USING EPANET TO OPTIMIZE OPERATION OF THE RURAL WATER DISTRIBUTION SYSTEM AT BRAGGS, OKLAHOMA

By

SARA NAMANDA SENYONDO

Bachelor of Science in Mechanical Engineering

Makerere University

Kampala, Uganda

2000

Submitted to the Faculty of the
Graduate College of the
Oklahoma State University
in partial fulfillment of
the requirements for
the Degree of
MASTER OF SCIENCE
December, 2009

USING EPANET TO OPTIMIZE OPERATION OF THE

RURAL WATER DISTRIBUTION SYSTEM

AT BRAGGS, OKLAHOMA

Thesis Approved:

Dr. Dee Ann Sanders

Thesis Adviser

Dr. Arthur Stoecker

Dr. Gregory Wilber

Dr. A. Gordon Emslie

Dean of the Graduate College

ACKNOWLEDGMENTS

Everything I have accomplished is simply a sign of God's grace to me. I have taken a journey that seemed very discouraging as I tried to settle in at the beginning of the Spring Semester of 2008. Now, looking back at what I have been able to achieve since then, I am glad I kept the faith and the resolve to see the end.

I would like to thank the Oklahoma Water Resources Research Institute (OWRRI) for providing the funding that made this study possible and my advisor, Dr. DeeAnn Sanders for her unwavering support and tremendous encouragement throughout the course of my graduate studies at Oklahoma State University and especially over the last couple of months as I worked to complete my thesis. I would also like to thank the other members of my thesis committee, Dr. Arthur Stoecker and Dr. Gregory Wilber. Dr. Stoecker has been of immense help since we started working on this project and helped me immensely as I tried to get my model running last summer. Dr. Wilber has provided me with a wealth of knowledge throughout the course of my graduate studies. My thesis committee members have been available throughout to answer the questions that came up as I worked to complete the study and I am very thankful to them. I would like to acknowledge Neha, Mike, Deepayan, Dr. Whitacre and Atta, the other members of the team with whom I worked on the OWRRI project.

I would like to thank my family whom I love dearly. We are a large family and I am unable to mention each and every one of you. However, I thank you all for believing in me and cheering me on when I was down.

I believe my parents deserve special mention for they have been the cheerleaders. Cate and Daniel, thank you for making this journey a lot easier than it would have been otherwise. Tatu and David, you made sacrifices and God will reward you. To my special son Bradley, the joy of my life; you have blessed me and changed me in ways that I could never have imagined.

Finally, to my friends Emmanuel, Gloria, Brenda, Martha, Beatrice and Emily for the tremendous love and support you have shown me and my son. To Francis, thank you for opening your home and sharing the love of God with all of us. To all my Nigerian friends that have prayed with and for us, I pray that the good Lord will reward all of you according to his glorious riches.

TABLE OF CONTENTS

Chapter Page

I INTRODUCTION 1

1.1 Purpose.. 1

1.2 Project Background.. 1

II REVIEW OF LITERATURE.. 6

2.1 Municipal water demands.. 6

2.2 Water distribution systems.. 8

2.3 Rural water systems 11

2.4 The effects of design factors and pipe materials.. 12

2.5 Compliance with drinking water regulations.. 16

2.6 Rehabilitation and trends for replacement of infrastructure 18

2.7 Funding for regulatory compliance and system upgrades 20

2.8 Personnel requirements.. 24

2.9 EPA drinking water research programs 25

2.10 Water distribution system modeling.. 26

2.10.1 EPANET 28

2.10.1.1 Physical components of the network model.. 30

2.10.1.2 Non physical components of the network model 32

2.10.1.3 The hydraulic simulation model.. 33

2.10.1.4 The water quality simulation model.. 34

2.11 Sources of water distribution system information for Oklahoma 35

III METHODOLOGY 36

3.1 Obtaining system information for Braggs... 36

3.2 Creating a model of the pipeline for use with EPANET 37

3.3 Hydraulic modeling using EPANET 39

IV FINDINGS... 43

4.1 Water distribution system information for Braggs 43

4.2 Hydraulic simulation of existing conditions.. 44

4.3 Location and performance of existing fire hydrants.. 52

4.4 Energy cost for running pumps……………………………………………………..54

4.5 Water quality simulation under existing conditions 54

V CONCLUSIONS.. 66

REFERENCES 69

APPENDICES 72

Appendix A – EPANET *.inp file for Braggs RWS…………………………………….73

LIST OF TABLES

Table Page

Table 4.1: Water Storage Facilities at Braggs, Oklahoma …………………………44

Table 4.2: Junctions with pressure below 20psi after 2 hours ……………………..53

Table 4.3: The effect of pipe material on the wall decay constant k_w …………….. 56

Table 4.4: Effect of looping on water distribution system …………………………60

Table 4.5: Simulated loops to control water age ………………………………….. 63

Table 4.6: The effect of looping on water age …………………………………….. 64

LIST OF FIGURES

Figure Page

Figure 1.1: Location of the city of Braggs ……………………………………….. 2

Figure 1.2: Central water tower at Braggs, Oklahoma ……………………………… 3

Figure 1.3: Artesian well at Braggs, Oklahoma……...……………………………… 4

Figure 2.1: Physical components in a distribution system …………………………. 30

Figure 4.1: EPANETZ model of Braggs Rural Water System ……………………. 45

Figure 4.2: Layout of pipelines within the city of Braggs …………………………. 46

Figure 4.3: Operating curve for the pumps at Braggs ……………………………… 47

Figure 4.4: Variation of pressure with time at Node BrJ245……………………….. 49

Figure 4.5: Nodes that experience pressure above 90psi …………………………... 51

Figure 4.6: Energy report for 48 hour simulation ………………………………….. 54

Figure 4.7: Nodes with non zero demand & chlorine residual less than 0.2mg/l…… 57

Figure 4.8: Water age at nodes with demand & chlorine residual less than 0.2mg/l…58

Figure 4.9: variation of water age with time at BrJ55 before looping to BrJ84 …... 61

Figure 4.10: Variation of water age with time at BrJ84 after looping to BrJ55…….. 62

CHAPTER I

INTRODUCTION

1.1 Purpose

The purpose of this study was to assess the performance of the drinking water distribution system at Braggs, Oklahoma with regard to water distribution system requirements using hydraulic simulation software and to address any improvements required to existing infrastructure and/or the mode of operation, in order to improve quantity and quality of water distributed to the customers. The study of the drinking water distribution system at Braggs also aimed to establish how common problems experienced by rural water systems can be detected and addressed using hydraulic simulation software. The main focus of the study was water quality, pressure at different points within the distribution system, fire flow requirements, pipe materials and age of the distribution system.

