Demands Pressure Results

The velocities in the potable system are fairly low across the system, with the highest velocities occurring along the main streets. None of the velocities are greater than 6.0 fps, which is appropriate during average day demands, as shown in Figure 58.

Figure 58. 2012 Potable System Average Day Demands Velocity Results

2012 Potable System with Maximum Day Demands

The second hydraulic analysis for the 2012 potable distribution system evaluates the system with maximum day demands of 17.4 MGD. Overall, the system continues to perform well, with a minimum pressure of 31.2 psi and an average pressure of 61.9 psi. These results are higher than the ODEQ minimum pressure of 25.0 psi. The spatial distribution of the pressures under maximum day demands is shown in Figure 59.

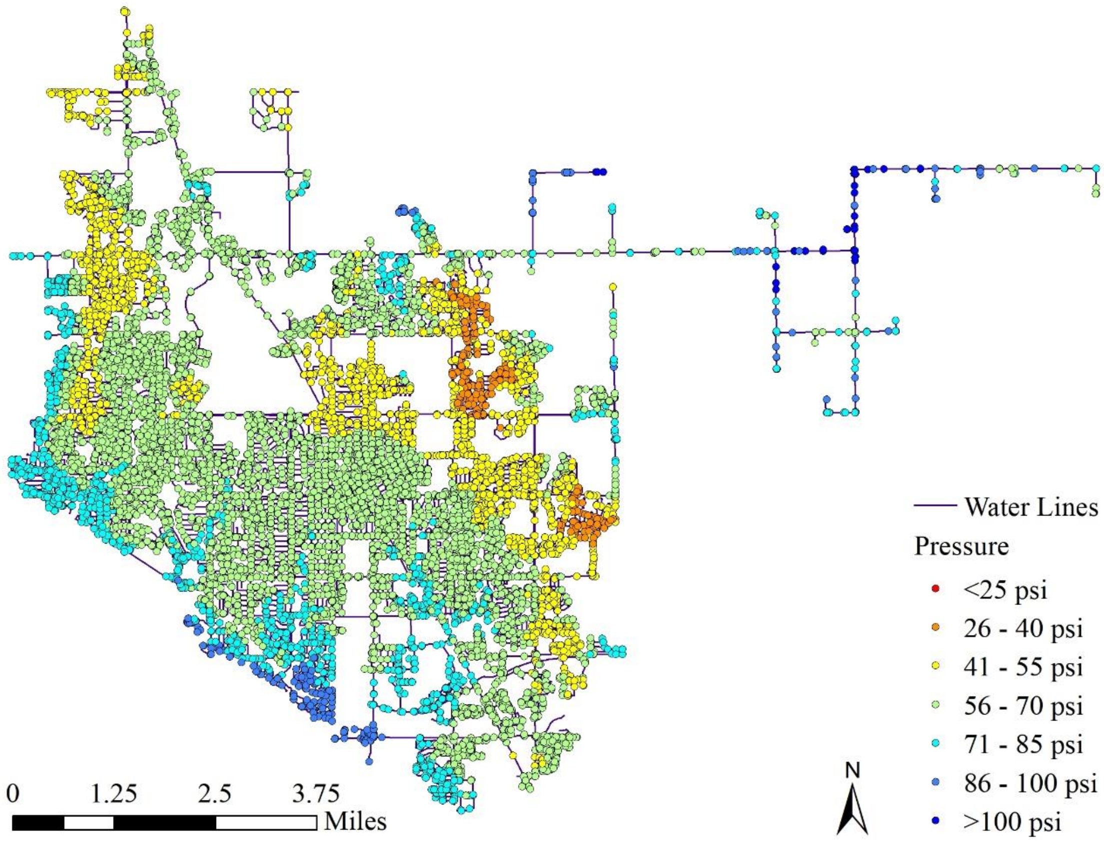

Figure 59. 2012 Potable System Maximum Day Demands Pressure Results

Because the fire flow demand remains on the potable system, the available fire flow under maximum day demands is also evaluated. In this demand scenario, 80 fire hydrants cannot provide the minimum 1,000 gpm (the minimum available fire flow is approximately 420 gpm) and 1,173 fire hydrants can provide at least 5,000 gpm. These results indicate that the potable system can typically provide adequate fire flow during the potable system maximum day demands. The spatial distribution of the available fire flow is shown in Figure 60.

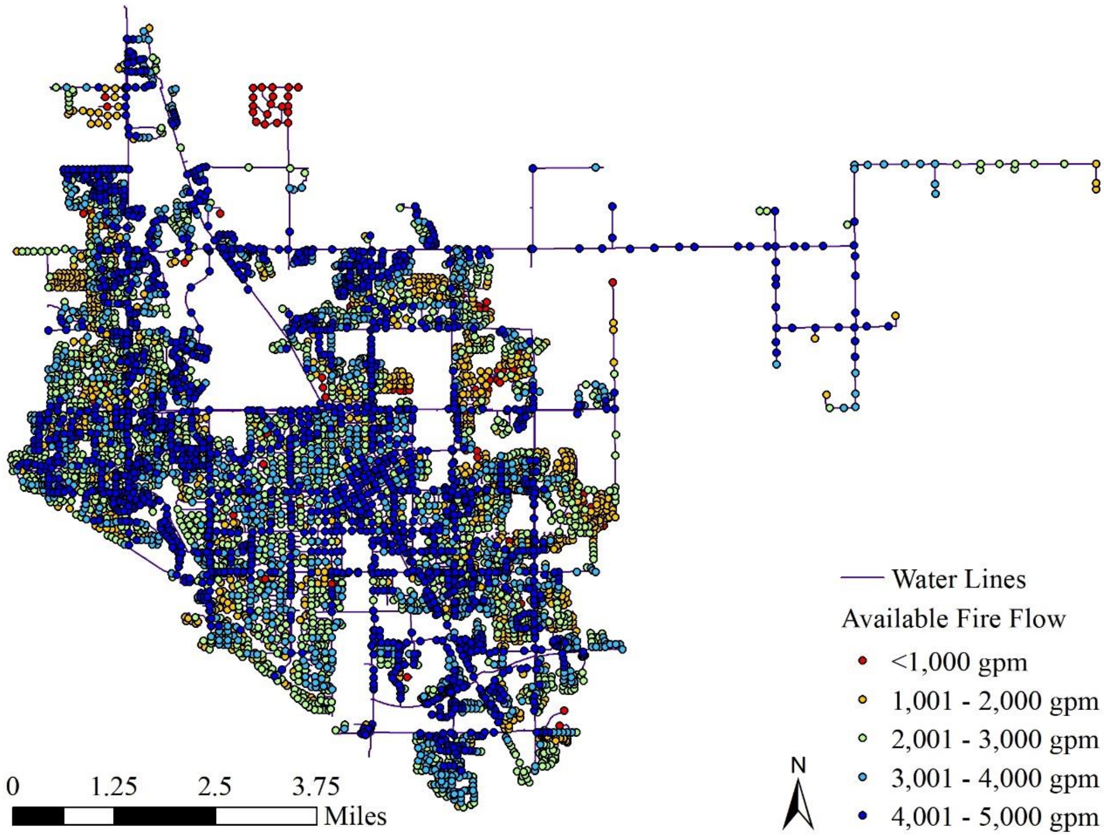

Figure 60. 2012 Potable System Maximum Day Demands Available Fire Flow

Finally for the potable distribution system under maximum day demands, the velocities are evaluated. Overall, the velocities in this demand scenario remain below 2.50 fps, which is much less than the maximum of 6.0 fps. This indicates that the water lines can provide the flow with appropriate velocities. The spatial distribution of the velocities can be seen in Figure 61.

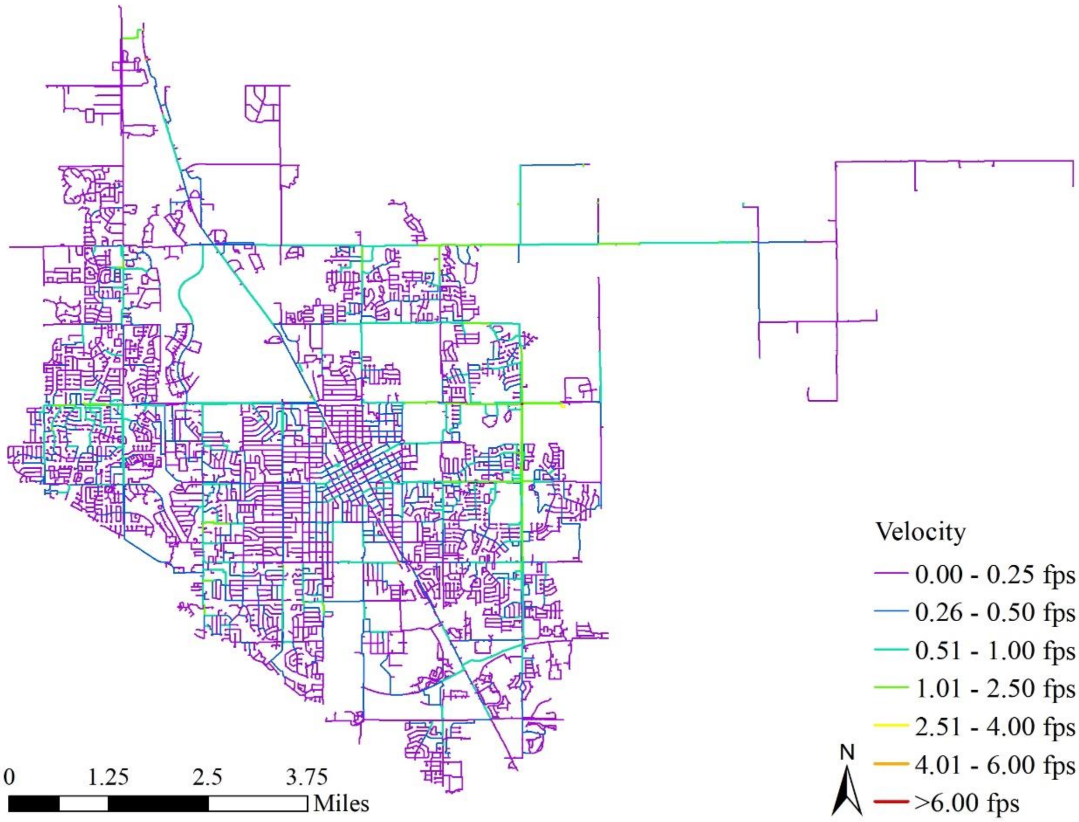

Figure 61. 2012 Potable System Maximum Day Demands Velocity Results

2012 Potable System with Peak Hour Demands

The third demand scenario for the potable distribution system is peak hour demands, with a demand of 34.5 MGD – more than 10 MGD less than the single distribution system peak demand. With the reduced demand, the minimum pressure in the distribution system is 29.3 psi, compared to 27.9 psi in the single distribution system under the peak hour demand. The average pressure is 60.1 psi, compared to 58.8 psi in the single distribution system under the peak hour demand. The spatial distribution of the pressures is shown in the figure below.

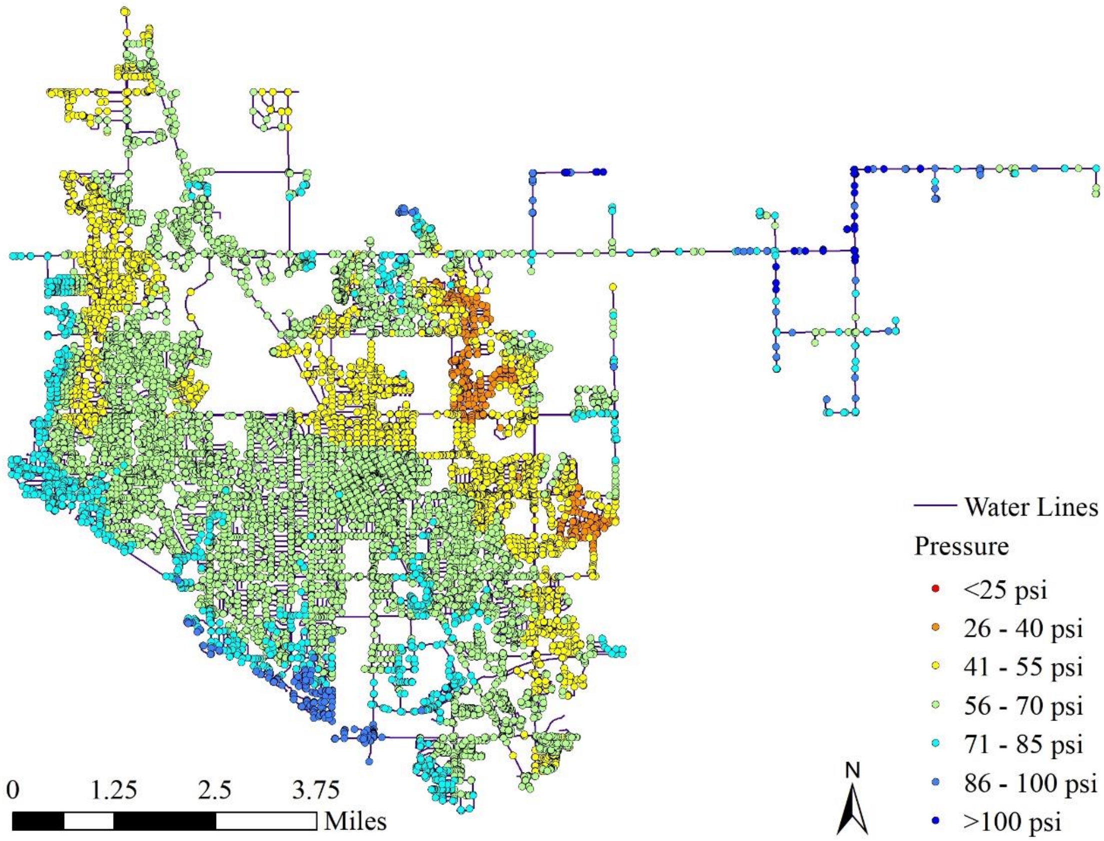

Figure 62. 2012 Potable System Peak Hour Demands Pressure Results

The velocities with the potable peak hour demands are the highest of all the potable demand simulations, yet all the velocities are less than 2.5 fps except for one short and small pipe that acts as a bottleneck in the center of the distribution system. The spatial distribution of the velocities is shown in the figure below.

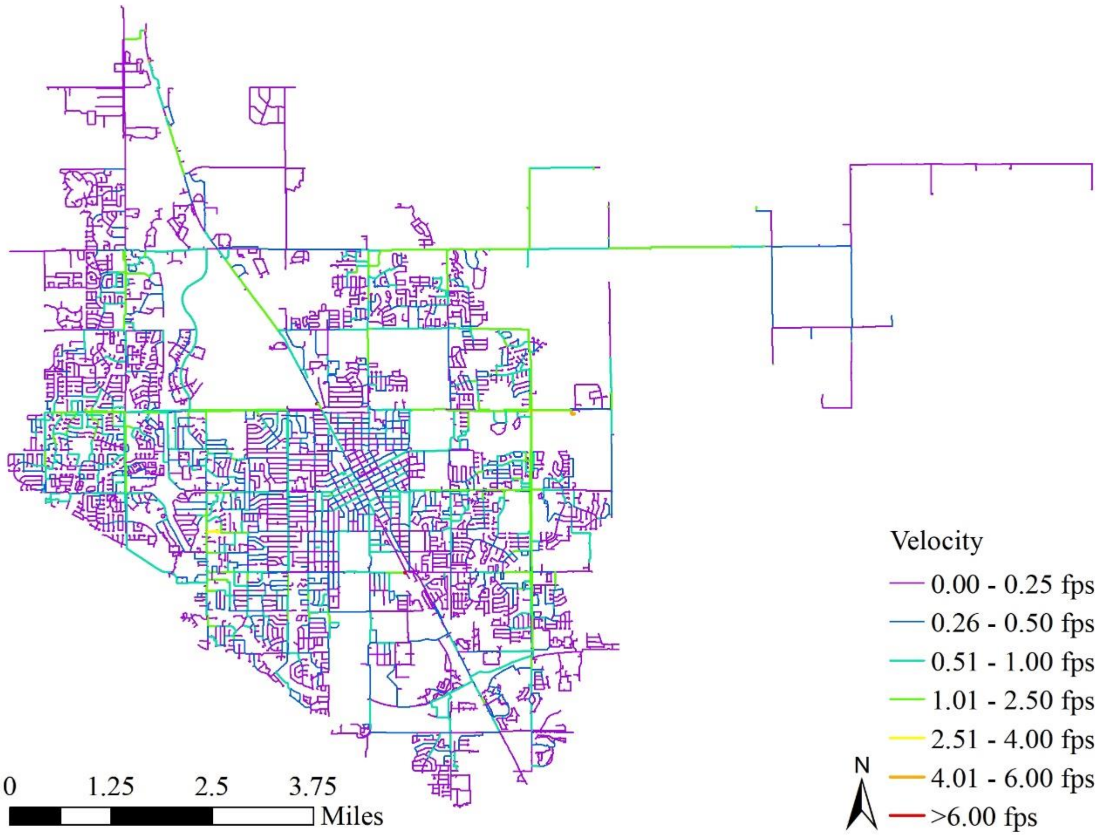

Figure 63. 2012 Potable System Peak Hour Demands Velocity Results

Summary of 2012 Potable Distribution System

In general, the potable distribution system performs better than the single distribution system does under the four demand scenarios. This is as expected because the potable demands are less than the total demands that are on the single distribution system. Most importantly, the pressures were simulated to all be over 25.0 psi, even under peak hour demands and the available fire flow capacity with maximum day demands was typically greater than the minimum flow of 1,000 gpm.

2012 Nonpotable Distribution System

The secondary distribution system supplies the nonpotable demands for 2012, which consist of the estimated irrigation demands as determined in Chapters 3 and 4.

These nonpotable demands are supplied only from the WRF and the 15 wells with high arsenic content. While the nonpotable demands exceed the capacity of the nonpotable water supplies and the estimated storage requirement, at approximately 275 MGD as determined in Chapter 3, is greater than what would be feasible even for a potable distribution system, for the purposes of the 2012 hydraulic simulations, it is assumed that there is adequate storage in the distribution system to supply the demands. This is simulated by not placing a control valve on the water line coming from the WRF, which allows the WRF to provide as much as can be provided through the connecting water line. During peak hour demands, this means that the WRF supplies over 90 percent of the demand.

The single and the potable distribution systems were both evaluated with a minimum required pressure of 25.0 psi under all demands, as set by ODEQ regulations; however, the nonpotable distribution system is evaluated with a minimum required pressure of 30.0 psi, as discussed in Chapter 4.

2012 Nonpotable System with Average Day Demands

The first demand scenario evaluates the 2012 nonpotable system with average day demands of 3.23 MGD. With this demand, the pressures are good overall, with an average pressure of 63.3 psi and a minimum pressure of 32.7 psi (located at the highest point in the system). Overall, the pressures mostly correlate with the topography in the distribution system, with higher pressures at lower elevations and lower pressures at higher elevations, as shown in Figure 64 below.