1.2 Project Background

The City of Braggs was selected because it fits the description of a Rural Water District (RWD). Braggs is located in eastern Oklahoma, 56 miles south east of Tulsa. Figure 1.1 shows the location of Braggs, Oklahoma. The population of the city is 308. The largest section of the existing water distribution system was installed in 1982 and has been serving the local population and 650 people in surrounding areas for the last 27 years.

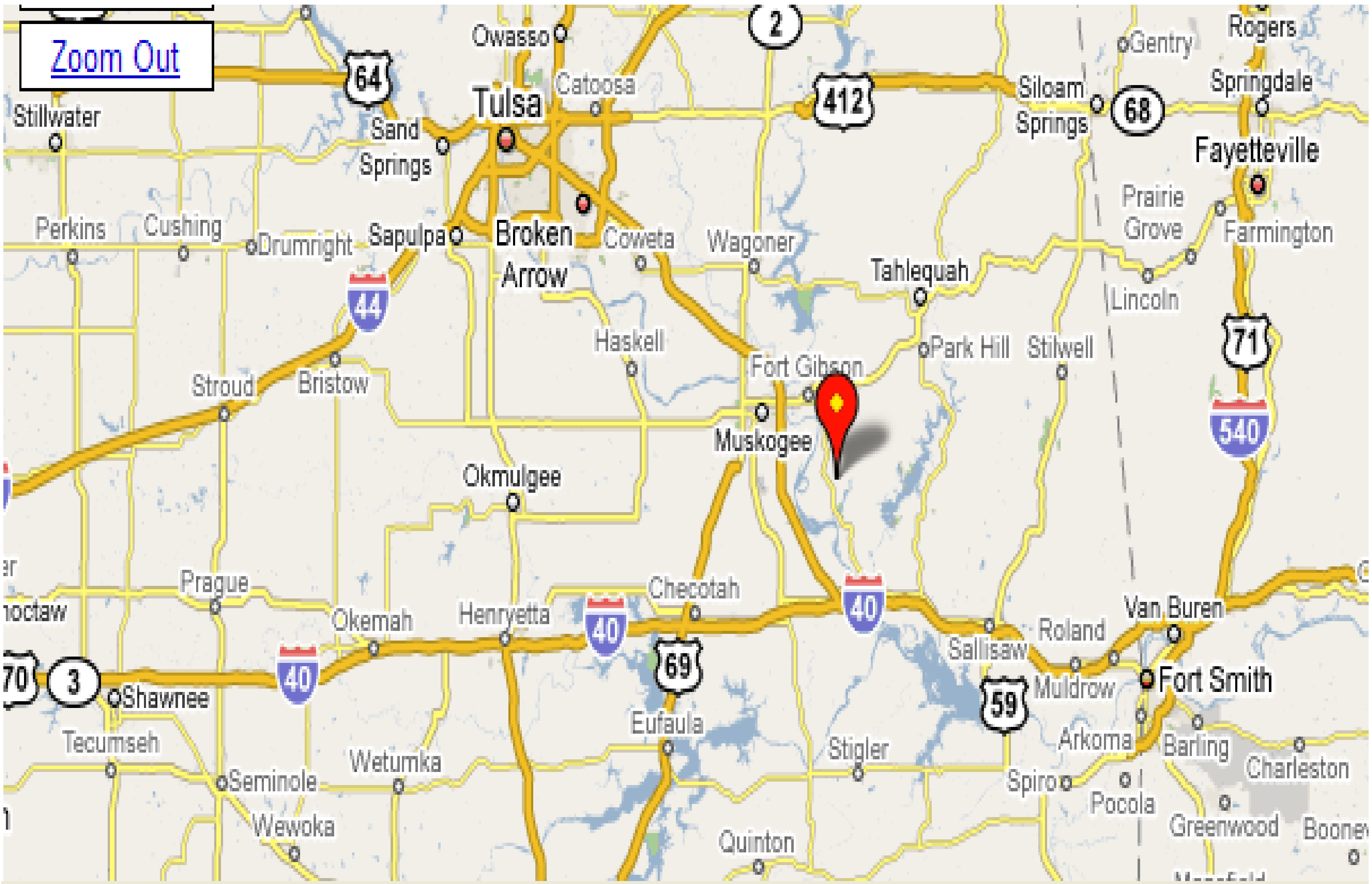

Figure 1.1: Location of the city of Braggs 

Currently the system has 416 service connections and serves 1030 people from its primary water source which is ground water artesian wells. The distribution system network consists of three water towers; one located in the center, one at the north end and one on the south end of the city, giving a total storage capacity of 200,000 gallons. Figure 1.2 shows the central water tower while figure 1.3 shows one of the artesian wells in Braggs. The piping consists mainly of long two inch branches pipes which are interconnected by a few four and six inch supply mains.

Figure 1.2: Central water tower at Braggs, Oklahoma

Figure 1.3: Artesian well at Braggs, Oklahoma

The study was conducted in coordination with the Department of Agricultural Economics at Oklahoma State University as part of a larger project funded by the Oklahoma Water Resources Research Institute (OWRRI). The main aim was to create an easily accessible and cost effective way to help RWDs assess their water distribution infrastructure and plan for improvements.

As part of the OWRRI funded project, a similar study had been carried to assess the water distribution system at the city of Beggs, Oklahoma. The study raised a number of issues, which included high water age at dead ends in the system, aging infrastructure which was likely to result in pipe failures and low pressure at certain points within the distribution system. The studies at Braggs and Beggs both used EPANET, hydraulic simulation software which was developed by the United States Environmental Protection Agency and can be downloaded free from their website at http://www.epa.gov/nrmrl/wswrd/dw/epanet.html. The scope of both studies did not include the design of distribution system components but rather looked at ways of providing economical methods for RWDs to analyze their systems and plan for any improvements that might be deemed necessary to improve quality of service delivered to customers.

The studies generated EPANET models of the distribution systems, data regarding the key components of the systems and proposed improvements to the systems which could prove useful to city authorities for purposes of planning and decision making. Although both studies employed similar methodologies, the study at Braggs only addressed ways of improving the current system conditions in order to improve performance. The study of Beggs also looked at future conditions. The major reason for this was the small population at Braggs which is not expected to increase significantly in the near future.

CHAPTER II

REVIEW OF LITERATURE

2.1 Municipal water demands

The water supplied by a municipal system has two major functions. One is to supply consumer demand, which represents the flow in gallon per minute required to meet daily supply to homes, businesses, institutions and municipal services; and the second to maintain adequate and reliable supply for fire protection. The determination of consumer demands involves assessing the utilization of water based on the three levels of usage below (Hickey, 2008).

The average daily demand reflects the total amount of water used per day and does not consider uses by different classes of occupancy such as commerce and industry. This figure varies considerably by state and region. In 2003, the American Water Works Association (AWWA) estimated this figure to 141 gallons per capita per day on average.

The maximum daily consumption reflects the day within a year long period on which the consumption was highest. The AWWA reports that for any community, this figure is approximately 150% of the average daily demand. The maximum daily consumption is usually reached during the summer months or in the periods of peak demand for industrial use.

The instantaneous flow demand represents the two peak periods of the day between 7a.m to 9a.m and between 5p.m and 7p.m. when consumption is greatest. During these periods, the demand can peak 225% of the average daily demand. These figures must be predicted so that the amount of water delivered to the distribution system and the pressure at any particular point will meet the system requirements.

Municipal water supplies must be able to deliver required fire flows at any time to potential fire risks through properly located fire hydrants. Municipal supplies that do not meet the needed fire flow criteria result in property owners paying higher insurance rates. The decision for a public water supply to provide fire flows can have significant impact on the design and operation of the system. Large amounts of water are necessary to control, confine, and extinguish fires in structures. These quantities often greatly exceed consumer demand. This is the main reason that many small towns with populations less than 5000 do not have fire hydrants (Hickey, 2008).

The amount of water required for fire suppression differs throughout the municipality based on building and occupant conditions. Therefore, water demand for fire protection must be determined at different locations throughout the municipality. The locations are usually selected by the Insurance Services Office (ISO) for purposes of insurance rating and represent typical fire risks such as residential, commercial, institutional and industrial. According to the ISO, the minimum credible water supply is 250 gpm for 2 hours giving a total of 30,000 gallons. Most residential occupancies have a minimum water requirement of 500 gpm and commercial properties can range up to 12,000 gpm for 4 hours (Hickey, 2008).

2.2 Water distribution systems

Water utilities seek to provide customers with a reliable and continuous supply of high quality water while minimizing costs. This is achieved through water distribution systems. Water distribution systems are networks of storage tanks, valves, pumps, and pipes that transport finished water to consumers. Finished water is that which has gone through all the processes in a water treatment plant and is ready for delivery.

During the design of a distribution system, it is necessary to consider the projected lifespan of the system, which is also referred to as the design life; the projected population at the end of the design life; per capita water consumption; the relationship between average and peak demand, and the allowable system pressure and velocity of flow. It is also necessary to determine all design flows that are representative of the occupied regions of the community and any foreseeable expansions. The fundamentals that must be considered in selecting a design flow for the system are average daily demand, maximum daily demand, maximum hourly demand and the required fire flow.

Due to their design, water distribution systems include areas of vulnerability where contamination can occur. Dead ends in the system are usually associated with low water pressure and high water age, and all attempts should be made to eliminate them (ODEQ, 2008). Water traveling through distribution systems comes into contact with a wide range of materials, some of which can significantly change the quality of the water delivered to customers. Corrosion in water distribution pipelines, valves and fixtures, can cause the degradation of drinking water quality (EPA, 2008). Solids can settle out during low flow conditions and can be suspended again during conditions of high flow. Disinfection agents and water additives react with organic and inorganic materials to generate byproducts in a community's water supply and there is also the problem of biofilm formation. (EPA, 2008)

Typical designs consider a flow velocity of 4-6 ft/s. The normal working pressure in a distribution system should be approximately 50 psi and no less than 35 psi at maximum hour. The pressure in most systems will vary between 50 and 56 psi. However, a minimum pressure of 20 psi is required at ground level at all fire hydrants on the system under fire flow conditions. Pipes are commonly designed on the basis of average rather than maximum hourly demands which results in considerably lower investment costs and a reasonable compromise on reliability. Pumps are usually designed to provide the physical head required to fill the water towers and overcome any friction in water distribution system pipes. The pump selected must be able to fill the water tower in 6-12 hours (Salvato, 1992).

Storage facilities within the distribution system enable the system to meet demand when the treatment facility is idle or unable to produce demand. It is more advantageous to provide several smaller storage units at different parts of the system than to provide an equivalent large capacity at a central point within the system. The best economical arrangement is to bring the storage to full capacity at night when there is minimal domestic consumption and then increase when storage falls to 40 or 50 percent during the day (Hickey, 2008).

Storage equalizes demand on supplies, transmission and distribution mains, resulting in smaller facilities than would be required if there were no storage. Storage can also improve or balance system pressure and provides reserve supplies for emergencies, such as power outages. The amount of water required for equalizing water production is

30 to 40 percent of total storage available for water pressure equalization and emergency water supply reserves (Hickey, 2008). However, water storage can have a negative impact on water quality by providing conditions for loss of disinfectant residual, bacterial re-growth, taste and odor production, and formation of disinfectant byproducts as the water age increases. Improper mixing in storage facilities can exacerbate water age problems by creating dead zones with even older water (Grayman et al., 2000).

Domestic supplies are usually fed from the top 25 to 30 percent of the storage capacity, after which controls for high service pumps start in order to satisfy demand and fill the tanks. The remaining 70 to 75 percent is normally held in reserve as dedicated fire storage. The reserve automatically feeds the distribution system when the demand at a certain point exceeds the capacity of the system's high service pumps (Hickey, 2008).

The distribution and location of fire hydrants based on needed fire flows forms an important part of a community's ISO fire suppression rating. The ISO rating is used to establish public protection classifications using a scale of 1 to 10, with 1 representing the best and 10 indicating no recognized fire protection, for establishing insurance rates. For any hydrant to be rated, it must lie within 1000 feet of the building to be protected. Flow tests are conducted to determine whether the hydrants can deliver 250 gpm at 20 psi residual pressure (Hickey 2008).

It is recommended that fire hydrants in congested and high risk areas be no more than 300ft apart and a maximum of 500ft apart in residential areas with building separations of over 50ft. It is also good practice to have fire hydrants installed at every street intersection and near the end of dead end streets. Today it is generally recommended that fire hydrants be installed on pipes that are at least 6 inch diameter (Hickey 2008).

2.3 Rural water systems

A rural water system is a water supply and distribution system that is built for low density, predominantly unincorporated rural areas. Rural water systems primarily serve domestic and livestock needs and usually do not meet fire fighting requirements. A common feature of rural water systems is that they predominantly un-looped and have dead ends (Robinson, 1976). Rural water systems are normally operated by rural water associations. A rural water association is a non-profit corporation whose primary function is to finance, construct, operate and maintain a rural water distribution system.