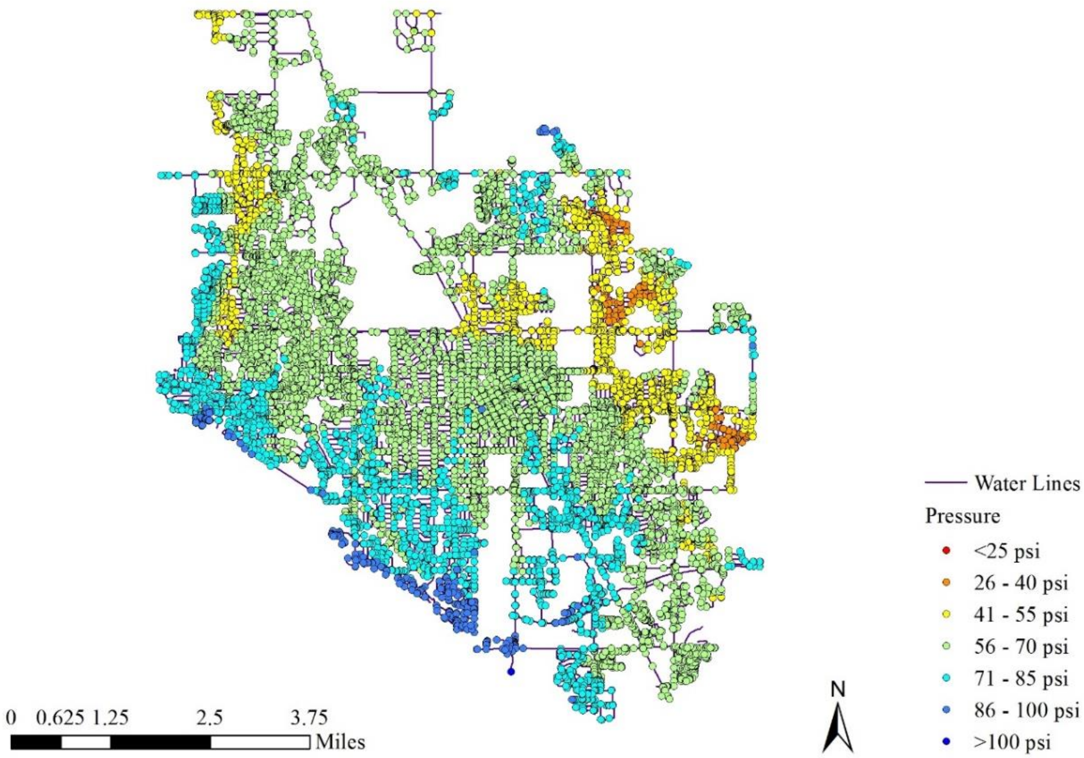

Figure 64. 2012 Nonpotable System Average Day Demands Pressure Results

The velocities during average day demands are all under 6.0 fps, with a maximum velocity of 3.7 fps. This indicates that the diameters are large enough to accommodate the flows during the average day demands. The velocity results for the 2012 nonpotable average day demands are shown in the figure below.

Figure 65. 2012 Nonpotable System Average Day Demands Velocity Results

2012 Nonpotable System with Maximum Day Demands

The second nonpotable demand scenario evaluates the 2012 nonpotable system with a maximum day demand of 10.1 MGD. The nonpotable system does not perform as well as it did with the average day demands. The demands are greater than the available supply, though it is assumed that the WRF provides the additional supply via hypothetical storage. Overall, the system maintains an average pressure of 56.6 psi, though the minimum pressure drops below 30 psi to 25.6 psi at the highest point in the distribution system. The pressure results for the 2012 nonpotable distribution system during maximum day demands are presented in Figure 65.

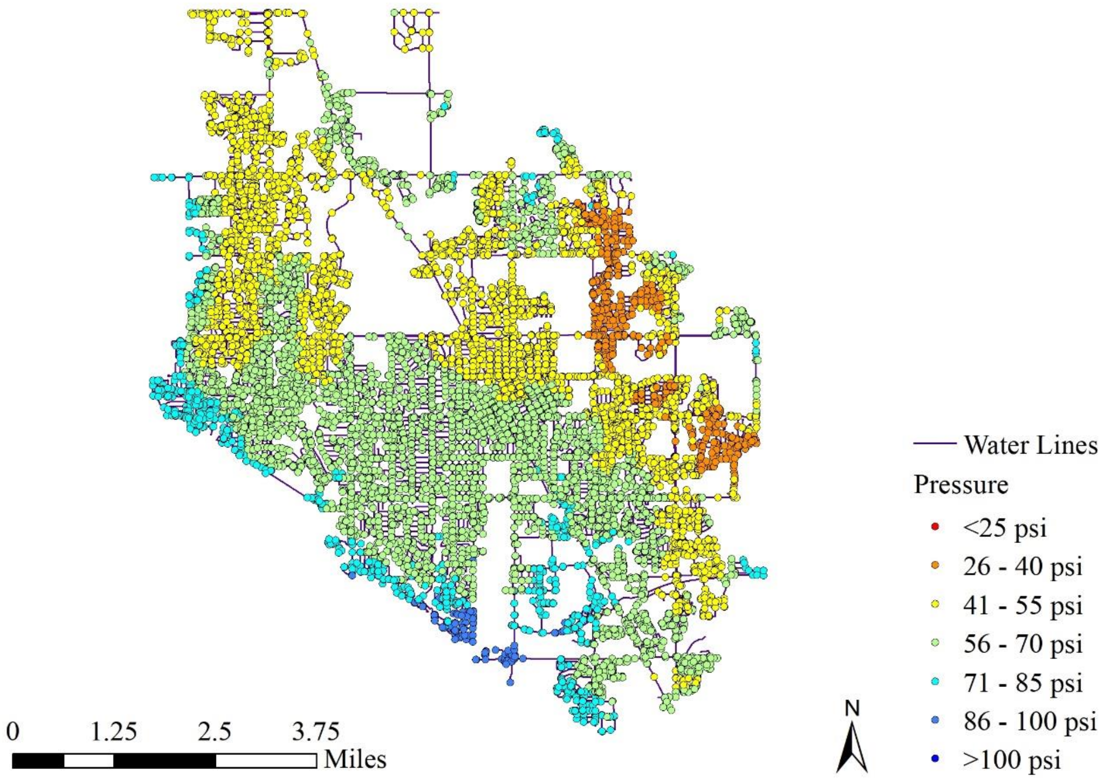

Figure 66. 2012 Nonpotable System Maximum Day Demands Pressure Results

The velocities in the water lines increase during maximum day demands, with a maximum velocity of 7.2 fps, though this only occurs in the water line from the WRF. Other than this water line, the velocities in the rest of the water lines are all less than 5.0 fps. The velocity results for the 2012 nonpotable maximum day demands are shown in the figure below.

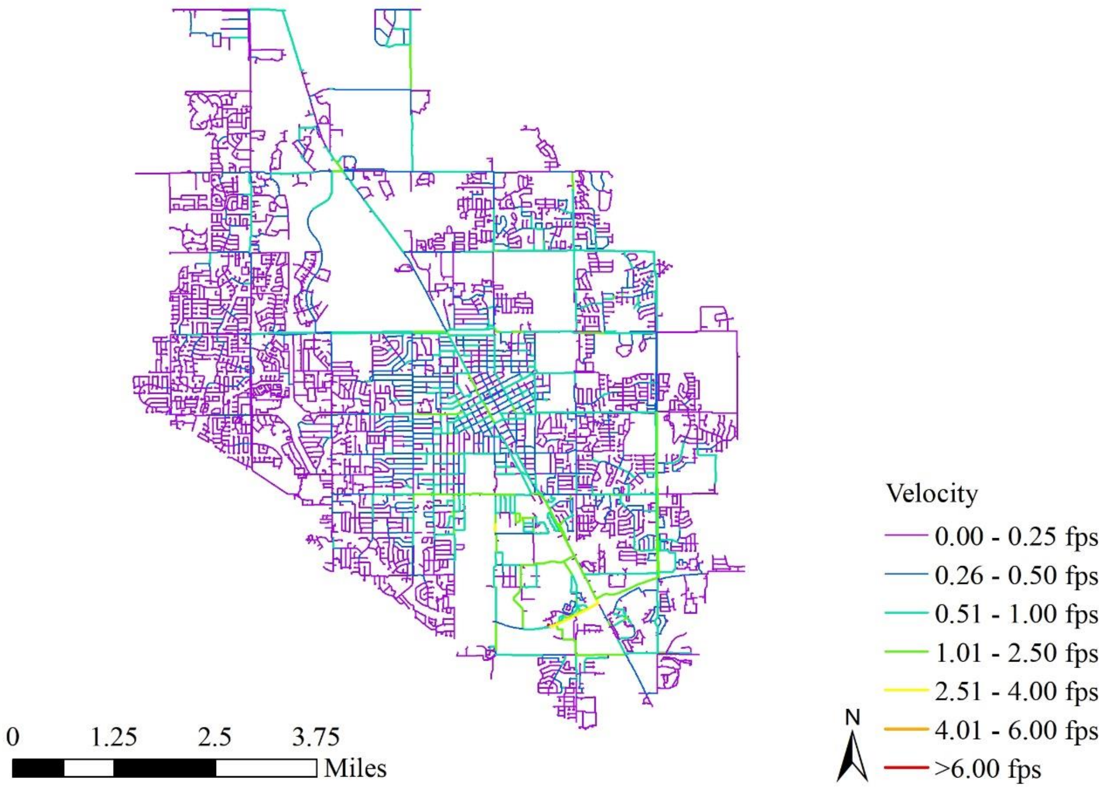

Figure 67. 2012 Nonpotable System Maximum Day Demands Velocity Results

2012 Nonpotable System with Peak Hour Demands

The third demand scenario evaluates the 2012 nonpotable peak hour demand with 34.5 MGD, which the sources cannot supply. However, the WRF is simulated to provide the excess supply via hypothetical storage, which results in large flows emanating from the WRF through the rest of the distribution system, while the nonpotable wells produce their maximum capacity. This causes some parts of the distribution system – mainly, a few nodes on the west side of Norman and nearly all the nodes on the east side of Norman – to experience very low pressures, with a minimum pressure of 6.5 psi. However, the majority of the distribution system has adequate pressures with an average pressure of 38.5 psi. The pressure results are provided in Figure 68 below.

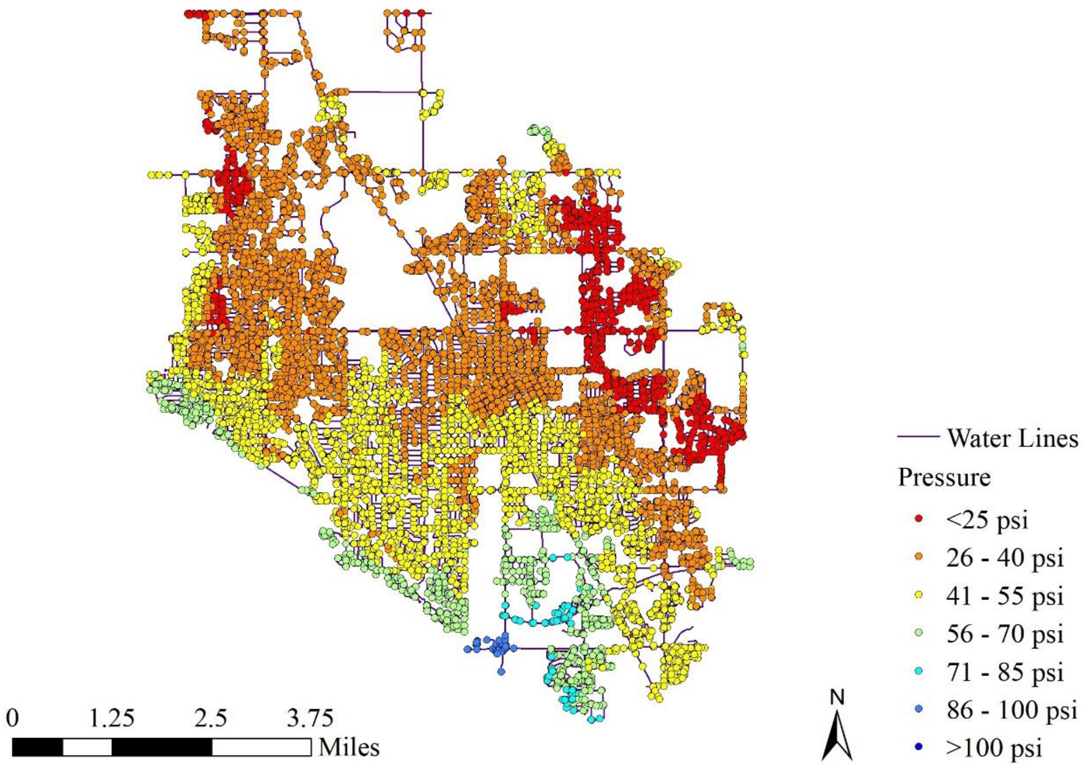

Figure 68. 2012 Nonpotable System Peak Hour Demands Pressure Results

The peak hour demands create the highest velocities of the demand scenarios, as expected. The highest velocities in the system are all on the water lines immediately adjacent to the WRF, with a maximum velocity of 14.3 fps. This is also as expected because the WRF provides a large percentage of the supply. The rest of the distribution system has a maximum velocity of 5.7 fps. The velocity results are provided in Figure 69.

Figure 69. 2012 Nonpotable System Peak Hour Demands Velocity Results

Summary of 2012 Dual Distribution System

For the 2012 dual distribution system, the potable system performs well during the various demand scenarios, while the nonpotable system is only sufficient for average day demands. Reducing the demands on the existing distribution system create better pressures and available fire flow results for the system, though this solution will not be feasible for extending the service life of the existing system because the nonpotable system cannot accommodate the nonpotable demands, as simulated. In particular, the peak hour nonpotable demands overwhelm the distribution system so much that there are negative pressures through much of the system. These negative pressures are likely caused though by forcing a large flow rate through a water line that is too small to convey it effectively, rather than any particular deficiencies in the system. Incorporating an odd-even watering scheme could potentially reduce the demands by approximately half, which is slightly less than the simulated maximum day demands. This would in turn raise pressures throughout the distribution system to match the results in Figure 62. This could make a nonpotable distribution system that serves all of Norman more feasible in that the system could provide adequate pressures to the users.

When comparing the 2012 dual distribution system to the 2012 single distribution system, the dual distribution system does not perform as well because the nonpotable system must be included. The pressures are so low in the nonpotable system during peak hour demands that the dual distribution system cannot reliably serve the customers of Norman for urban irrigation. A key take-away is that reducing the demands on the existing distribution system would benefit it, whether that happens through conservation or creating a dual distribution system to serve at least some of the nonpotable demands.

2062 Single Distribution System Hydraulic Analyses

The single distribution scenarios are evaluated with both potable and nonpotable demands on the same distribution system. The 2062 single distribution system is modeled with the new 12-inch and 16-inch water lines in the expanded areas of the distribution system and the new elevated storage tower in the southeastern corner, as discussed in Chapter 4. All demands are supplied by the WTP and the 26 active wells. Overall, the 2062 distribution performs well, though the pressures in the southeastern corner and in Hall Park are below the minimum ODEQ pressure of 25.0 psi during the peak hour demand scenario, as will be shown in the relevant subsection.

2062 Single System with Average Day Demands

The first demand scenario for the 2062 single system evaluates the system with the estimated average day demand of 25.05 MGD. The results show that it performs well in terms of the evaluation criteria (e.g., pressure, velocity, and HGL), as expected. The minimum system pressure is 30.9 psi and the average pressure is 61.8 psi. The highest pressures are typically in the lowest elevations near the Canadian River on the southeastern side, while the lowest pressures are at the highest elevations and in the southeastern corner of the expanded distribution system. This is shown in the figure below.

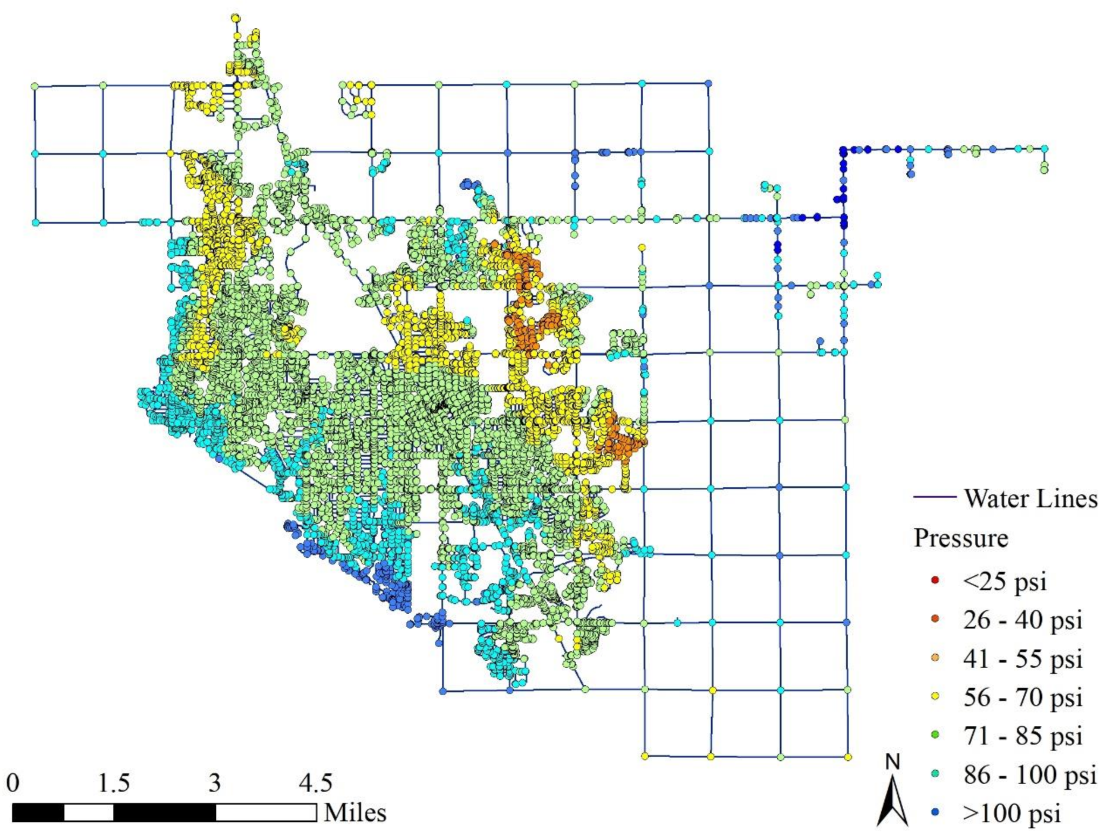

Figure 70. 2062 Single System Average Day Demands Pressure Results

Finally, most of the velocities in the distribution system during 2062 average day demands are all well below the maximum velocity of 6.0 fps. There is one line with a velocity of 7.4 fps, which occurs in a 4-inch water line that connects a well to the distribution system. The majority of water lines have velocities less than 0.5 fps and all the other water lines have velocities less than 2.5 fps, as shown in the figure below.