Approximately 27% of the U.S. population lives in areas defined by the Census Bureau as rural. The Safe Drinking Water Act imposes requirements regarding drinking water quality in rural areas. Many rural communities need to complete water and waste disposal projects to improve the public health and environmental conditions of their citizens (Copeland, 1999).

Numerically, water systems with service areas of less than 10,000 persons account for 94% of all community water systems, yet they supply water to only 20% of the population served by community water systems. The smallest water systems, serving fewer than 3,300 persons, account for 85% of all systems and a similar percentage of systems that are in significant noncompliance with drinking water regulations. Most very small systems have no credit history and have never raised capital in financial markets; while most are non-public entities and thus do not have access to federal grants and loans (Copeland, 1999).

Rural water systems provide drinking water for more than 600,000 people on farms and in small communities in Oklahoma. They are sometimes required to provide free water for volunteer firefighters to help protect homes and property in these areas (OWRB, 2005). Often, these communities have incomplete records and are therefore unable to assess the status of their infrastructure and determine the requirements necessary to accommodate future growth.

The National Ground Water Association advocates the use of the most environmentally sound and cost-effective methods of providing safe water supplies to rural and farm communities. Individual domestic water systems in most areas of the country are the best method of bringing water to rural homes. In a few areas, where there are water quality and/or quantity problems, rural water districts may be a viable alternative (NGWA, 1993). The U.S. Department of Agriculture recognizes individual and cluster wells as an option to long dead end pipes in community-owned water system design. Individual and cluster wells can be integrated into rural system designs to provide an alternative to reliance solely on long-pipe distribution and the huge capital expenditures (NGWA, 1993).

2.4 The effects of design factors and pipe materials

The decision to provide fire flow results in increased water supply pipe diameters, leading to higher capital costs, greater provision for reliability and redundancy in the distribution system. However, it may also have negative impacts on water quality. Since fires are infrequent events, over-sizing the system results in longer water residence time in larger pipes, with the increased possibility of loss of disinfectant residual, thereby enhancing the formation of disinfection byproducts and bacterial growth in water mains.

Water age is a major factor that affects the quality of water in the distribution system. Water quality degrades with time as chlorine residuals decay and disinfection byproducts are formed. Since many distribution systems are sized for fire flow, pipes are significantly oversized with respect to other uses. High water ages result in water quality issues associated with odor, taste and bacterial re-growth. Many utilities solve this problem by periodically flushing the distribution system at dead ends (Talton Jr., 2009).

Decay and formation of disinfection byproducts in distribution systems are influenced by bulk water reactions and pipe wall effects. Bulk water effects can be determined using simple bottle tests, while pipe wall effects can be determined by sampling the distribution system using modeled water ages. Water from different sources can have dramatically different characteristics, so all finished water inputs should be analyzed (Talton Jr., 2009).

Larger pipes are also associated with lower flow velocities in the system, which leads to deposition of sediments. Degradation of water quality in distribution systems has been shown to be a function of the time the water is retained within the system and the low velocities within the lines. Sediments can protect microorganisms from the disinfectant and over time will restrict the flow capacities of the pipes. Therefore, it is necessary to evaluate the need to provide fire flows with the associated operation and maintenance costs against the resulting impact on water quality. The use of hydraulic water quality modeling can be used to evaluate the economic and water quality impacts associated with the provision of fire flows. Observing the residence time of water in the distribution system can be a key indicator of water quality as it plays a major role in determining disinfectant residuals and the formation of disinfectant byproducts.

The commonly accepted design life for a water distribution or transmission main is approximately 50 years. Many pipes, especially older cast iron water mains, which had very thick pipe walls compared to today's AWWA standards requirements, have had useful service lives in excess of 100 years. However, the minimum wall thickness requirements for ductile iron pipes have dropped over the years due to competitive pressure from thermoplastic materials, resulting in an increased frequency of failures in ferrous pipes. Many of the ferrous and polymer-based water pipes will provide adequate service for at least 50 years, especially at reduced service pressures, after which the frequency of failures will rapidly increase. For corrosion related failures, the rate of increase can be exponential depending on the service conditions (EPA, 2009).

Up to the 1940s, water mains were chiefly made of unlined cast iron and steel. Cast iron pipe eventually gave way to ductile iron pipe and ceased being used altogether in the mid 1980s. Today, 56 percent of all underground water mains are cast iron. The primary problem with unlined cast iron pipe is both internal and external corrosion. Internal corrosion causes tuberculation, which can lead to water quality issues and reduced flow and pressure. Internal corrosion can also result in wall thinning that weakens the pipe and form holes that cause leakage or eventually fail. Cast iron pipe is also susceptible to external corrosion if not protected (Sterling et al., 2009).

Graphitization of cast iron pipe is a type of corrosion that weakens the pipe wall by the removal of iron, leaving graphite behind. It is not easily detected, because the appearance of the pipe remains unchanged. Since the relative thickness of cast iron pipe was gradually reduced over the years as production and material technology improved, weakened pipes can fail under much smaller fluctuations in pressure, frost heave, ground movement or thermal stress due to rapid changes in water temperature. For unprotected ferrous pipes, the National Standards Bureau found the rate of corrosion to be similar for all ferrous pipe types. Consequently, younger unprotected cast iron pipe with thinner walls can actually pose a greater failure threat as less time is needed to penetrate the reduced wall thickness. Cast iron pipe is also susceptible to failure at corporation stops due to galvanic action between the two dissimilar materials, which leads to leakage (EPA, 2009).

Asbestos cement was introduced to the U.S. market in the late1940s. Being non-metallic, asbestos cement pipe was not subject to galvanic corrosion. However, soft water will remove calcium hydroxide from the cement and eventually lead to deterioration of the pipe interior due to softening, accompanied by release of asbestos fibers. External exposure to acidic groundwater such as mine waste or sulfates in the soil can also lead to deterioration of the cement matrix. The production of asbestos cement pipe in the U.S. stopped in 1983. However, despite the cessation of production, approximately 15 percent of all water mains today are asbestos-cement. This percentage is almost 20 percent on West Coast where asbestos cement pipe was more widely used (Sterling et al, 2009).

Approximately 22 percent of the existing underground water main infrastructure is ductile iron pipe. Ductile iron pipe was introduced to the utility market in 1955 and eventually displaced cast iron pipe completely. Initially, ductile iron pipe was unlined, but by 1975 most ductile iron pipe marketed for water service was lined with cement mortar. Also, as external corrosion issues were observed, un-bonded loose polyethylene (PE) sleeves were later made available for field application to electrically isolate the pipe from the soil (EPA, 2009).