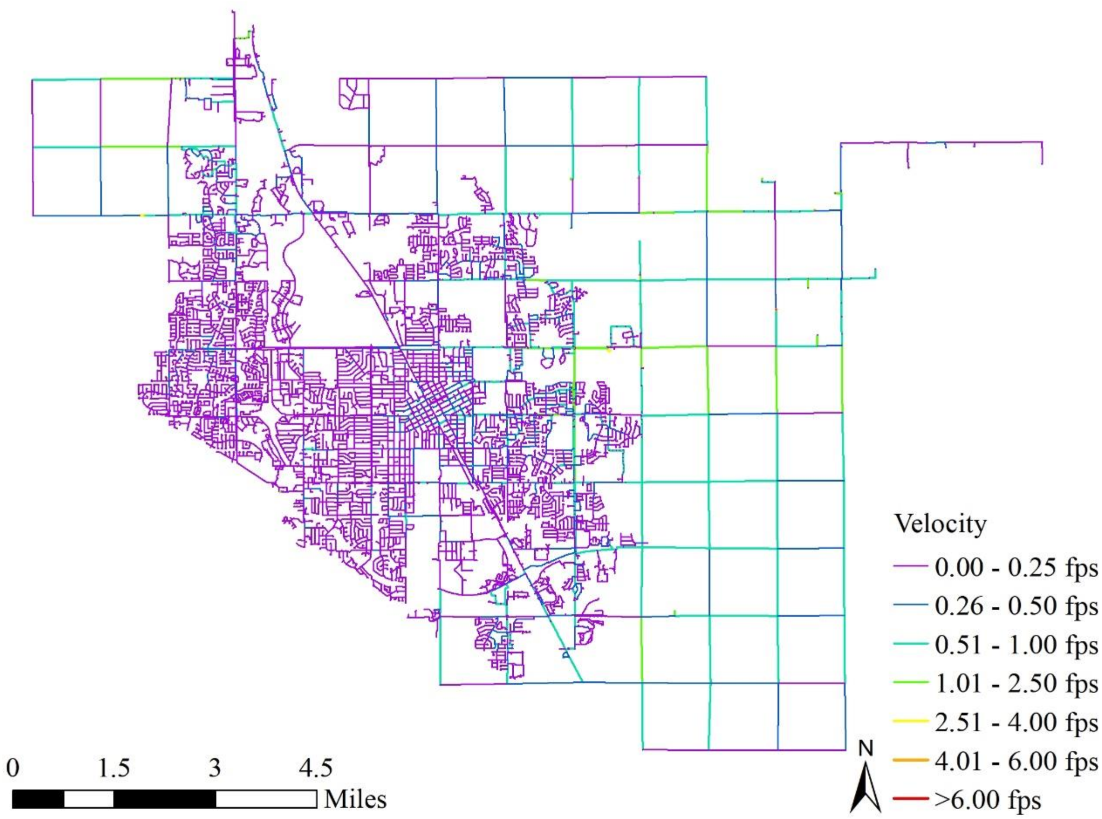

Figure 71. 2062 Single System Average Day Demands Velocity Results

2062 Single System with Maximum Day Demands

Next, the 2062 single system is simulated with a total maximum day demand of 45.8 MGD. The results for pressure, HGL, available fire flow, and velocity show that the 2062 distribution system continues to perform well with the increased demands. The pressures are still all above the minimum 25.0 psi with a minimum pressure of 28.9 psi and an average pressure of 60.1 psi. The junctions with reduced pressures mirror the results with the 2012 single distribution system, as expected. Additionally, the expanded portion of the distribution system performs well – in part because the additional water tower helps maintain adequate pressures in the southeastern corner of the distribution system. The spatial distribution of the pressures is shown in Figure 72 below.

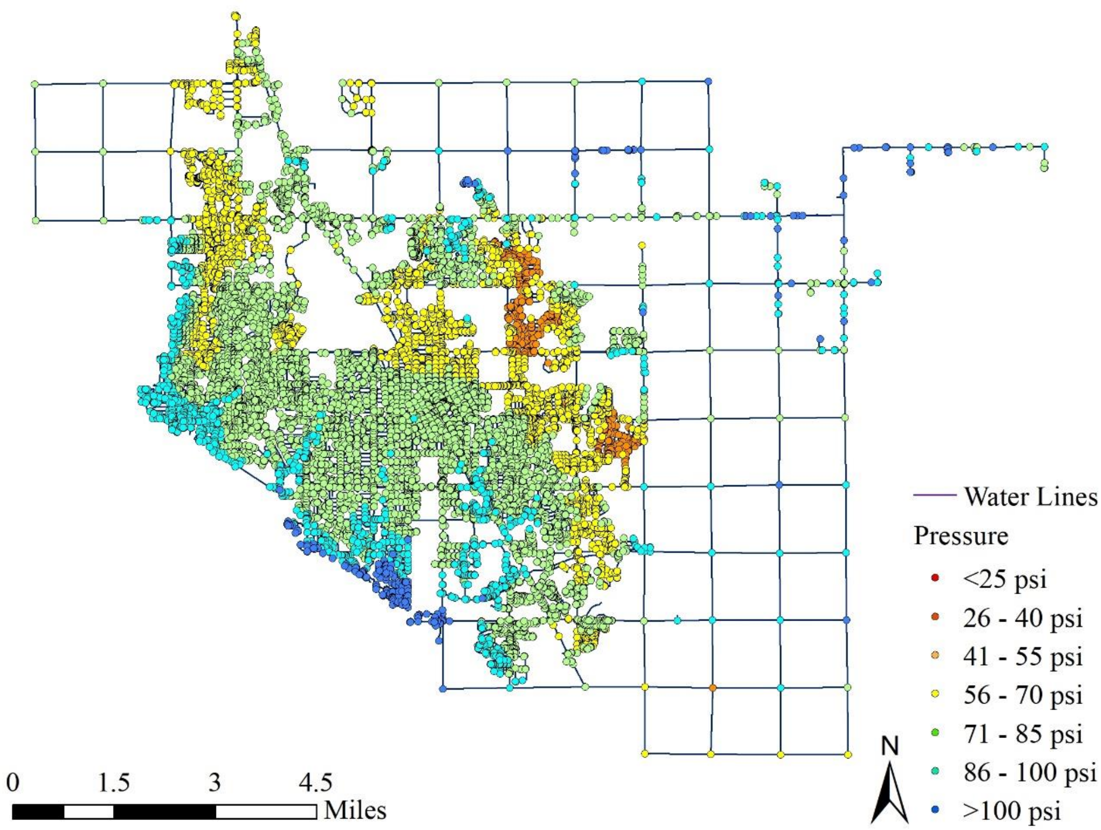

Figure 72. 2062 Single System Maximum Day Demands Pressure Results

During 2062 maximum day demands, 99 percent of the fire hydrants can provide fire flows greater than 1,000 gpm and 22 percent of the fire hydrants can provide fire flows greater than or equal to 5,000 gpm. All the fire hydrants on the new water lines can provide at least 2,000 gpm in available fire flow. The spatial distribution of fire flow is provided in Figure 73.

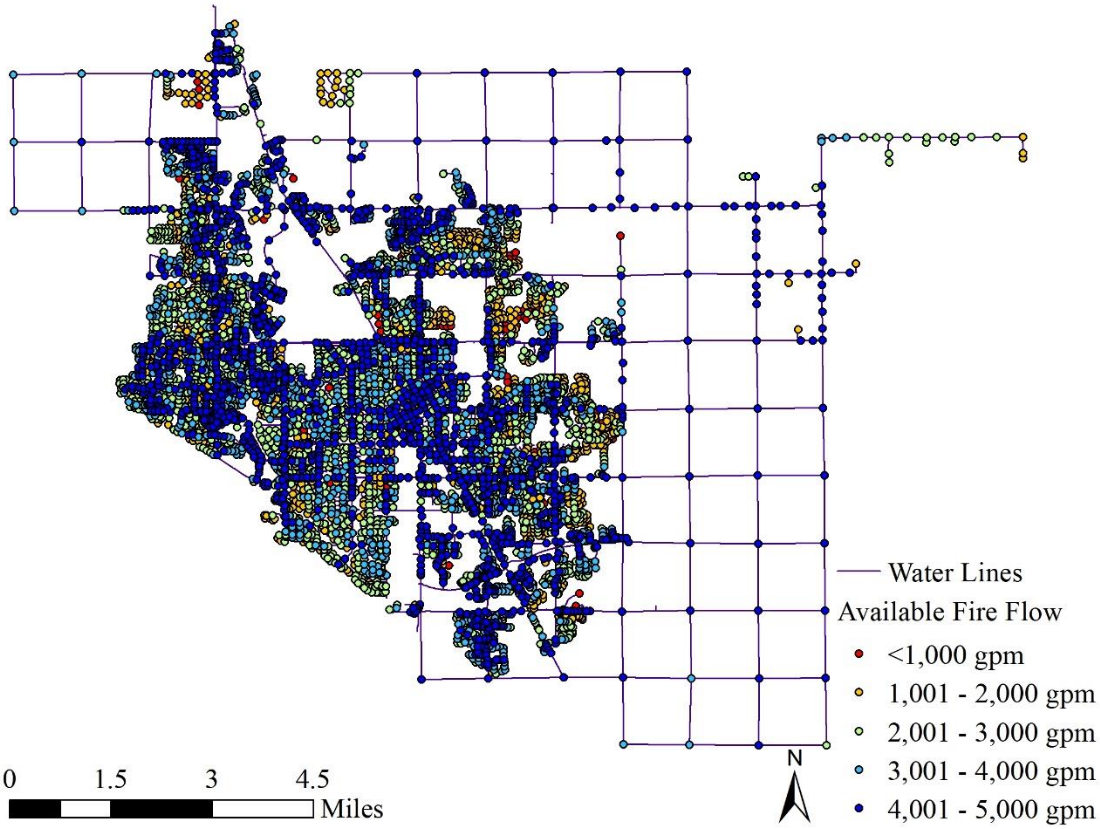

Figure 73. 2062 Single System Maximum Day Demands Available Fire Flow

Finally, the velocities in the 2062 single system during maximum day demands are nearly all below 6.0 fps, except for the water lines connecting the water towers to the distribution system, which have a maximum velocity of 7.9 fps. The rest of the water lines have velocities less than 4.0 fps, as shown in the figure below.

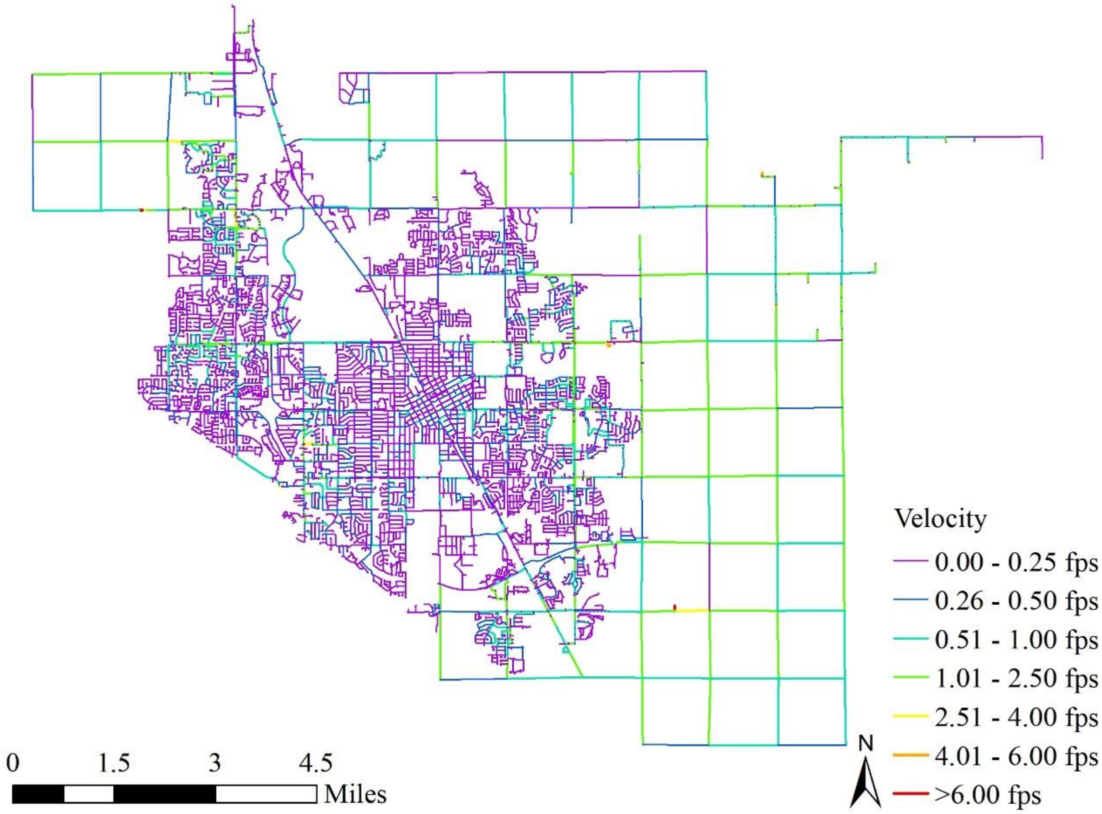

Figure 74. 2062 Single System Maximum Day Demands Velocity Results

2062 Single System with Peak Hour Demands

The final hydraulic analysis for the 2062 distribution system simulates the peak hour demand of 90.9 MGD. This is the greatest flow through the distribution system of all the demand scenarios. The minimum pressure in the distribution system is 19.1 psi, which is below the minimum allowed pressure, at one particular high point in the system, and a few other junctions have pressures below 25.0 psi. The below-minimum pressure only occurs during peak hour demands, though four of the junctions are located in the southeastern corner of the expanded distribution system. Overall, the average pressure is 52.5 psi. The spatial distribution of the pressure results is shown in the figure below.

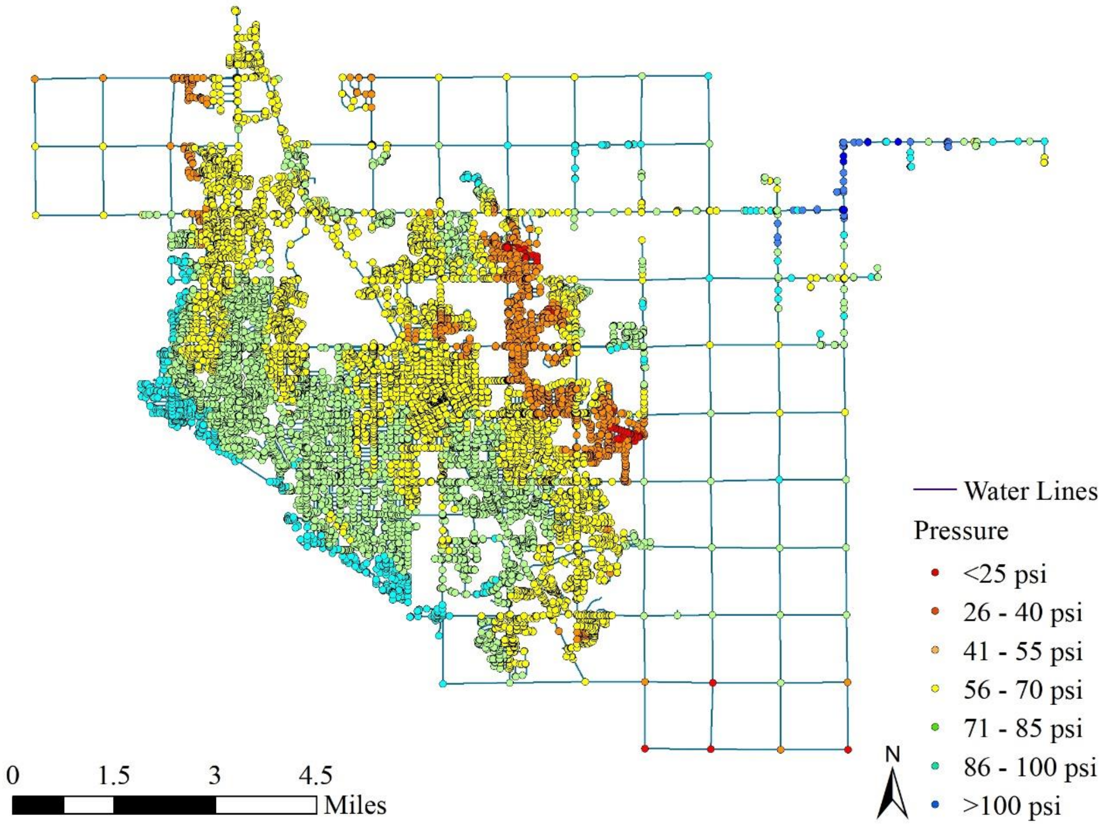

Figure 75. 2062 Single System Peak Hour Demands Pressure Results

Finally, the velocity results for peak hour demands show that nearly all the water lines have velocities less than 6.0 fps during peak hour demands, though all the water lines connecting water towers to the distribution system have velocities above 10 fps with a maximum velocity of 18.2 fps. In general, the water line diameters are large enough to accommodate the peak demands, though some of the water lines connecting the wells to the distribution system should be increased to accommodate the maximum flows. The velocity results are provided in the figure below.