Thermoplastic pipes, initially in the form of polyvinyl chlorine (PVC) and more recently poly ethylene (PE), have also found use as underground water mains. In the U.S., PVC represents 10 percent of the underground water main infrastructure. Thermoplastic pipes are not subject to electrochemical or galvanic corrosion. Most PVC pipe failures tend to be brittle failures and PVC pipes have experienced premature fatigue-related failures when used in cyclic pressure applications such as irrigation systems and force mains (EPA, 2009).

2.5 Compliance with drinking water regulations

Public water supply systems currently are subject to a number of drinking water regulations issued by EPA under the Safe Drinking Water Act (SDWA). Federal regulations which limit levels of contaminants in treated water are implemented by local water suppliers. Approximately 160,000 public water systems in the U.S are subject to the Safe Drinking Water Act (SDWA) (EPA, 2009). The SDWA requires EPA to establish National Primary Drinking Water Regulations (NPDWR) for contaminants. Mandatory maximum contaminant levels (MCLs) and non-enforceable maximum contaminant level goals (MCLGs) are established by EPA. These require, for example, system monitoring, treatment to remove certain contaminants, and reporting. As new regulations are developed, additional compliance burdens are imposed on all public water systems (Copeland, 1999).

For the quality of drinking water supply, requirements of the Safe Drinking Water Act (SDWA) apply to communities which are served by public water supply systems, both government-owned and privately-owned systems. As defined in this Act, public water supply systems are those having at least 15 service connections. Public water supply systems serve approximately 243 million persons while 16 million households, including 45% of rural communities are served by non-community systems such as individual wells, which are not subject to the Act or its regulations (Copeland, 1999).

EPA estimates that compliance with the regulations already promulgated provides millions of people protection from numerous industrial chemicals, microbes, and other contaminants in public water supplies. However, to comply, many systems have to invest in capital equipment, operation and maintenance, and increased staff technical capacity. Among the regulations with particularly costly implications for small towns are water filtration, lead control, and inorganic and organic contaminant control. Overall, EPA estimates that 68% of total compliance costs for drinking water regulations currently being implemented will fall on those systems that each serve fewer than 3,300 persons (Copeland, 1999).

Rural water systems must therefore comply with stringent federal and state minimum drinking water quality standards. Complying with applicable regulations is often quite difficult for large municipalities and nearly impossible for small rural communities, which generally have a higher percentage of low-income residents and aging infrastructure and far fewer resources. In particular, rural communities are struggling financially to meet new or more stringent arsenic regulations. Since January 2006, many water systems in the Southwest are technically out of compliance with the new standards and will probably remain so indefinitely due to the financial hardships involved in upgrading (Chochezi, 2006).

2.6 Rehabilitation and trends for replacement of infrastructure

The Government Accounting Office (GAO) Performance and Accountability Report 2002 report (GAO, 2003) stated that 33 percent of water utilities did not adequately maintain assets and a further 29 percent had insufficient revenues to even maintain current service levels. The EPA's 2002 report regarding the clean water and drinking water infrastructure gap analysis attempted to reach a common quantitative understanding of the potential magnitude of investment needed to address growing population and economic needs (EPA, 2002). Numerous other studies, including the annual ASCE Infrastructure Report Card (ASCE, 2007), clearly show the impact of lack of significant investment on the performance of aging underground infrastructure in the US (EPA, 2009).

As private homes age, pipes and household plumbing fixtures especially old toilets and faucets, start to leak. For public water systems, the consequences of aging infrastructure are even greater. When water system main lines break, repairs can cause a ripple effect and lead to more breaks in the brittle, old pipes (Chochezi, 2006). To determine infrastructure needs, rural communities must consider when their water systems were installed and predict how long the systems are likely to last. Important factors that influence the appropriate timing of expansions to these systems include recent or expected population increases, regulatory requirements, and funding availability.

After large population increases were seen across the Southwest in the 1940s and 1950s, regulatory requirements for water and wastewater increased in the 1970s, as did funding availability for treatment facilities. Thus, many water systems were built 20 to 70 years ago and will require replacement in the next 20 to 30 years. In-ground piping for both water and wastewater systems can last up to 100 years, while mechanical and electrical equipment in pumping stations may last only 15 years (Chochezi, 2006). The useful life of other components depends on their durability and type of service environment. For example, concrete tanks used in wastewater systems have a shorter life span than those used for drinking water applications, due to the more corrosive nature of wastewater. Pumps that are well-maintained will last longer, as will electric motors operated to minimize starts and stops.

The variety of tools available to utility engineers today is remarkably different from what it was during the 1960s. However, the average rate of system rehabilitation and upgrading is not adequate to keep pace with increasing needs, quality demands and continually deteriorating systems. The opportunity lies in the fact that while the tools being used today are generally effective, there is still considerable room for improvement in existing technologies and/or development of new technologies. Such improvements or new technologies offer the chance to make the investments in rehabilitation more effective and extend the ability of utilities and local governments to fix larger portions of their systems with current funding levels (EPA, 2009).

At the current pace of replacement of less than 1 percent per year and installation of new pipes, the average age of the underground pipe infrastructure will gradually approach the commonly accepted design life of 50 years in 2050. Many pipes have been known to operate longer than their design life, but the frequency of failures increases with the age of the infrastructure. This means that unless a more aggressive rehabilitation program is adopted now, communities are going to be hit with significantly increasing repair costs in the not too distant future (EPA, 2009).

Water systems also are hard to inspect as ordinary visual inspections will reveal little about the structural condition of the mains, and require expensive and/or time consuming temporary services and disinfection in conjunction with rehabilitation (EPA, 2009). Many water utilities simply wait for breakdowns to occur, then fix the problem.

Many states have administrative penalty authority, and various types of formal enforcement actions are possible. However, fines are small in comparison to those for wastewater overflows, so utility efforts tend to focus more on source water and treatment issues rather than distribution and transmission improvements. For example, service interruptions as a result of a failure, inadequate flow, or low pressure, all of which can be very upsetting to the utility customers, do not warrant enforcement action under the SDWA (Sterling et al., 2009).

Effective inspection and condition assessment of water pipe is generally difficult or extremely costly to carry out. Targeting mains for rehabilitation and replacement is largely centered on performance assessment of main break frequency or severity, water quality problems or poor hydraulic characteristics (EPA, 2009). Recently, emphasis on structural defects has shifted to improved leak-detection technologies that seek to reduce the loss of water and quickly identify faulty pipes to reduce the cost of repair and the consequence of failure. Predictive models for deterioration based on pipe materials, ground conditions and failure history are considered useful in identifying the extent of the present and near future needs for rehabilitation.

2.7 Funding for regulatory compliance and system upgrades

Funding needs for regulatory compliance and system upgrades are high. The Drinking Water Infrastructure Needs Survey conducted by EPA in 1999 estimated that small systems would require approximately 31.2 billion dollars over the next 20 years for infrastructure (EPA, 1999). Several federal programs assist rural communities in meeting these requirements. The largest federal programs are administered by the Environmental Protection Agency, but they do not focus solely on rural areas.

The General Accounting Office in its 1980 overview of water issues facing the nation observed that the distribution lines, storage and treatment facilities in many systems need repair or replacement. Lack of revenue has aggravated this situation since water rates charged to users do not provide sufficient revenue to hire trained operators and maintain and operate systems properly (GAO, 1980).

In 1997, EPA reported that small community water systems serving up to 3,300 persons have funding needs of $37.2 billion (27% of the total national need) to provide safe drinking water through the year 2014. It was observed that more than 80% of small systems need to upgrade distribution systems. Two-thirds need to improve their water sources, which are usually wells (Copeland, 1999).

Due to their design, the consumer group they serve, and the number of regulations affecting them, rural water systems are complex and expensive mechanisms to maintain and administer. The majority of systems involve long pipe distribution systems spread over sparsely populated regions and present significant design, construction, and operation challenges (NGWA, 1993). EPA has estimated that, because small systems lack economies of scale, their customers face a particularly heavy financial burden. The smallest cities are likely to experience the largest overall percentage increases in user charges and fees as a result (Copeland, 1999).

Many customers of the Rural Water Districts are not able to pay large monthly bills for their water supply. Therefore, financial assistance programs, in the form of low interest loans and grants, have been established to provide capital for the development of these systems. Some states give financial aid to rural water systems, but primary help comes through the federal government. The Rural Development Administration (RDA) is the principal federal agency that administers a grant and loan program. The federal subsidies provided by this program are intended to make water available to rural users at an affordable price (NGWA, 1993).

Many rural water systems often encounter lower than expected water usage coupled with increases in operating costs. In such cases, a system may be unable to both fund operating expenses and meet their debt obligation. Problems and delays in initial construction may increase capital commitment beyond per user estimates, thus placing systems in difficult financial positions at their outset (NGWA, 1993).

The 1996 Amendment to the SDWA established the Drinking Water State Revolving Fund program which makes funds available to drinking water systems. States can use the funds to help water systems make infrastructure improvements or assess and protect source water. The program also emphasizes providing funds to small and disadvantaged communities and to programs that encourage pollution prevention as a tool for ensuring safe drinking water (EPA, 2009). Unfortunately, the amount of money allocated to the revolving funds has decreased over the years. The amount of money available in the State Revolving Funds (SRFs) is small compared to the amount needed to rebuild the infrastructure (Sterling et al, 2009).

In Oklahoma, Drinking Water State Revolving Funds, a low-interest loan program administered cooperatively between the OWRB and Oklahoma Department of Environmental Quality, assists communities with public water supply infrastructure construction projects. Communities that are considered disadvantaged may be eligible for extended term financing, up to 30 years, under this program.

SRFs are insufficient to cover the costs for rehabilitating aging water systems. Rate structures for pubic water utilities typically are not designed to provide the level of funds needed either. Water rates are politically sensitive and, without significant increases over the years, often do not even cover the cost of providing clean water to stakeholders. One of the paradoxes of the water industry is that life is impossible without water, yet we are only willing to pay a fraction of the cost of what it takes to deliver safe, clean water to our homes and offices. Water rates have historically been set at levels that do not reflect the true value of water, and politicians have been reluctant to adopt rate structures that would provide the necessary funding to make water utilities self supporting and sustainable. Public utilities will need to find a way to raise rates to match the value of water to society to make money available for renewing the aging infrastructure. In recent years, the federal government has been unwilling to step in and provide those funds (EPA, 2009).

Drinking water systems historically have relied on state and federal grants to perform periodic system improvements and upgrades. However, grant amounts have diminished in recent years while the cost to replace system infrastructure increased tremendously. Small systems find themselves ill-equipped to meet the financial burden. Although low-interest loans are available, community water boards and utility customers often balk at incurring long-term debt to finance system improvements. In order to qualify for low-interest loans and grants, water systems must show that their rate structures sufficiently meet annual operations and maintenance expenses, debt service payments, and a variety of reserve accounts that cover items such as emergencies, debt reserve, capital improvements, and operations.

After recognizing that grant funds appropriated to small systems in the past often provided only a temporary fix, state and federal sources are currently awarding fewer projects with larger amounts, with the aim of completing single or multiple phases of infrastructure improvements to maximize funds and address public health and welfare concerns. Funding also is being directed toward regionalizing small systems in an attempt to resolve ongoing issues such as billing and collections, certification of water operators, and water quantity and quality control (Chochezi, 2006).

Presently, funding is directly related to a system's ability to sustain itself over the long-term. Sustainability is linked to water rates, membership fees, planning, collaboration and cooperation with neighboring systems, and water conservation. This means that water systems must now operate as successful businesses to survive.

2.8 Personnel requirements

For small systems to meet these challenges and serve their customers into the future, they require strong leaders with advisory, managerial, and technical skills. Technical leadership unlocks the power of new technologies and is the foundation for affordability and reliability. Existing physical deficiencies and compliance problems may stem from underlying personnel issues. Small rural systems do not have the management resources of their larger, urban peers. Many rural systems do simply do not have the resources to hire and retain qualified personnel to manage and operate their networks. Therefore responsibilities are often shared among multiple individuals rather than a single specialist, increasing the need for coordination (Chochezi, 2006)

2.9 EPA drinking water research programs

The US Environmental Protection Agency is mandated to formulate and implement actions that ensure a sustainable balance between human activities and the ability of natural systems to support and nurture life. In order to meet this mandate, EPA has research programs that provide technical support and data necessary to solve today's environmental problems. The National Risk Management Research Laboratory is EPA's center for investigation of technology and approaches to reduce risks to humans and the environment. One part of the goals of this Institution is to provide technical support and information transfer to ensure effective implementation of environmental regulations and strategies (EPA, 2000).

EPA's drinking water studies are based on the multi-barrier concept of selecting the best available water source and protecting it from contamination, using water treatment to control contaminants, and preventing water quality deterioration in distribution systems. EPA's Environmental Technology Verification (ETV) Program develops testing protocols and verifies the performance of innovative technologies that have the potential to improve the protection of our drinking water. ETV has verified monitoring and treatment technologies for drinking water distribution systems. (EPA, 2008)

EPA announced the availability of the Check Up Program for Small Systems (CUPSS) in 2008. CUPSS is a user-friendly computer-based program that assists owners and operators in developing and using plans for maintaining their systems and providing service to their customers (EPA, 2008). CUPSS was developed by the Office of Water as part of the Agency's Sustainable Infrastructure Initiative. "The effort received input from a large stakeholder workgroup, including representatives from several states, the National Rural Water Association, the Rural Community Assistance Partnership, and Environmental Finance Centers" (EPA, 2008).