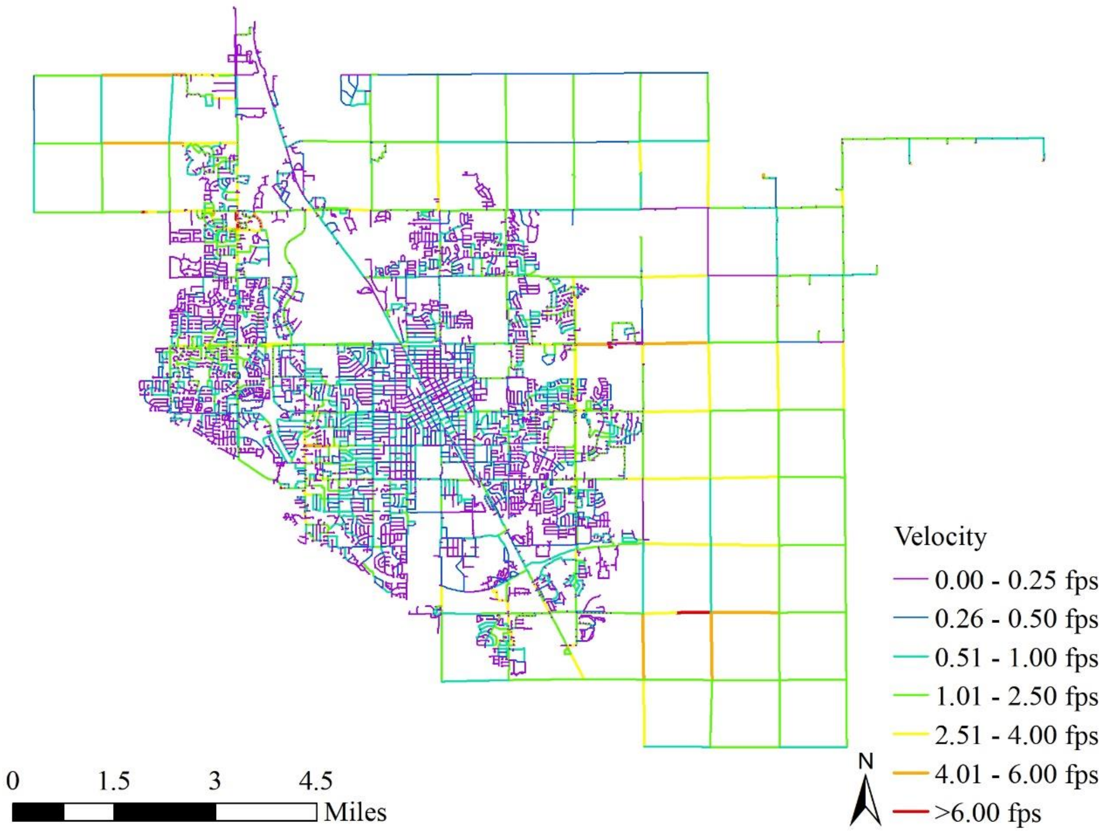

Figure 76. 2062 Single System Peak Hour Demands Velocity Results

2062 Dual Distribution System Hydraulic Analyses

In the 2062 simulations of the dual distribution system, the demands are divided between the potable distribution system, which is the single distribution system modeled above, and two designs for the nonpotable distribution system. As with the 2012 hydraulic models, the potable distribution system performs well, particularly when compared to the single distribution system. However, the nonpotable distribution system is limited by the supply capacity of the nonpotable sources. During the 2012 simulations, the WRF hypothetically provided the additional supply to simulate long-term storage. For the 2062 simulations, the service area is limited to a maximum day demand of 3.8 MGD. Two service areas are evaluated, as discussed in Chapter 4. The first service area includes areas close to the sources and some parts of the expanded distribution system. The second area only serves the expanded distribution system, though it still cannot supply all the estimated demand. The two options perform well during average and maximum day demands. The hydraulic results are presented below.

2062 Potable Distribution System

The 2062 potable distribution performed comparatively better than the 2062 single distribution system because it had less demand on it than the single system did. This happened with the 2012 distribution systems also. For the evaluation, the ODEQ regulations still apply to the potable distribution system, with minimum allowed pressures of 25 psi during any demand scenario and maximum velocities of 6.0 fps during average day demands. The results for average day, maximum day, and peak hour demands are provided below, with available fire flow included for the maximum day demand scenario.

2062 Potable System with Average Day Demands

The first hydraulic analysis for the 2062 potable system evaluates how the pressures and velocities with the 2062 average day demand of 18.9 MGD, which is the baseline demand for the single distribution system and the additional nonpotable demands that cannot be accommodated by the nonpotable sources. In general, the entire distribution system performs well, with an average pressure of 62.0 psi. The minimum pressure is 31.2, which is above the minimum allowed pressure. The pressure results are shown in Figure 77 below.

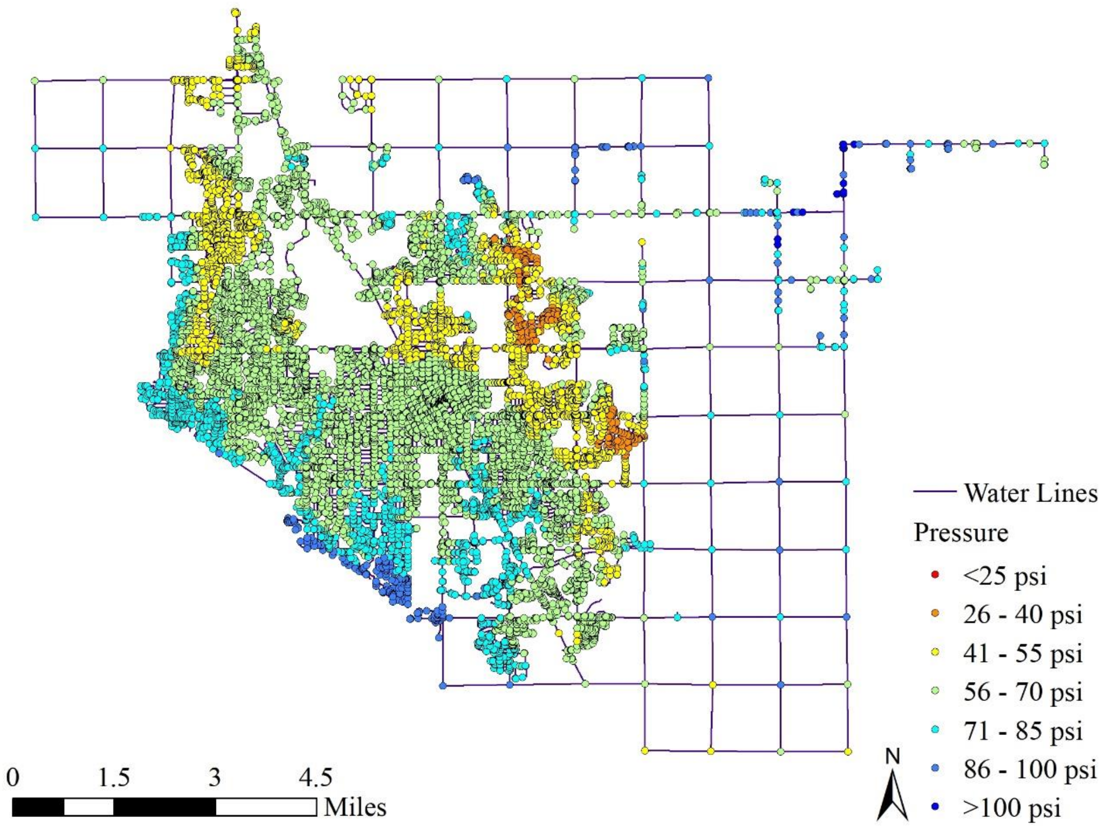

Figure 77. 2062 Potable System Average Day Demands Pressure Results

The velocity results for the potable system are similar to the results for the single distribution system, with a maximum velocity of 7.4 fps that occurs in a water line connecting a well to the rest of the system. However, the velocities are slightly lower overall than the velocities in single distribution system with average day demand. The velocity results are shown in the figure below.

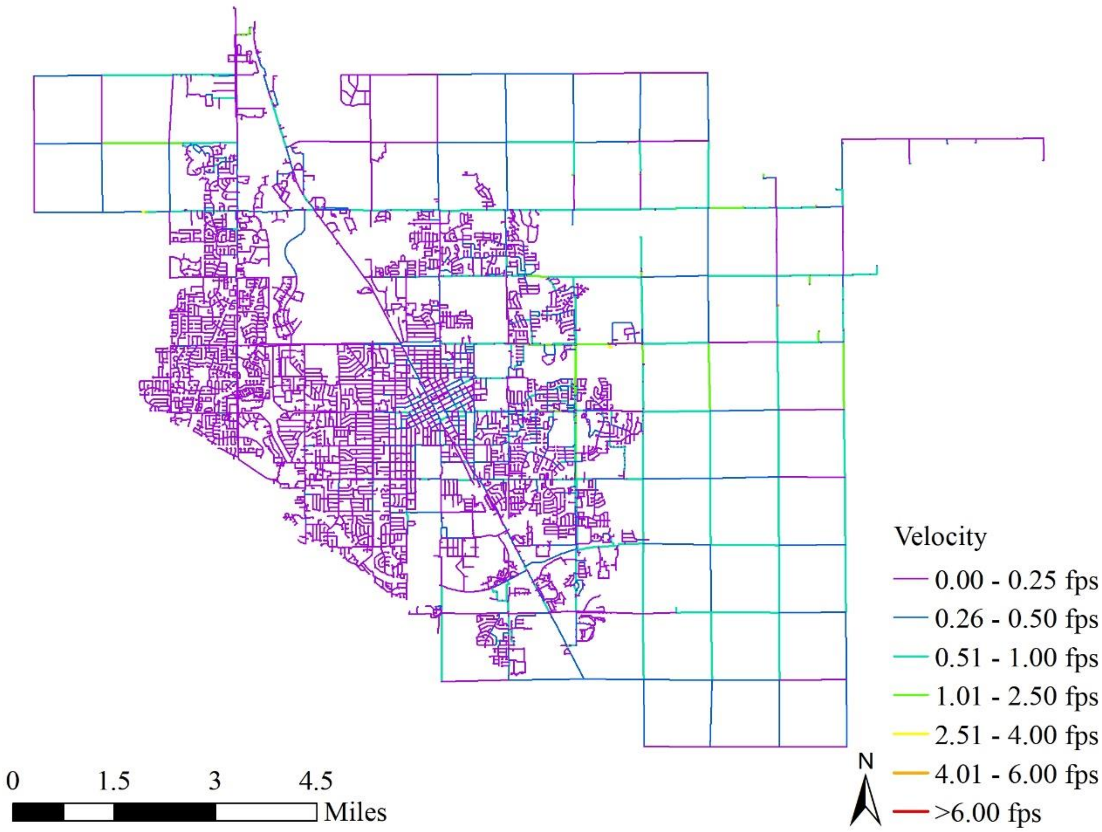

Figure 78. 2062 Potable System Average Day Demands Velocity Results

2062 Potable System with Maximum Day Demands

The second hydraulic analysis evaluates how the potable system responds with maximum day demands of 34.7 MGD. The increased demand lowers the pressures in the system somewhat. The average pressure is 61.1 psi, with a minimum pressure of 30.1 psi. Consistent with other pressure results, the lowest pressures are at the highest points in the system and the highest pressures are primarily at the lowest points in the system. The pressure results are shown in Figure 79 below.

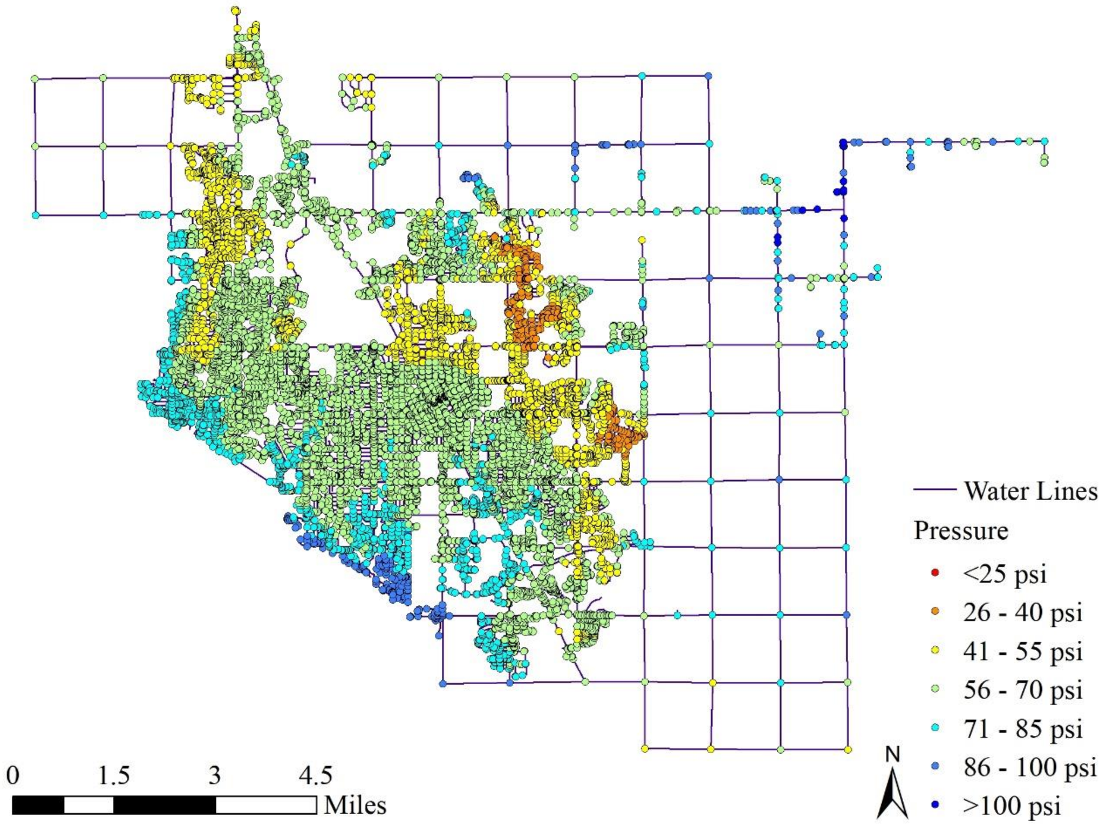

Figure 79. 2062 Potable System Maximum Day Demands Pressure Results

During the maximum day demands, the potable system can provide adequate fire protection. The results show that 99 percent of the fire hydrants can provide at least 1,000 gpm and 22 percent of fire hydrants can provide at least 5,000 gpm. This shows that the potable system can provide fire protection for most of the service area, as shown in Figure 80 below.

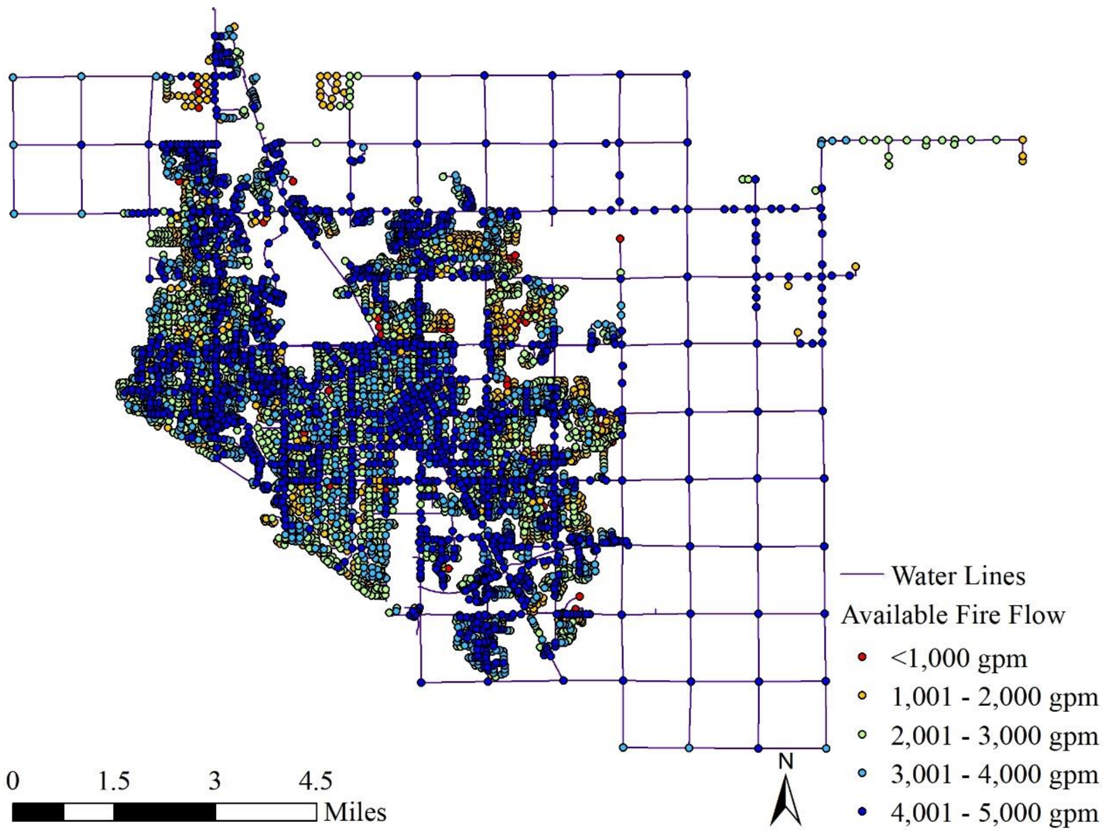

Figure 80. 2062 Potable System Maximum Day Demands Available Fire Flow

The velocity results for the maximum day demand are nearly the same as for the average day demand. The maximum velocity is again 7.4 fps in the same water line connecting a well to the system, because the flow from the well is limited by the maximum flow from the well. The velocities throughout the system increased slightly, though none of the other water lines have velocities greater than 6.0 fps. The velocity results are shown in Figure 81 below.

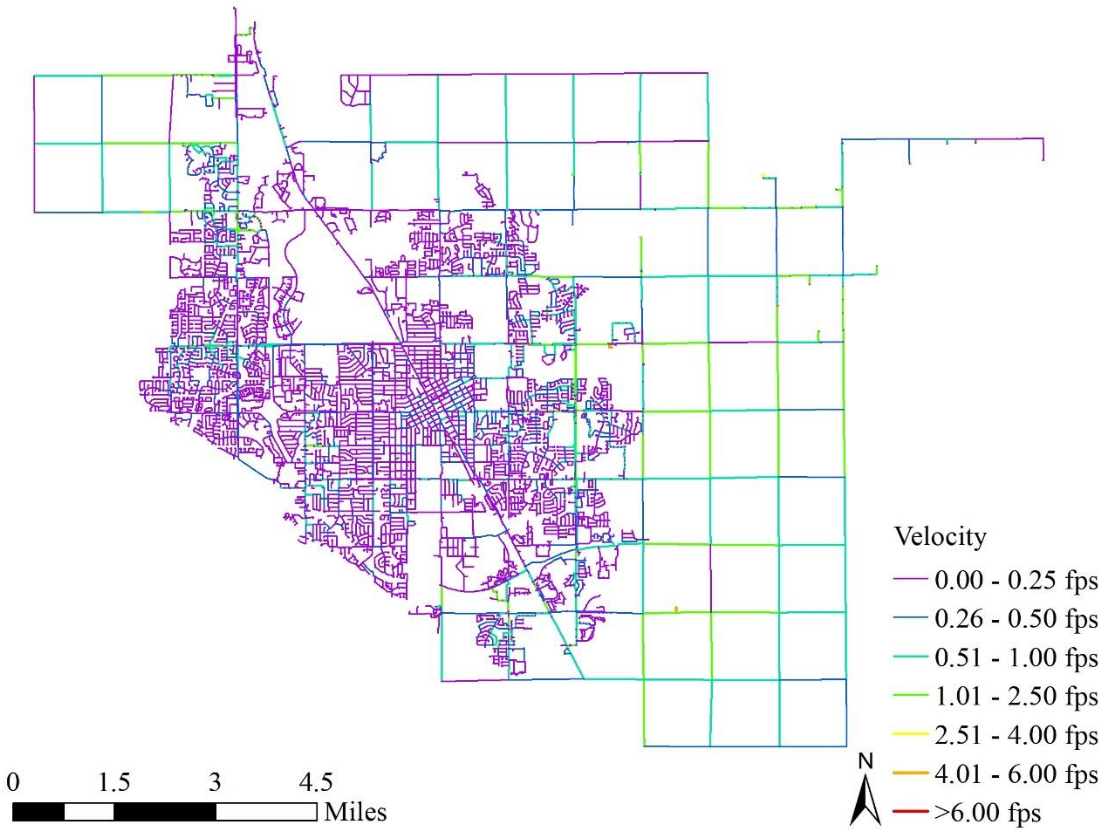

Figure 81. 2062 Potable System Maximum Day Demands Velocity Results

2062 Potable System with Peak Hour Demands

The final hydraulic analysis for the potable system evaluates the system with peak hour demands of 68.8 MGD. This demand puts the most stress on the distribution system, though all the pressures remain above 25 psi. The minimum pressure is 25.8 psi and the average pressure is 57.4 psi. These results show that the potable distribution system can provide the flows for the distribution system while sustaining adequate pressures. The pressure results are shown in Figure 82 below.