CUPSS is intended to assist EPA's partners by giving them a tool to better preserve and enhance America's water resources and is expected to make a difference by helping to bridge the growing financial gap faced by small drinking water and wastewater systems as they repair, and replace infrastructure (EPA, 2008).

"The CUPSS program uses information provided on the system's assets, operation and maintenance activities and financial status to produce a prioritized asset inventory, financial reports and a customized asset management plan. Asset management programs support informed budget discussions, boost efficiency of the utility, and improve customer service by ensuring clean and safe water at competitive prices" (EPA, 2008)

2.10 Water distribution system modeling

It is difficult to rely on monitoring data alone to understand the fate of substances as water moves within a distribution system. Distribution systems are typically made up of miles of pipelines making it impossible to achieve widespread and effective monitoring. The flow paths are highly variable due to varying demands, the often looped nature of systems and the common use of storage tanks. The pipes may receive new water from the treatment plants during the filling of tanks and old water while the tanks are being emptied (Hickey 2008).

It is impractical to physically determine how changes in treatment plant operations will affect the quality of water received by the consumer. For these reasons, mathematical modeling of water quality behavior in distribution systems has become attractive to support monitoring. Distribution system modeling involves the use of a computer model of the system to predict its behavior and solve a variety of design, operational and water quality problems. Models can simulate complex demand scenarios such as the occurrence of a major fire, predict the pressures in a system and compare the operation performance against design standards.

Models provide a cost effective way to study the variation of water quality constituents such as the fraction originating from a particular source, the age of water at different points in the system, the concentration of a non reactive tracer in the system and the concentration and rate of loss of a secondary disinfectant such as chlorine. Models can also assist in determining modifications necessary to improve the performance of the system, such as modifying operation to blend water from different sources, pipe replacement, and reduction of storage holding time and disinfectant injection rate at booster stations to maintain residual levels within the system.

To aid in its research activities, EPA uses Windows-based software called EPANET to model water distribution piping systems (EPA, 2008). EPANET is a computerized simulation model which was developed by the National Risk Management Research Laboratory to help water utilities meet the growing need to better understand the movement and transformations undergone by treated water that is introduced into their distribution systems in order to meet regulatory requirements and customer expectations. EPANET performs extended period simulation of the hydraulic and water quality behavior within pressurized pipe networks (EPA, 2008). It predicts dynamic hydraulic and water quality behavior within a drinking water distribution system operating over an extended period of time (EPA, 2000).

2.10.1 EPANET

EPANET is a Windows-based program that performs extended period simulation of the hydraulic and water quality behavior within pressurized pipe networks. The software is available free to the public.

“EPANET was developed to help water utilities maintain and improve the quality of water delivered to consumers through distribution systems. It can also be used to design sampling programs, study disinfectant loss and by-product formation, and conduct consumer exposure assessments. It can assist in evaluating alternative strategies for improving water quality, such as altering source use within multi-source systems, modifying pumping and tank filling/emptying schedules to reduce water age, using booster disinfection stations at key locations to maintain target residuals, and planning cost-effective programs of targeted pipe cleaning and replacement” (Rossman, 2000).

Distribution system networks consist of pipes, nodes (pipe junctions), pumps, valves, and storage tanks or reservoirs. “EPANET tracks the flow of water through each pipe, the pressure at each node, the height of the water in each tank, and the concentration of different chemicals throughout the network during a simulation period”. “The software provides an integrated computer environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats including color-coded network maps, data tables, time series graphs, and contour plots" (Rossman, 2000).

EPANET provides an extended-period hydraulic analysis package that can handle systems varying size, compute friction head loss using the Hazen-Williams, the Darcy Weisbach, or the Chezy-Manning head loss formula, including minor head losses for bends and fittings. It can model constant or variable speed pumps, compute pumping energy and cost, model various types of valves and storage tanks of different shapes. It can consider multiple demand categories at nodes, each with its own pattern showing variation with time variation, model pressure-dependent flow from emitters and base system operation on simple tank level or timer controls as well as on complex rule-based controls (Rossman, 2000).

EPANET can be used to plan and improve a system's hydraulic performance and assist with pipe, pump, and valve placement and sizing. It can also be used to determine ways for minimizing energy usage; conduct fire flow analyses; vulnerability studies; and operator training programs (Rossman, 2000). EPANET has a water quality analyzer that can model the following:

- the movement of a non-reactive tracer material through the network over time
- the movement and fate of a reactive material as it grows or decays over time
- the age of water throughout a network,
- the percent of flow from a given node reaching all other nodes over time.
- the reactions both in the bulk flow and at the pipe wall
- growth or decay reactions that proceed up to a limiting concentration,
- global reaction rate coefficients that can be modified on a pipe-by-pipe basis

- time-varying concentration or mass inputs at any location in the network
- storage tanks that are complete mix, plug flow, or two-compartment reactors

(EPA, 2000)

EPANET's Windows user interface provides a network editor that simplifies the process of building piping network models and editing their properties. Various data reporting and visualization tools such as graphical views, tabular views, and special reports, and calibration are used to assist in interpreting the results of a network analysis (EPA, 2000).

2.10.1.1 Physical components of the network model

EPANET models a water distribution system as a collection of links connected to nodes. The links represent pipes, pumps and control valves while the nodes represent junctions, tanks and reservoirs. The figure below shows the physical components of a distribution system (Rossman, 2000).

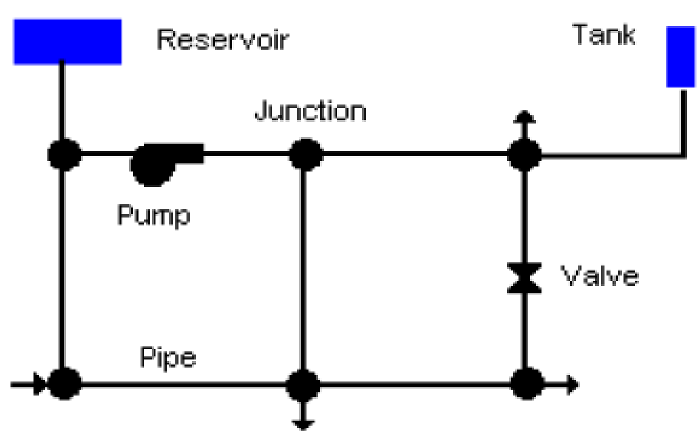

Figure 2.1: Physical components in a distribution system (Source: Rossman, 2000)

Junctions are points where links join together or where water enters or leaves the network. The model requires elevation, usually the mean above sea level, water demand

and initial water quality as input parameters and it outputs hydraulic head, pressure and water quality for each node. Junctions can have varying demand, have multiple categories of demand assigned, have negative demand implying that water is entering the network, and be water quality sources where constituents enter the system or contain sprinklers that make outflow rate dependent on pressure (Rossman, 2000).