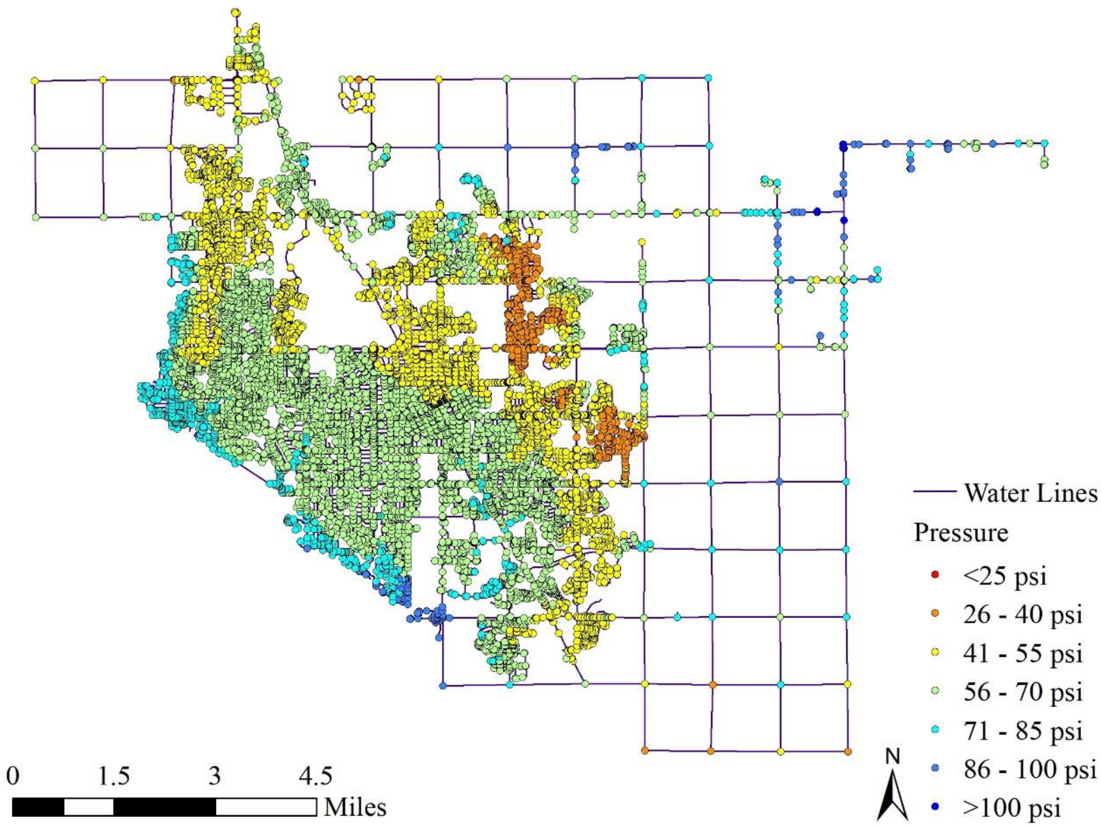

Figure 82. 2062 Potable System Peak Hour Demands Pressure Results

The velocity results during peak hour demands have a maximum velocity of 12.2 fps, which occurs in the water line connecting the new water tower to the distribution system. A few of the water lines have increased velocities, but all of these water lines connect the water towers to the distribution system, and none of the water lines in the distribution system itself have velocities greater than 6.0 fps. The velocity results are shown in Figure 83 below.

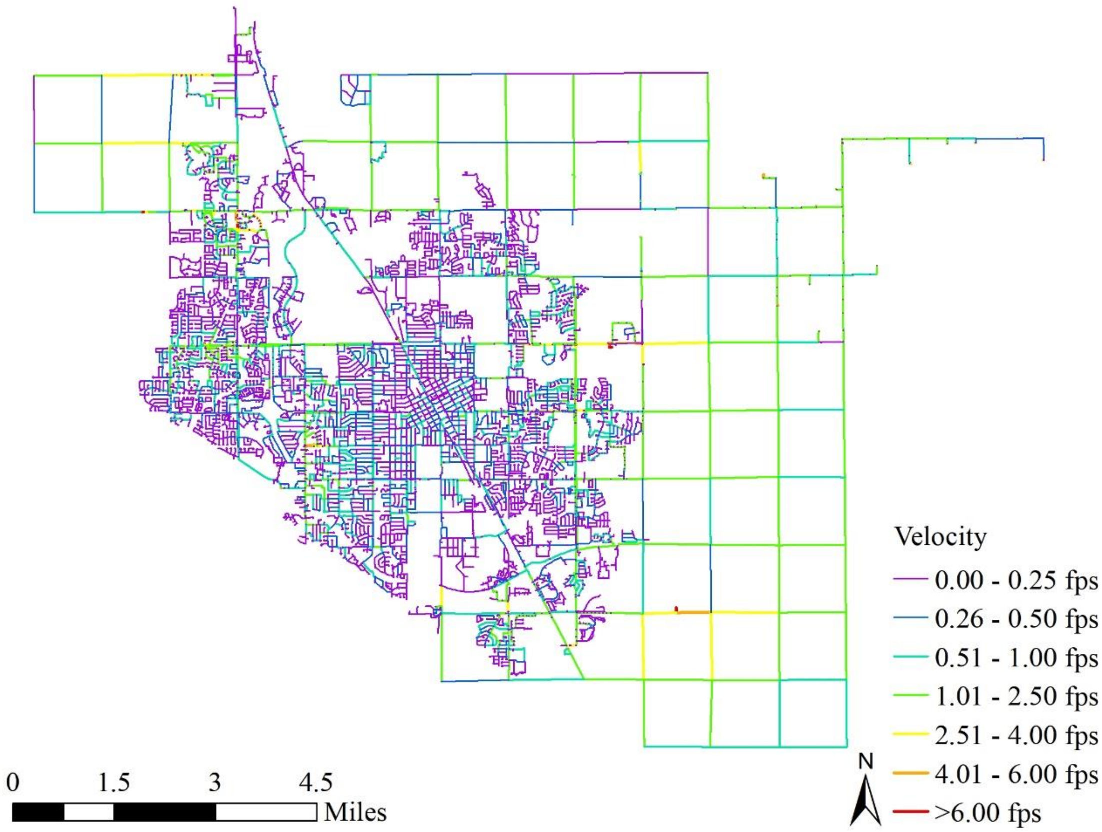

Figure 83. 2062 Potable System Peak Hour Demands Velocity Results

2062 Nonpotable Distribution System

As with the 2012 nonpotable distribution system, the nonpotable demands are supplied only from the WRF and the 15 wells with high arsenic content. However, the size of the nonpotable distribution system is restricted to the area with a maximum month demand of 3.8 MGD, which is the maximum supply of the nonpotable water sources without storage. For the 2062 nonpotable distribution system, two service area design options are presented. As previously discussed, the first service area covers the areas closest to the sources and parts of the expanded distribution system. The second service area covers most of the expanded distribution system. An assumed 400 MG of storage is included for the peak hour demand analysis because the system can supply demands up to the maximum day demands without storage, and this storage is assumed to be located at the WRF. This volume was determined in Chapter 3. The hydraulic results are provided below.

2062 Nonpotable System with Average Day Demands

While the total estimated 2062 nonpotable average day demand is 6.45 MGD, the nonpotable distribution is limited to a service area with a maximum day demand of 3.8 MGD. This equates to an average day demand of 2.1 MGD, using the peaking factors establish in Chapter 3. For the two service areas, Design 2 performs slightly better than Design 1, with minimum pressures of 36.1 psi and 32.8 psi, respectively. The average pressure in Design 1 is 57.2 psi and the average pressure in Design 2 is 60.0 psi. The pressure results for both designs are shown side by side in Figure 84 below.

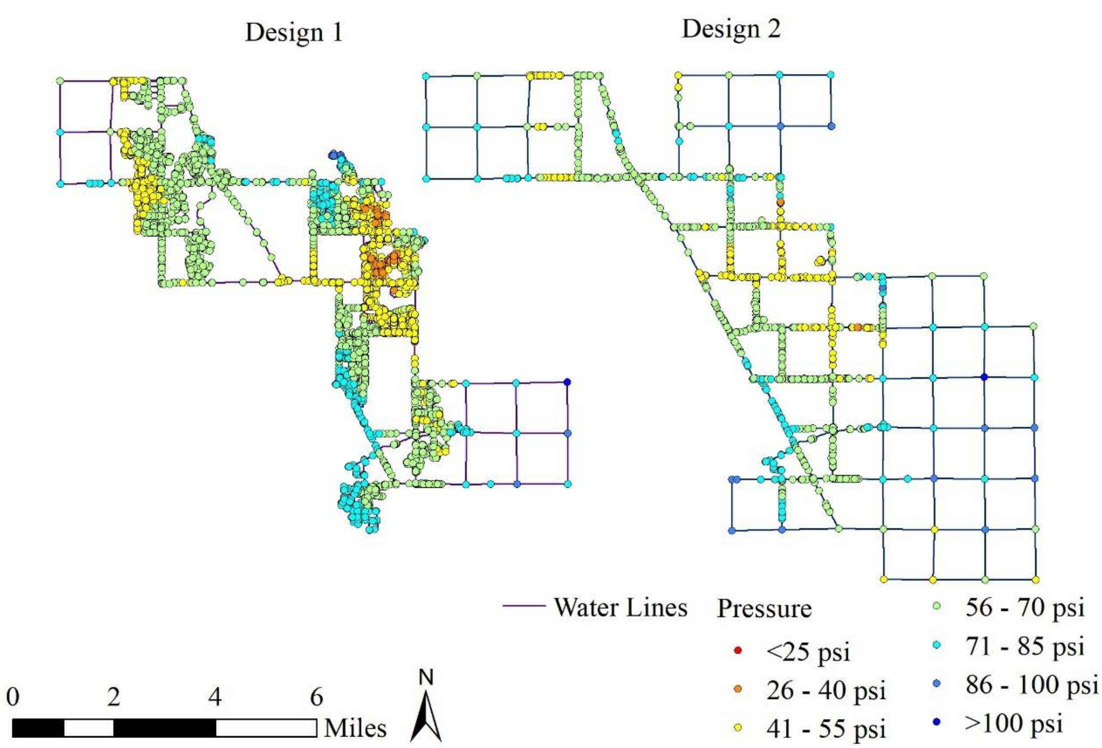

Figure 84. 2062 Nonpotable System Average Day Demands Pressure Results

The velocity results are similar to the pressure results under average day demand, with Design 2 having better statistics than Design 1 – though both designs have good results. The maximum velocity in Design 1 is 1.3 fps and the maximum velocity in Design 2 is 0.97 fps. When the results are compared side by side though, Design 1 has more water lines with lower velocities than Design 2. Regardless, both designs have adequate velocities under average day demands, as shown in the figure below.

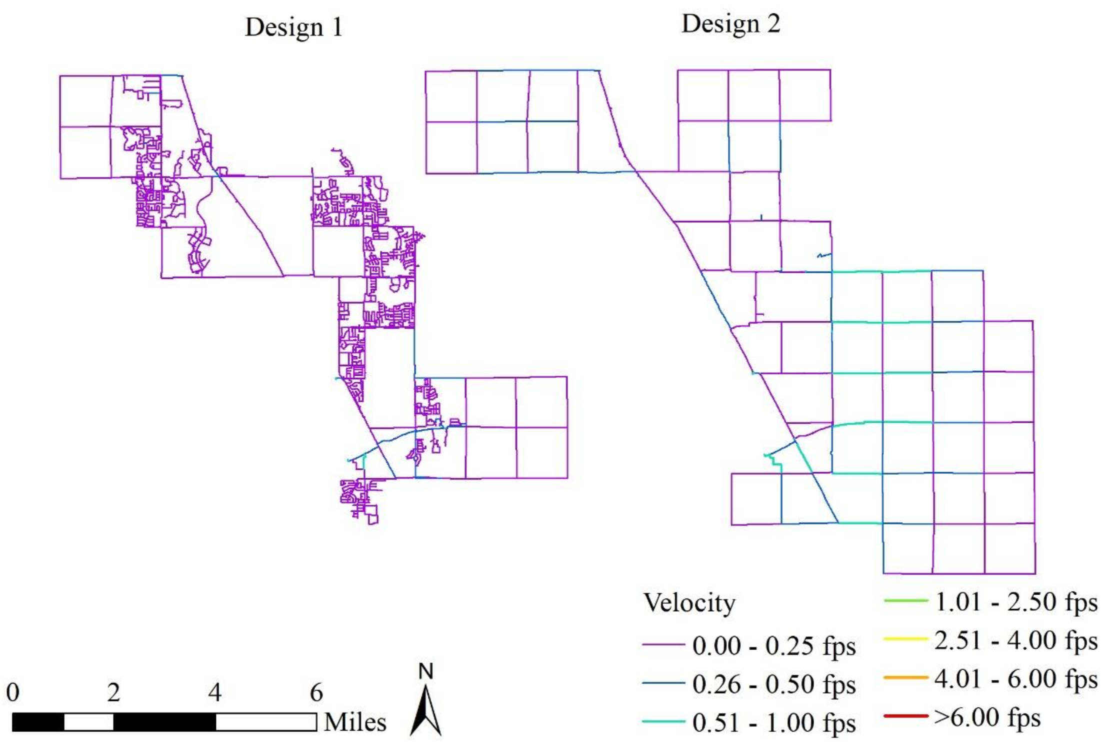

Figure 85. 2062 Nonpotable System Average Day Demands Velocity Results

2062 Nonpotable System with Maximum Day Demands

It has already been stated several times that the 2062 nonpotable distribution system is limited to a service area with a maximum day demand of 3.8 MGD. Thus, the demand under this scenario is 3.8 MGD. Again, Design 2 has better pressure results than Design 1. The minimum pressure for Design 2 is 36.0 psi and it is 32.7 psi for Design 1, while the average pressure for Design 2 is 60.4 psi and it is 57.9 psi for Design 1. The more complex design for Design 1 likely causes these differences because the demands are placed more specifically, whereas the demands for Design 2 are more general. The pressure results for both designs are shown in Figure 86 below.

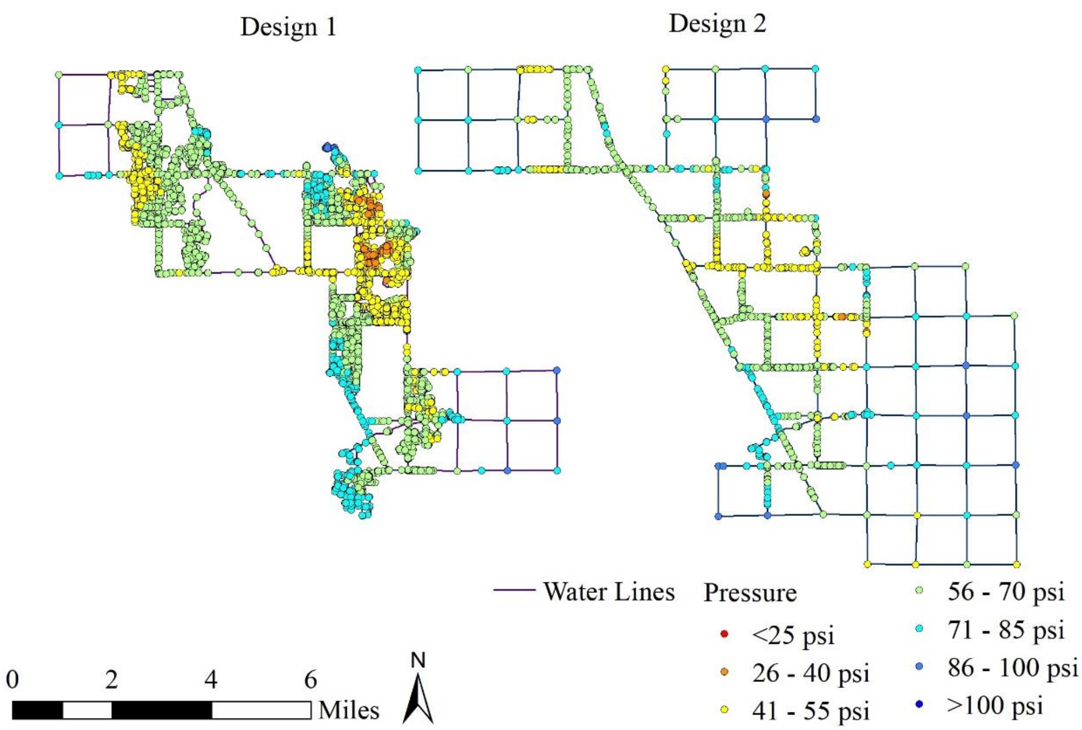

Figure 86. 2062 Nonpotable System Maximum Day Demands Pressure Results

The velocity results for both designs show that, again, both systems perform well. Design 2 has lower maximum velocities, with a maximum of 2.0 fps, while Design 1 has a maximum velocity of 3.1 fps. However, the average velocity for Design 1 is lower than it is for Design 2, which is apparent when the results are compared side by side in Figure 87.

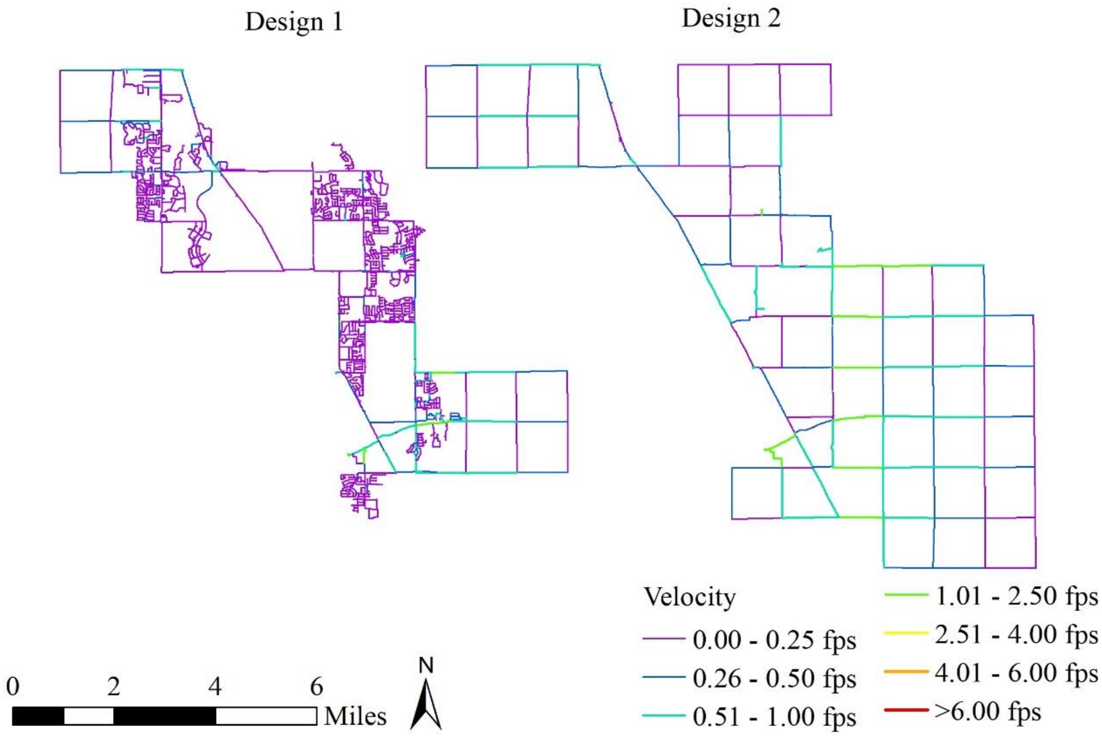

Figure 87. 2062 Nonpotable System Maximum Day Demands Velocity Results

2062 Nonpotable System with Peak Hour Demands

While the service area is limited to a 3.8 MGD maximum day demand, the two service areas were analyzed with the peak hour demands. The peak hour demand was simulated to be 10.5 MGD using the nonpotable peak hour peaking factor determined in Chapter 3; this caused low pressures in both systems. The low pressures are caused primarily because the only storage in the system is located by the WRF, which forces a lot of the water to the north side of the distribution system where the wells are already operating a maximum capacity. Additionally, the results show a “worst-case” scenario because the storage could be located elsewhere in the distribution system to provide and there are no water conservation measures in place, such as odd-even watering. If water conservation measures were employed, then the demands could theoretically be reduced by half, which would alleviate headlosses and raise pressures.

For the peak hour demand, Design 1 had better results, though both systems experienced negative pressures. Design 1 had a minimum pressure of -4.5 psi and an average pressure of 29.9 psi, while Design 2 had a minimum pressure of -7.4 psi and an average pressure of 18.8 psi. The negative pressures are likely caused by the high flows going through the system and lack of supply. Both designs are unacceptable, as shown in Figure 88 below.

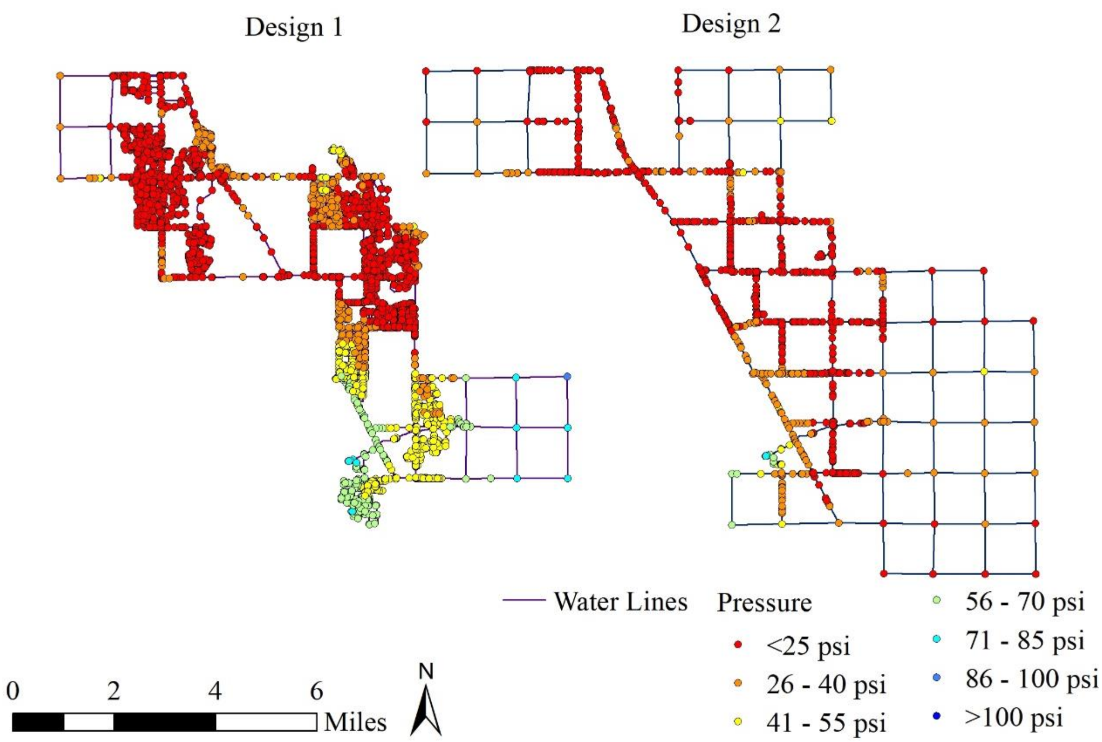

Figure 88. 2062 Nonpotable System Peak Hour Demands Pressure

The velocity results are comparable for both designs too, though Design 1 has slightly better results than Design 2. The maximum velocity in Design 1 is 12.5 fps and it is 14.1 fps in Design 2, and Design 1 has a lower average velocity than Design 2. Figure 89 shows that the higher velocities are more concentrated in local areas in Design 1 while they are spread out across the system in Design 2.

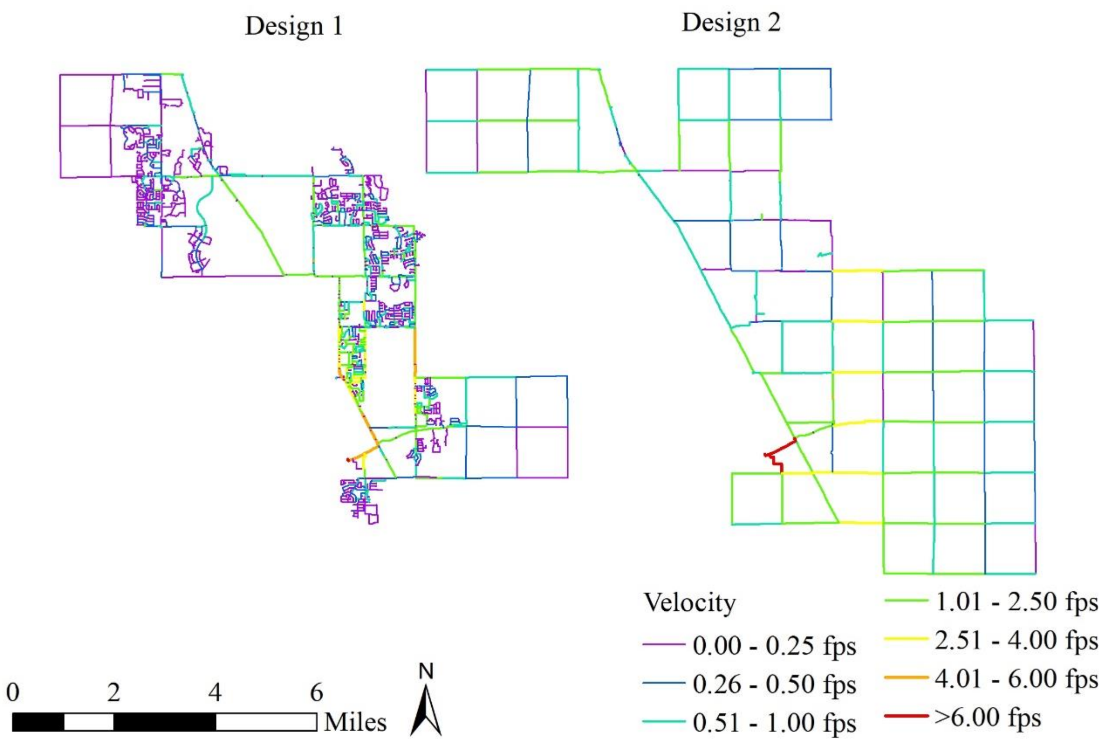

Figure 89. 2062 Nonpotable System Peak Hour Demands Velocity Results

The demands used for the peak demand scenario simulate the system with the all users irrigating at the same time because the demands are based on the historical demands before mandatory odd/even watering was implemented. The mandatory odd/even watering would theoretically reduce the nonpotable peak hour demands by approximately half – enough that either system would not experience negative pressures. While this would have to be verified with analysis, the maximum day results are a good indicator because they show the system with total flows of 3.8 MGD and half of the peak hour demand is 5.3 MGD. Thus with odd/even watering, either system would likely perform similarly to the maximum day demand does.

Summary of 2062 Dual Distribution System

The 2062 dual distribution system performs well, though its biggest problem is that the nonpotable system only serves a small portion of the potential service area, under the assumption of zero nonpotable storage. Between the two proposed nonpotable systems, Design 1 is considered to have better hydraulics overall than Design 2, even though Design 2 had slightly higher pressures for the average day demands and maximum day demands. The increased level of detail for Design 1 shows what the simulated pressures would likely be if the system were installed, whereas Design 2 has a relatively general design. It is hypothesized that a skeletonized version of Design 1 would perform just as well as Design 2, though that step is beyond the scope of this research.

The potable system performs well and could perform even better if some of the deficiencies in the existing system were addressed, such as increasing the water lines connecting the wells to the distribution system and increasing some of the 4-inch water lines that are throughout the distribution system. Another way to improve the potable system would be to fine-tune the design of the expanded parts of the distribution system; the hydraulic analyses provided in this research were rough approximations of a design and an optimal design with more specific demands could potentially be achieved. Regardless, the differences between the single and potable distribution systems show that the distribution system would benefit from reduced demands, whether that happens by installing a secondary distribution system for nonpotable demands or by conservation.

Summary of Hydraulic Analyses

The distribution systems all had results that mostly met the design and evaluation criteria established in Chapter 4. These results show that a dual distribution system can provide adequate pressures under both the potable and nonpotable scenarios. The 2012 distribution systems all had higher pressures than the 2062 distribution systems, and the potable distribution systems all had higher pressures than the single distribution systems. This is to be expected because lower demands lead to lower head losses and hence higher pressures. Finally, the hydraulic results for the nonpotable distribution systems show that the available nonpotable water supply greatly limits the extent of service. If additional water sources could be provided or the contribution from the WRF increased, then the hydraulics would likely increase. Additionally, storage would benefit the systems by assisting with peak demands in terms of supply and pressures.

The key findings from the hydraulic analyses are that the existing distribution system would benefit from reducing the demands and that the nonpotable distributions systems can adequately supply only a limited service area. The costs for the various systems are discussed in the next chapter.

Chapter 6: Cost Comparison

Introduction

Chapter 5 shows that a dual distribution system can provide adequate pressures to serve the water distribution service area in Norman, and can provide better pressures than a single distribution system can for the same peak demand on the potable distribution system. In Chapter 6, the costs for the single distribution system are compared to the costs for the dual distribution system to develop water sources and treat the water for either potable or nonpotable uses. An end-to-end approach evaluates the costs to develop alternate or additional water sources as needed, treat the water, expand or upgrade the treatment facilities as needed, install water lines, and operate and maintain the systems between 2012 and 2062. The largest portion of the cost difference between the single and dual distribution systems comes primarily from the water sources, rather than installing the water lines for the dual distribution system. The single distribution system would require developing additional water sources to supply the increased demands, whereas the dual distribution system is able to use existing water sources to supply most of the demands.

The comparison study herein estimates the costs to supply the 2062 demands with a single distribution system and the costs to supply the 2062 demands with a dual distribution system. The costs estimated in this section are preliminary, which means that they were not calculated with specific quantities, and include a 25 percent contingency and 20 percent for professional services.

2062 Single Distribution System

Norman currently has an approximate potable water supply capacity of 23.0 MGD, without purchasing any water from Oklahoma City. The water treatment plant was designed to meet maximum day demands, which is standard for facilities that treat surface water (Chin, 2013). The estimated maximum day demand is 45.8 MGD in 2062, so Norman will either have to reduce the demand to 23.0 MGD, which is unlikely because the population is growing so quickly, or supply the demand with alternate or additional water sources. Once the new supply is determined, the water will probably need to be treated and the source will need to be maintained, which will often include maintaining pumps and purchasing treatment chemicals.

The water treatment plant and wells together can provide up to 23.0 MGD of potable water at current capacities: the WTP has a peak design capacity of 17.00 MGD and the wells have a 6.0 MGD capacity. With a single distribution system, this production capacity was reached in 2010, as shown in the figure below.

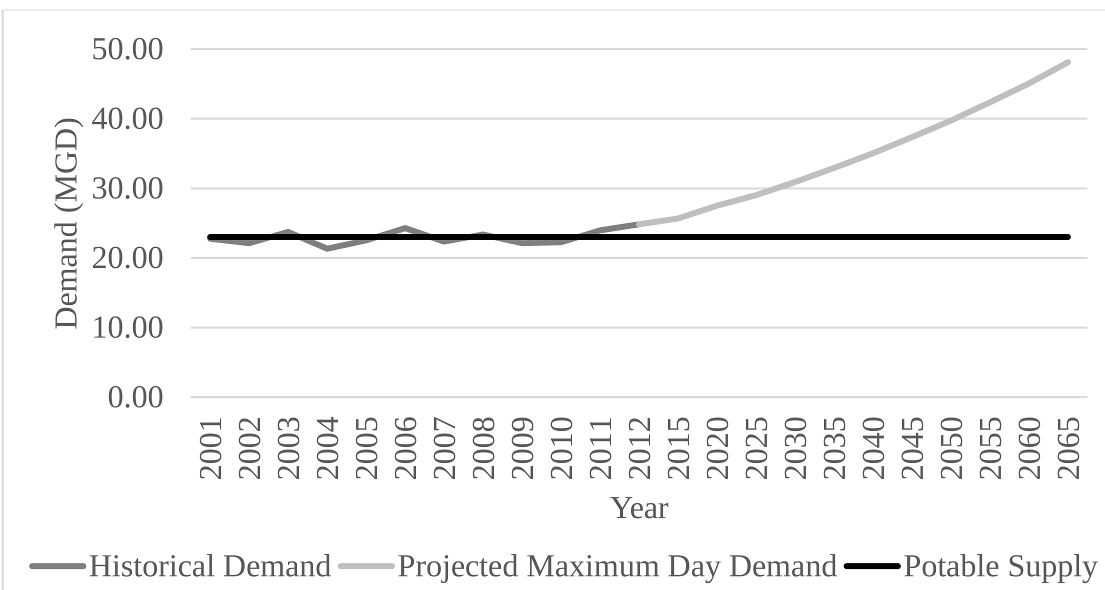

Figure 90. 2012 Production Capacity versus Single System Demands

Figure 90 shows that the deficit between the 2062 maximum day demand of 45.8 MGD and the potable capacity is approximately 22.8 MGD. Somehow, Norman will have to meet this deficit, either by increasing the surface water component and expanding the WTP or increasing the groundwater supply.

Multiple studies have evaluated how Norman will meet future water demands, and the most recent is the 2060 Strategic Water Supply Plan (Carollo, 2014). This report provides cost estimates for a variety of water sources to supply Norman's 2060 water demands, which include the following (Carollo, 2014):

- Drilling additional wells into the Garber-Wellington Aquifer,
- Constructing a new water supply reservoir,
- Augmenting Lake Thunderbird with indirect potable reuse from the Norman WRF,
- Treating the nonpotable wells for arsenic,
- Continuing to purchase water from Oklahoma City, and
- Partnering with Oklahoma City for raw water from southeast Oklahoma.

Each of these options represents a significant investment from Norman for the capital costs to install or develop, and then maintain. Few of them are capable of providing the entire 22.8 MGD needed for maximum day demands. Thus, a combination, or portfolio, of additional and alternate sources is required to meet the 2062 demands. The 2060 Strategic Water Supply Plan recommended a portfolio of options that uses "existing sources, expand[s] conservation, and [has some] reclaimed water projects" (Carollo, 2014). Specifically, this includes augmenting Lake Thunderbird, expanding the WTP, expanding the nonpotable reuse system, treating the nonpotable wells, drilling new wells, and adopting conservation measures (Carollo, 2014). These measures were selected with input from the Norman community and, as such, will be the basis for estimating the costs associated with the 2062 single distribution system.

The recommended portfolio has an average day supply capacity of 29.1 MGD and a peak supply capacity of 55.3 MGD; which is more than adequate to meet the projected 2062 demands. While this research aimed to examine single distribution systems without incorporating nonpotable sources, the 2060 Strategic Water Supply Plan shows how unfeasible this is for a community that wishes to provide its own water while accommodating regulatory conditions. This plan, however, does not utilize water reuse and other nonpotable water sources for urban irrigation; instead, the nonpotable reuse is used at the University of Oklahoma golf course and for daily operations at the WRF (Carollo, 2014).

The estimated capital costs for the recommended portfolio total approximately $270 million (in 2012 dollars), which includes approximately 25 percent for contingency, 20 percent for design fees, and 20 percent for land acquisition where needed (Carollo, 2014). These capital costs include approximately $70 million for new improvements required by regulatory changes and $193 million for new infrastructure to accommodate the capacity increases (Carollo, 2014). The $70 million for new improvements includes approximately $17.6 million to provide arsenic and chromium treatment at the wells. The $193 million includes approximately $52.7 million to expand the WTP to 25.8 MGD and approximately $41.7 million to upgrade the WRF for the indirect potable reuse to Lake Thunderbird. The estimate for the WTP expansion is consistent with cost obtained for industry standards, which range from $4 to $11 per gallon of treated water (John Cutright, October 17, 2017, personal email to author). Another cost included in the $270 million estimate is "a unit cost of $630,000 per new well" (Carollo, 2014), and the 2060 Strategic Water Supply Plan estimated that 10 new wells would be drilled. These costs are summarized in the table below.

Table 40. Summary of 2062 Single System Capital Costs (in 2012 dollars) (Carollo, 2014)

Improvement	**Cost (millions of 2012 dollars)**
New Improvements for Regulatory Changes	
Arsenic and Chromium VI Treatment at Existing Wells	$ 17.6
Other New Improvements, Including Arsenic and Chromium VI Treatment at New Wells	$ 52.4
New Improvements Total	*$ 70.0*
New Infrastructure for Capacity Increases	
Expand WTP	$ 52.7
Upgrade WRF for IPR	$ 41.7
Other New Infrastructure, Including New Wells	$ 98.6
New Infrastructure Total	*$ 193.0*
Unspecified Capital Costs	*$ 7.0*
Total	**$ 270.0**

The O&M costs for the WTP are derived based on the life cycle costs presented in the 2060 Strategic Water Supply Plan (Carollo, 2014). The fixed costs for conventional water treatment are $3.3 million (2012 dollars) per year and additional variable costs, including power, are $0.42/1000 gallons of treated water per year. The variable costs increase as the demand increases, while the fixed costs are assumed to remain constant. When the demands are analyzed with the variable costs, the total WTP O&M costs are approximately $154,054,000 in 2012 dollars.

The O&M costs for the existing wells have an estimated fixed cost of $250,000 (in 2012 dollars) per year and variable costs of $2.01/1,000 gallons, which includes power costs and arsenic and chromium VI treatment (Carollo, 2014). The total O&M costs for the existing 33 active wells are approximately $104,585,000 (in 2012 dollars). Using the same O&M costs, the 10 new wells will have a total O&M cost of approximately $38,609,000 (in 2012 dollars).

Finally, the upgraded WRF treatment facilities for the indirect potable reuse will cost approximately $54,010,000 (in 2012 dollars). This total comes from approximately $2,000,000 (in 2012) above the existing WRF operating costs in annual fixed fees, and approximately $0.24/1,000 gallons (in 2012 dollars) above the existing WRF operating costs in annual variable fees. Combined, the O&M costs for the entire supply portfolio total approximately $440.3 million from 2012 to 2062 (in 2012 dollars) (Carollo, 2014). The O&M costs are over-estimated somewhat because they would increase as the recommended portfolio is instituted, instead of starting in 2012. These costs include the O&M costs for the existing infrastructure also, including the WTP and the potable wells.

Between 2012 and 2062, the total estimated cost for the recommended portfolio is $710.3 million (in 2012 dollars), as broken down in the table below.

Table 41. Summary of 2062 Single Distribution System Costs (in 2012 dollars)

Cost Component	Total Cost
Capital Costs for 2062 Single System	$ 270,000,000
Total O&M Costs for 2062 Single System	$ 440,300,000
Total Cost for 2062 Single System	**$ 710,300,000**

Note that these costs are only for expanding existing water sources and developing new water sources; they do not include the costs to expand the service area for the distribution system or include a new elevated water tower(s). These costs would be incurred regardless of whether or not the 2062 demands are provided by a single or dual distribution system, as designed for this research.

2062 Dual Distribution System

As described above, Norman will have to find a way to supply the 2062 demands, and, with a dual distribution system, the demands are divided between a potable and nonpotable distribution system. The maximum day potable demands are estimated to be approximately 34.7 MGD in 2062 and the total maximum day nonpotable demands are 11.7 MGD for the entire 2062 service area, though the two nonpotable system designs are limited to a maximum day demand of 3.8 MGD, as determined in Chapter 3. The limited service nonpotable area is based on serving a portion of Norman instead of the entire city. Separating the potable and nonpotable demands takes advantage of existing water sources by keeping the potable demands low enough that they can be supplied by existing potable sources until 2029 and by supplying the nonpotable demands with existing nonpotable sources.

Potable Distribution System

By 2062, the potable demands are estimated to be only 19.4 MGD during average day demands and 34.7 MGD during maximum day demands. By only focusing on the potable demand, the existing potable capacity of 23.0 MGD could supply the maximum day potable demand until 2029. The figure below shows the estimated potable demand will not exceed the current potable supply until 2029. This would delay the need to find new water sources and to expand the WTP from 2012 to 2029.

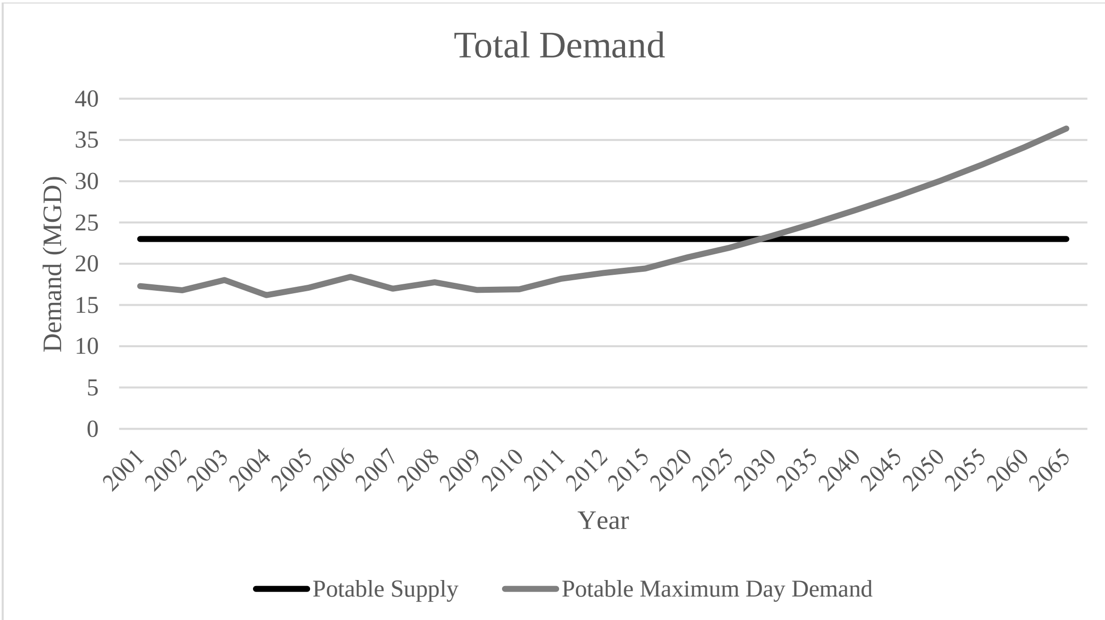

Figure 91. Potable Capacity versus Potable Demands over Time

The potable supply capacity will be reached in 2029, and the difference between the maximum day potable demand and the supply capacity in 2062 will be approximately 13.8 MGD. For comparison, the single system had a deficit of approximately 25.8 MGD between the 2062 maximum day demands and the potable capacity. To supply the deficit, Norman could incorporate parts of the 2060 Strategic Water Supply Plan, including indirect potable reuse or drilling additional wells.

For this research, a major assumption was that the WRF discharge flows would not increase substantially between 2012 and 2062, based on flow rates between 2004 and 2014. This assumption is a conservative assumption because the population is expected to nearly double in that time frame. One report estimated that the discharge flows would increase from approximately 10 MGD to 17 MGD (Garver and Carollo, 2011), which would provide approximately 7 MGD for indirect potable reuse above current discharge rates. Another option to supply the potable demands after 2029 would be to drill new wells. From the 2060 Strategic Water Supply Plan, the estimated cost per new well is $630,000 (in 2012 dollars), which includes a 20 percent design fee, 25 percent contingency, and 20 percent for land acquisition fees (Carollo, 2014). No flow information was provided for new wells in the 2060 Strategic Water Supply Plan, so it is assumed that new wells can provide approximately 170 gpm or approximately 0.2 MGD, which is the average supply capacity of the current wells. In order to supply an extra 13.8 MGD, Norman would need an additional 50 wells, which would cost approximately $31.5 million (in 2012 dollars) in capital costs. Assuming that new wells can provide the additional supply for the demand, the WTP does not need to be expanded. This means that the capital costs associated with the 2062 potable distribution system are only for new wells, at $31.5 million (in 2012 dollars).[8]

The O&M costs for the WTP would remain the same as for the single system with $3.3 million per year in fixed costs and $0.42/gallon for variable costs (Carollo, 2014). For a treatment capacity of 17 MGD and assuming 4 percent inflation each year, this totals $132,800,000 (in 2012 dollars) between 2012 and 2062. The O&M costs for the wells are estimated to be $250,000 (in 2012 dollars) per year in fixed costs and $2.01/1,000 gallons for variable costs, such as power and arsenic and chromium VI treatment (Carollo, 2014). Between 2012 and 2062, the 33 existing wells would cost approximately $104,585,000 (in 2012 dollars). Assuming that the 50 additional wells are drilled starting in 2028, the O&M costs for the potable wells would add approximately

[8] Drilling new groundwater wells is likely the cheapest way to supply the additional 13.8 MGD needed for the 2062 maximum day demand. If the WTP were to be expanded, then that would increase the capital costs associated with the potable distribution system.

$201.5 million to the operating costs for the wells. When the costs are combined for the WTP, the existing wells, and the proposed wells, the total O&M costs for the 2062 potable distribution system are $438.9 million (in 2012 dollars).

The total estimated cost for the 2062 potable distribution system – including the capital costs and total O&M costs – is $470,400,000 (in 2012 dollars). The capital costs to drill the wells are slightly less than 10 percent of the total cost, which shows that the initial investment can sometimes be almost negligible compared to the overall cost of operating a distribution system. The costs are broken down (and rounded) in Table 42.

Table 42. Summary of Costs for 2062 Potable System (in 2012 dollars)

Cost Component	Cost
Capital Costs for 2062 Potable System	$ 31,500,000
O&M Costs	
WTP O&M Costs	$ 132,800,000
Existing Wells O&M Costs	$ 104,600,000
New Wells O&M Costs	$ 201,500,000
Total O&M Costs for 2062 Potable System	$ 438,100,000
Total	**$ 470,400,000**

These costs might be affected by smaller demand peaks and/or smaller water lines resulting from the smaller demand peaks. No costs are included for the expanded distribution system because the water lines and proposed elevated storage tank design for the expanded service area are assumed to be the same for both the potable and single distribution systems.

2062 Nonpotable Distribution System

To meet the nonpotable 2062 demands, a nonpotable distribution system is proposed that can supply a maximum day demand of 3.8 MGD, with 400 MG of long-term storage to accommodate the peak hour demands for the 2062 nonpotable service area. As discussed in Chapters 3 and 4, there are two design options presented; capital costs and O&M costs are estimated for both design options.

Capital Costs

The capital costs for the 2062 nonpotable distribution system include costs to upgrade the WRF, install the secondary distribution system, and install a long-term storage reservoir at the WRF. The capital costs for the WRF upgrade are obtained from the 2060 Strategic Water Supply Plan, at $2.78 per gallon of water treated for reuse. The maximum day demand of 3.8 MGD is used to estimate the capital cost, which equals approximately $10.5 million (in 2012 dollars). This is one-quarter of the estimated $41.7 million to upgrade the WRF for indirect potable reuse, as estimated for the 2062 single system because less water is being treated.

The capital costs to install the nonpotable distribution system are based on water line diameters, which are assumed to include the pipe, fittings, valves, meters, bedding, and labor. The cost provided in the 2060 Strategic Water Supply Plan was approximately $7/diameter inch-linear feet (Carollo, 2014), while Norman uses $8/diameter inch-linear foot (Chris Mattingly, October 16, 2017, conversation with author) to estimate capital costs for new projects. This research uses the more conservative cost of $8/diameter inch-linear foot and adds a 25 percent contingency with 20 percent design fee and 20 percent for land acquisition and easement, as was done for the single and potable distribution systems. This equates to approximately $16/inch-linear foot installed.

Costs are estimated for both Design 1 and Design 2 for the 2062 nonpotable distribution system. The two designs have different service areas that both have a maximum day demand of 3.8 MGD. Design 1 serves residential and commercial neighborhoods that are in the immediate vicinity of some of the nonpotable wells and part of the expanded distribution system for 2062. Design 2 only serves the residential and commercial demand for the expanded part of the distribution system for 2062. Design 2 has larger diameter water lines, but the total length of new water lines is 64 percent of the total length estimated for Design 1. Design 1 provides a finer scale picture than Design 2, so the additional water line length delivers water nearly to the point of use, while Design 2 only delivers water along the section lines. The total estimated cost for Design 1 is $139,070,400 and the total estimated cost for Design 2 is $115,139,200, and it is broken down by water line diameter in the table below.

Table 43. Cost Estimates to Install the 2062 Nonpotable Distribution Systems

Diameter	Unit Cost	Design 1		Design 2	
		Length (ft)	Estimated Cost	Length (ft)	Estimated Cost
4"	$ 64	295,100	$ 18,886,400	-	$ -
6"	$ 96	279,500	$ 31,304,000	-	$ -
8"	$128	18,900	$ 2,419,200	-	$ -
10"	$160	176,800	$ 28,288,000	529,800	$ 84,768,000
12"	$192	112,700	$ 21,638,400	20,100	$ 3,859,200
14"	$224	69,500	$ 15,568,000	71,300	$ 15,971,200
18"	$288	72,800	$ 20,966,400	36,600	$ 10,540,800
Total		**1,025,300**	**$139,070,400**	**657,600**	**$115,139,200**

The Strategic Water Supply Plan also included a cost of $5,500/acre-foot for storage reservoirs (Carollo, 2014). This is a relatively high cost to reflect “the likely urban location and lower economy of scale for a smaller, terminal storage reservoir” (Carollo, 2014). This cost is increased to $6,000/acre-foot to include the cost for a pump station to supply water during peak hour demands. In Chapter 3, it was calculated that the storage demand would be 400 MG to accommodate the difference between the 2062 average day supply and the 2062 peak demands, which equates to 1,228 acre-feet. At $6,000/acre-foot, the total cost for a storage reservoir and small pump station would be approximately $7,800,000.

A final consideration for Design 1 is the cost to retrofit the homes on the existing distribution system with service lines and reconnecting hose outlets to the nonpotable distribution system. It is suggested that Norman incur this cost instead of passing it on to the users because the new connection would be instigated by the City of Norman and not the users. This cost would be similar to the cost to install a service line and meter to a home, which the City of Norman charges $60 per meter (City of Norman 2017). There are approximately 1,800 residential parcels along the existing distribution system in Design 1, which equates to $108,000 in additional costs for Design 1. This cost is not included in Design 2 because Design 2 exclusively serves the expanded parts of the distribution system, where it would be relatively easy to include the nonpotable system in the design-phase of the new neighborhoods instead of retrofitting existing neighborhoods.

The total estimated capital costs for Design 1 is approximately $157,478,400 and the total estimated capital costs for Design 2 are approximately $133,439,200 (both values in 2012 dollars). The cost difference between these designs is entirely based on the scale of the service area. More specifically, Design 1 serves existing customers instead of potential customers; as such, this requires retrofitting neighborhoods and homes for nonpotable irrigation. Design 2 does not have any retrofits and can mandate that new neighborhoods are designed and constructed for nonpotable irrigation, which helps to pass the design costs to the developers. The capital costs for each of the components are shown in Table 44 below.

Table 44. Capital Cost Estimates for 2062 Nonpotable Distribution System

Cost Component	Design 1 Costs	Design 2 Costs
WRF Upgrade	$ 10,500,000	$ 10,500,000
Install 2062 Nonpotable Distribution System	$ 139,070,400	$ 115,139,200
Long-Term Storage Reservoir	$ 7,800,000	$ 7,800,000
Retrofits	$ 108,000	$ -
Total 2062 Nonpotable Capital Costs	**$ 157,478,400**	**$ 133,439,200**

O&M Costs

The O&M costs for the nonpotable distribution systems include maintaining the WRF, the existing wells, the distribution system, and the storage reservoir. The WRF fixed O&M costs are estimated at $2,000,000 per year above the existing fixed O&M and the variable costs are at $0.24/1,000 gallons above the existing O&M costs for variable costs (Carollo, 2014). With an estimated 4 percent inflation, this equates to $45,500,000 between 2012 and 2062.

The O&M costs for the nonpotable wells are estimated to be the same as the O&M costs used for the single and potable distribution systems. The wells have an estimated $250,000 fixed O&M and $0.33/thousand gallons in variable O&M costs. Between 2025 and 2062, this equates to $5,055,000 in 2012 dollars for the wells with a 2.8 MGD flow.

The distribution system O&M costs assume a pipe rehabilitation and replacement amount of 1 percent of the total length of pipes per year at a rate of 75 percent of the cost to install new water line. The cost is less than installing a new water line because there are no design fees or land acquisition/easement fees. For the two designs, the yearly O&M costs were estimated by calculating:

- The average diameter for each design
- 1 percent of the total water line length for each design
- The rehabilitation and replacement cost at 75 percent of new water line

Design 1 has a smaller average diameter but more water line length than Design 2, which makes it have a higher estimated yearly O&M cost. The O&M costs for the nonpotable distribution system begin in 2025, when it is assumed that the first nonpotable users would start using the nonpotable water for irrigation. Assuming 4 percent inflation, the total estimated O&M costs for the nonpotable distribution system with the well O&M costs are provided in the table below.

Table 45. Yearly O&M Costs for 2062 Nonpotable Distribution Systems

Cost Component	Design 1	Design 2
Average Diameter (inches)	8.2	10.9
1% of Total Length (feet)	10,253	6,576
Yearly Rehab/Replace Cost (2012 dollars)	$ 504,500	$ 430,100
Total O&M Costs 2025-2062 (2012 dollars)	**$ 15,215,900**	**$ 13,717,800**

For the hydraulic analyses, the storage reservoir was assumed to be located at the WRF. If it is located at the WRF, it is assumed that the O&M costs are incorporated into the WRF O&M costs. This reasoning is justified because the estimated WRF O&M costs are for treating water indirect potable reuse, which would require more advanced treatment than treating water for direct nonpotable reuse. Thus, the storage reservoir is estimated to have no additional O&M costs for the 2062 nonpotable distribution systems

The total O&M costs for 2012 to 2062 for Design 1 are estimated to be approximately $65,770,900 (in 2012 dollars), and the total O&M costs for 2012 to 2062 for Design 2 is estimated to be approximately $64,272,800 (in 2012 dollars), as shown in

Table 46. The total O&M costs for the two designs for the 2062 nonpotable distribution system are very close, with a difference of approximately $1.5 million (in 2012 dollars), which is only 2.3 percent of the two O&M costs.

Table 46. Total Estimated O&M Costs for 2062 Nonpotable Distribution Systems

Cost Component	Design 1 Costs	Design 2 Costs
WRF O&M Costs	$ 45,500,000	$ 45,500,000
Nonpotable Wells O&M	$ 5,055,000	$ 5,055,000
Distribution System O&M	$ 15,215,900	$ 13,717,800
Total	**$ 65,770,900**	**$ 64,272,800**

Summary of Dual Distribution System Costs

When the costs for the 2062 potable and nonpotable distribution systems are combined, the O&M costs for the potable system constitute the greatest amount of the total costs for the 2062 dual distribution system, followed by the capital costs for the nonpotable systems. Overall, the total cost for a dual system with Design 1 is approximately $575.8 million (in 2012 dollars) and the total cost for a dual system with Design 2 is approximately $540.3 million (in 2012 dollars), as presented in Table 47.

Table 47. Summary of 2062 Dual Distribution System Costs (in 2012 dollars)

Cost Component	Design 1 Costs	Design 2 Costs
Potable System Capital Costs	$ 31,500,000	$ 31,500,000
Nonpotable System Capital Costs	$ 157,478,000	$ 133,439,000
Potable System O&M Costs	$ 470,400,000	$ 470,400,000
Nonpotable System O&M Costs	$ 65,770,900	$ 64,272,800
Total	**$ 693,648,900**	**$ 668,111,800**

The cost difference between the two designs is relatively small at approximately $25.5 million (in 2012 dollars). In some design cases, the lower cost option controls the recommended design; in this case, the two cost estimates are relatively similar and either design would provide nonpotable water for urban reuse for approximately the same total cost.

Comparison

The total (capital plus O&M) 2062 single distribution system costs are compared to the two 2062 dual distribution system costs. The results, illustrated in Figure 92, show that the single distribution system costs more than Design 1 of the dual distribution systems by nearly $16 million over the life of the study period. The difference in costs between the single and dual systems is attributed to the need to provide additional potable water sources for the single distribution system, which necessitates an expansion of the WTP.

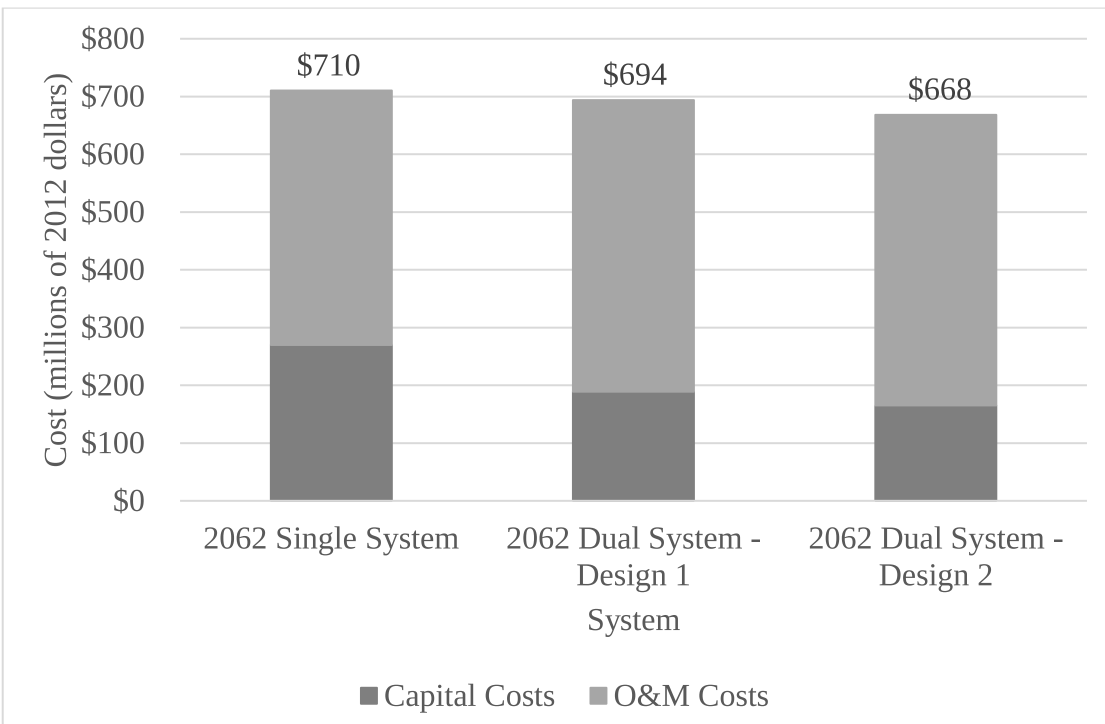

Figure 92. Total Cost Comparison of the 3 Systems

The total costs for the distribution systems show that Norman could potentially save money by installing a dual distribution system because it would delay the need to develop new water sources.

Summary of Cost Comparison

The cost comparison of the single and dual distribution systems shows that constructing a secondary distribution system would cost less than the estimated costs to supply the 2062 demands with only potable water. The dual distribution systems for the nonpotable service has much higher capital costs that the WTP, but the WTP has much higher O&M costs than the dual distribution systems. This analysis is consistent with preliminary studies briefly discussed by Grigg et al. (2013), which surveyed more than 35 utilities across the United States that operate dual distribution systems and show that a nonpotable system could be feasible for extending potable water sources, optimizing nonpotable water sources, and providing Norman with an alternate water source.

Chapter 7: Conclusions and Future Research Opportunities

Between 2012 and 2062, the City of Norman is expected to increase its population by approximately 100,000 people, which will cause Norman to nearly double in size. These people will need water provided by the City. At the same time, the available water supplies are expected to remain constant or even decrease, particularly if extreme droughts become more frequent. The City of Norman has already invested in solving this problem with the 2060 Strategic Water Supply Plan (Carollo, 2014). However, one potential solution that was not explored in much depth was utilizing existing nonpotable water sources for urban irrigation. This might be because adding residential customers can "increase the risk and complexity of operating dual distribution systems" (Grigg et al., 2013), which includes monitoring cross-connections and a large public education/outreach campaign. Also, Norman has several industries that could potentially use the nonpotable water, and industrial nonpotable water use is more common than urban irrigation. However, as this research has shown, urban irrigation can comprise up to 44 percent of the total demand during the summer, so by putting some of this nonpotable demand onto a secondary nonpotable distribution system, Norman could alleviate some of the projected water supply problems. This question was addressed quantitatively during this research, first by estimating a variety of water demands for 2012 and 2062, then applying those demands to hydraulic models to see how the models performed. Finally, the costs were determined for various scenarios.

Since 2006, the City of Norman has had wells off-line because the arsenic levels in the wells exceeded the maximum contaminant level for arsenic. The wells could still produce water, but it would not be considered safe to drink without additional treatment.

As Chapter 4 noted, the nonpotable water sources could not supply all the estimated urban irrigation demands. As a result, the proposed nonpotable distribution system was scaled down such the nonpotable water sources could supply the urban irrigation demands on the distribution system.

In practical terms, a secondary distribution system is highly unlikely to be installed in established neighborhoods because the potential cost to retrofit the neighborhood and all the homes with new hose connections and sprinkler systems would be exorbitant and challenging. However, new areas would be easier to incorporate the nonpotable system into the design and install the systems when the rest of the neighborhood is under construction. Additionally, the existing nonpotable sources are limited to a maximum capacity of 3.8 MGD, which is only a portion of the total 2062 nonpotable demand.

A few final conclusions of this research are listed below.

- The existing single distribution system performs well and delivers all the demand at adequate pressures, with few deficiencies in the system, during all the demand scenarios.
- The existing distribution system performs better with only the potable demands on it than with all the demands on it, which indicates that the existing distribution system would benefit from reducing the total demands and that the service life of the distribution system could thus be extended.
- The nonpotable water sources have a total yearly capacity that is greater than the estimated 2012 nonpotable demand, but that the deficit during the

summer is too great to provide storage unless the City of Norman were willing to create a reservoir capable of storing approximately 400 MG. The cost to construct a reservoir is cheaper than initially expected and only a fraction of the cost to install a new distribution system.

- The extent of the nonpotable distribution system is limited more by the available nonpotable water supply and available storage than pressure problems in the system.
- The future nonpotable distribution system would best serve new developments because it would allow the secondary system could be integrated with the potable system during design instead of retrofitting the existing system. .
- Water reuse is not the only nonpotable water source available for Norman and, probably, many other communities. Groundwater contaminants that are dangerous for human consumption do not necessarily negatively impact grasses and plants, so groundwater sources with high levels of arsenic or chromium VI could be incorporated as nonpotable water sources. This would allow communities to take full advantage of their available water sources, while preserving their freshwater sources.
- The costs to install and maintain a new distribution system are comparable to the costs to expand and maintain a water treatment plant. Conventional wisdom in the water industry is that the cost to install a secondary distribution system is prohibitive, but this is likely because the costs to install a dual distribution system are compared to the costs to install a

single distribution system. A better comparison would be the costs to provide enough treated water to supply all the demands versus the costs to install a secondary distribution system and avoid treating the water used for urban irrigation to potable standards.

This research only scratches the surface of the possibilities for a nonpotable distribution system in Norman, Oklahoma and for other similarly-sized communities. While there are potential research opportunities for comparing single distribution systems to dual distribution systems, the biggest opportunities would be to develop more complex hydraulic models for the nonpotable distribution systems. The more complex hydraulic models could simulate a variety of storage options, different service areas, different water line diameters, putting fire flow on the nonpotable distribution system with the requisite storage, and extended period simulations to schedule irrigation demands (e.g., residential irrigation during early morning and evening hours, and commercial irrigation during nighttime hours). Other research opportunities include more detailed cost estimates with a variety of different costs, including incurring the full cost to retrofit the entire services area, drilling additional nonpotable wells, and constructing new treatment plants.

There are many possibilities for water reuse and nonpotable water sources. Throughout much of the United States, almost all the municipal water demand is supplied with potable water, even though the demand is comprised of both potable and nonpotable demands. States across western and southern United States are incorporating nonpotable use as part of their water portfolios, yet many states in the United States still ignore urban irrigation as a potential use. This research shows that urban irrigation is more feasible in terms of cost than many might expect. One key component is that all possible nonpotable sources should be included in the nonpotable portfolio, not just the effluent from wastewater treatment plants. Nonpotable sources, such as storm water detention reservoirs and nonpotable wells, can provide decentralized sources that allow a municipality to serve a large service area and maintain adequate pressures instead of relying on a sole source for the nonpotable water. As long as the health and safety of a community are preserved, nonpotable reuse for urban irrigation is a very real possibility to help protect and optimize use of our valuable freshwater sources.

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Appendix A: Technical Definitions

Some of the words and terms used throughout this research are fairly technical and so it might be helpful to many readers to have some of the technical words and terms defined. If a term has an acronym or abbreviation, this is also included with the definition. The words and terms are provided in alphabetical order.

Average Day Demands (ADD) The ADD is calculated by summing the entire demands for a calendar year and dividing the demand by 365 days (or 366 for leap-years).

ArcMap ArcMap is the desktop GIS software produced by Esri ArcGIS. This is the GIS software that was used to perform all shapefile modifications and edits, analyze the spatial distribution of population and water demands, and generate all maps included in this research paper.

Baseline Demand The baseline demand is the minimum demand established by the three months in a year with the lowest demand, which is when the urban irrigation demand would be lowest. The demands from these three months are averaged to estimate the potable demand needed for a distribution system. It is assumed that demands greater than the baseline demand are for urban irrigation.

Bentley® Bentley Systems® (Bentley®) is an American software company that creates and distributes the industry-standard hydraulic modeling software WaterGEMS.

Calibration The process of adjusting and checking the accuracy of the hydraulic model against field measurements, such as flow tests.

Direct Nonpotable Reuse (DNR) "The use of community wastewaters treated to a sufficient degree that they are acceptable for a wide range of nonpotable uses, such as irrigation, and direct discharge into a nonpotable distribution system that provides service to customers who obtain their potable water from a separate system" (AWWA, 2010).

Direct Potable Reuse (DPR) "The use of community wastewaters treated to a sufficient degree that they are acceptable for drinking and direct discharge into a potable-water distribution system" (AWWA, 2010).

Distribution System "A system of conduits (laterals, distributaries, pipes, and their appurtenances) by which a primary water supply is distributed to consumers. This term applies particularly to the network of pipelines in the streets of a domestic water system" (AWWA, 2010).

Distribution System – Single A single distribution system that delivers water to all customers. The water is either potable or nonpotable. There are no other distribution systems that supply water, whether it is potable or nonpotable.

Distribution System – Dual A dual distribution system has two separate distribution systems that deliver water to end-users. One of the distribution systems carries potable water and the second distribution system typically carries nonpotable water. The potable distribution system typically supplies all potable water needs, including household uses such as drinking, showering, and washing dishes. The nonpotable distribution system can supply water only for irrigation, only for toilet flushing, only for fire flow, only for other nonpotable uses, or a combination of these end-uses. No cross connections are typically allowed between the systems.

Esri — Environmental Systems Research Institute; a company that created and maintains ArcGIS, a standard GIS software program.

Fire Flow, Available — "The rate of flow, usually expressed in gallons or liters per minute, that can be delivered from a water distribution system at a specified residual pressure for firefighting. When delivery is to fire department pumpers, the specified residual pressure is usually 20 psi" (AWWA, 2010).

Fire Flow Tests — Fire flow tests are commonly performed to evaluate the pressure within a distribution system. The residual pressure at one hydrant (Test Hydrant) is measured as one to three downstream hydrants (Flow Hydrants) in a row are gradually opened until they are flowing at maximum flow. Then, the hydrants are slowly closed to prevent water hammer. The pressure drop is measured between when the first of the downstream hydrants is opened and when the last hydrant is flowing at maximum flow. The pressure drop is used to calibrate the hydraulic model.

GIS — GIS is the acronym for Geographic Information Systems. "A system that stores and links nongraphic attributes and geographically referenced data with graphic map features to allow a wide range of information processing and display operations, as well as map production, analysis, and modeling" (AWWA, 2010).

Hydraulic Grade Line (HGL) "A line (hydraulic profile) indicating the piezometric level of water at all points along a conduit, open channel, or stream. In a closed conduit under pressure, artesian aquifer, or groundwater basin, the line would join the elevations to which water would rise in pipes freely vented and under atmospheric pressure. In pipes under pressure, each point on the hydraulic profile is an elevation expressed as the sum of the height associated with the pipe elevation, the pipe pressure, and the velocity of the water in the pipe" (AWWA, 2010).

Hydraulic Model "A computer program that predicts pipe flows and pressures throughout a water distribution network. The model can be used to calculate and simulate various hydraulic conditions" (AWWA, 2010).

Indirect Nonpotable Reuse (INR) "The use of appropriately treated community wastewater discharged to received waters, either surface or underground, which is drawn, generally after additional treatment, for distribution for nonpotable purposes to customers who obtain their potable water from a separate system" (AWWA, 2010).

Indirect Potable Reuse (IPR) "The use of water from streams that have upstream discharges of wastewater" (AWWA, 2010).

Maximum Day Demands (MDD) The MDD is determined as "the volume of water consumption used on the highest-consumption day in a year" (AWWA 2010). In this research, the demand is typically reported in million gallons per day (MGD).

Maximum Month Demands (MMD) The MMD is the volume of water consumption used during the highest-consumption month in a year. In this research, this demand is reported as the average daily demand during this month, typically in million gallons per day (MGD).

Nonpotable Water Nonpotable water is water that does not meet local, state, and federal guidelines for drinking water. "Water that may contain objectionable pollution, contamination, minerals, or infective agents and is considered unsafe, unpalatable, or both for drinking" (AWWA, 2010). Typical sources for nonpotable water include wastewater treatment plant effluent, untreated surface water, and untreated groundwater.

Nonpotable Reuse "The use of reclaimed water for nonpotable purposes (i.e., not intended for direct human consumption). Such water is typically used for lawn watering, ornamental crop irrigation, industrial uses, and other purposes" (AWWA, 2010).

Parcel-Level Parcel-level data refers to data provided on an individual property parcel. This can include the number and size of buildings on a property, how much pavement is on the property, and even the number of toilets on a property in very detailed cases.

Peak Hour Demand (PHD) "The greatest volume per hour flowing through a treatment plant for any hour in the year" (AWWA, 2010).

Peaking Factors Peaking factors are ratios of a specified demand to the average day demand, such as maximum day to average day and peak hour to average day.

Per Capita Demand (pcd) "An estimate of the water usage in a community, determined by dividing the total water used by the number of persons using it. It is the average amount of water used by a person within a given period of time. It is most commonly expressed in units of gallons (or liters) per capita day" (AWWA, 2010).

Potable Potable water is "safely drinkable" (AWWA, 2010) and meets local, state, and federal guidelines for drinking water.

Reclaimed Water "Wastewater that has been treated and recovered for useful purposes, such as landscape irrigation" (AWWA, 2010)

Recycled Water "Water is the reused in the same process, with or without additional treatment, or reused by the same user" (AWWA, 2010).

Shapefile "A shapefile is an Esri vector data storage format for storing the location, shape, and attributes of geographic features. It is stored as a set of related files and contains one feature class" (ArcGIS 2017).

Skeletonization "The practice of deleting certain minor pipes and facilities from a system model. Ideally, this creates a manageable model that decreases execution time without a loss of overall accuracy in model performance" (AWWA, 2010).

Steady-State Analysis "A pipe network analysis that assumes that consumptions, supplies, storage-tank levels, and certain other variables are unchanging for one specific point in time" (AWWA, 2010). In other words, a steady-state analysis provides an instantaneous snapshot of what is happening in a water distribution system at a single point in time, as opposed to an analysis that looks at how a distribution system responds over a period of time.

Water Supply Specifically for this research project:

1) "The sources of water for public or private uses" (AWWA, 2010) (e.g., Lake Thunderbird and groundwater wells)
2) The volume of water supplied to meet water demands (e.g., the volume of water treated at the water treatment plant)

WaterGEMS WaterGEMS is an industry-standard software program created by Bentley systems that performs hydraulic modeling. This software was used to perform all hydraulic modeling for this research project.

Water Reclamation Facility (WRF) A wastewater treatment facility that reclaims the wastewater for direct or indirect potable or nonpotable uses.

Appendix B: Norman Zone Codes

The City of Norman adopted a Zoning Ordinance in 1954, which has been updated several times since then (City of Norman 2013). The Zoning Ordinance was first accessed for this research in 2013, though it has been updated by the City Council as recently as June 26, 2017 (City of Norman 2017). Because 2013 data was used for this research and the Zoning Ordinance from 2013 was downloaded, the zone codes from the 2013 Zoning Ordinance are included below.

PUD: Planned Unit Development District

A-1: General Agricultural District

A-2: Rural Agricultural District

RE: Residential Estate Dwelling District

R-1: Single Family Dwelling District

R-1-A: Single Family, Attached Dwelling District

R-2: Two Family Dwelling District

R-3: Multi-Family Dwelling District

RO: Residence-Office District

RM-2 Low Density Apartment District

RM-4: Mobile Home Park District

RM-6: Medium Density Apartment District

O-1: Office-Institutional District

C-1: Local Commercial District

G-2: General Commercial District

TC: Tourist Commercial District

CR: Rural Commercial District

C-3 Intensive Commercial District

M-1: Restricted Industrial District

I-1: Light Industrial District

FH: Flood Hazard District

PL: Park Land District

HD: Historical District

Airport Height Overlay District

The Zoning Shapefile also had two additional zone codes:

Uncategorized

ROW: Right of Way