Reservoirs are nodes that represent an external source of water to the network. They are used to model such things as lakes, rivers and ground water aquifers. They also serve as water quality source points. The input parameters for reservoirs are hydraulic head and water quality. Reservoirs are boundary points to the network and their head and water quality can not be affected by what happens in the network. However, the head at the reservoir can be set to vary with time (Rossman, 2000).

Tanks are nodes that have storage capacity. The volume stored can vary with time during simulation. The input parameters for tanks are the bottom elevation where the water level is zero, the diameter, the maximum and minimum water level and the initial water quality. The model computes and outputs hydraulic head and water quality over the simulation period. Tanks are required to operate between their maximum and minimum levels and EPANET will stop inflow at the maximum level and outflow at the minimum level (Rossman, 2000).

Pipes are links that convey water from one point in the network to another. EPANET assumes that all pipes are full all the time. Flow direction is from the end at higher hydraulic head to the end with lower hydraulic head. The hydraulic input parameters for pipes are start and end nodes, diameter, length, roughness coefficient and status. The model computes and outputs the flow rate, velocity of flow, head loss and the Darcy Weisbach friction factor (Rossman, 2000).

Pumps are links that impart energy to a fluid, thereby raising its hydraulic head. The input parameters are the start and end nodes and the pump curve which represents the combination of heads and flows that the pump can produce. The output parameters are flow and head gain. Flow through pumps is unidirectional and EPANET will not allow a pump to operate outside the range of its pump curve (Rossman, 2000). Pumps can be turned on and off at preset times or when certain conditions exist in a network. EPANET can also compute the energy consumption and cost of a pump. Each pump can be assigned an efficiency curve and schedule of energy prices. In the absence of these, a set of global energy options is used (Rossman, 2000).

Valves are links that limit the pressure or flow at a specific point in the network. The input parameters for valves are start and end nodes, diameter, setting and status. EPANET outputs are flow rate and head loss (Rossman, 2000).

2.10.1.2 Non physical components of the network model

These are informational objects of the model that are used in addition to the physical components. They include curves, patterns and controls that describe the behavior and operational aspects of a distribution system (Rossman, 2000).

Curves contain data representing a relationship between two quantities. Two or more objects can share a curve, and EPANET uses a number of curves. Pump curves represent the relationship between head and flow rate that a pump can deliver. A valid pump curve must have decreasing head with increasing flow. The shape of the curve used by EPANET will depend on the number of points provided. Efficiency curves define the pump efficiency as a function of pump flow rate. It is used for energy calculations. If it is not supplied, then fixed global pump efficiency is used. Volume curves determine how storage tank volume varies as a function of water level. It is used to model non cylindrical tanks where the cross-sectional area of the tank varies with height. Headloss curves are used to describe the headloss through a general purpose valve as a function of flow rate. They give the capability to model devices and situations with unique headloss-flow relationships (Rossman, 2000).

Time patterns represent a set of multipliers that can be applied to a quantity to allow it to vary with time. Time patterns can be applied to demands at nodes, reservoir heads or pumps schedules at time intervals set by the user (Rossman, 2000).

Controls are statements that describe how the network is operated over time. They specify the status of selected links function of time, tank water levels and pressures at given points in the network. The controls input to the software may be simple or rule based. Simple controls change the status of a link based on water level in a tank, time of day, time into a simulation or pressure at a junction and there is no limit to the number of simple control statements that can be used. “Rule-based controls allow for link status and settings based on a combination of conditions that might exist in the network after an initial hydraulic state of the system is computed. For example, a set of rules that shut down a pump and open a bypass valve when the level in the tank exceeds a certain level” (Rossman, 2000).

2.10.1.3 The hydraulic simulation model

EPANET’s hydraulic simulation model computes junction heads and link flows for a fixed set of reservoir levels, tank levels and water demands over a succession of points in time. These parameters are updated from one time step to another according to prescribed time patterns, while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving the conservation of flow at each junction and the headloss relationship across each link in the network, in a process which is known as hydraulic balancing. The process uses an iterative technique to solve the non linear equations involved. The hydraulic time step for an extended time simulation is set by the user and the default value is one hour (Rossman, 2000).

2.10.1.4 The water quality simulation model

EPANET also has a water quality simulator that uses a time based approach and tracks the fate of discrete parcels of water as they move along pipes and mix together at junctions between fixed lengths of pipe. The water quality time steps are shorter than the hydraulic time steps. The chemical concentration and size of a series of non overlapping segments of water that fill each link are tracked. For each water quality time step, the contents of each segment are subjected to reaction. An account of the total mass and flow volume entering each node is updated and new node concentrations are calculated (Rossman, 2000).

EPANET also models the changes in the age of water throughout a distribution system. Water age is the time spent by a parcel of water in the network. New water entering the network from the source nodes or reservoirs has an age of zero. Water age provides a simple, non-specific measure of the overall quality of the drinking water delivered. EPANET treats water age as a reactive constituent with zero order kinetics (Rossman, 2000).

2.11 Sources of water distribution system information for Oklahoma

OWRB published a document named the “Rural Water Systems of Oklahoma” in 1980 and 1988. The latest online update of Oklahoma's rural water supply systems includes the most comprehensive and concise information available on these vitally important facilities. Water Board staff continue to collect and maintain digital information from more than 750 individual rural water systems located throughout Oklahoma. Geographic Information System (GIS) attribute data including water line sizes and system/municipal boundaries are available for each system, along with contact information, system size, population served, and type of water source. GIS coverage of the water systems and lines can be downloaded and updated system maps may also be viewed through the OWRB's Water Information Mapping System (WIMS) (OWRB, 2008).

This information assists in the development and improvement of existing systems and serves as an important tool for making local economic development decisions. It is particularly useful to system managers, engineers, water resource managers, and planning officials. Systems commonly utilize the information in planning system extensions, merging customer information, and reducing response times to local emergencies (OWRB, 2008).

CHAPTER III

METHODOLOGY

This chapter describes the sources of information and the methodology used to attain the goals that were set out at the beginning of the study.

3.1 Obtaining system information for Braggs

The map of the Braggs water distribution system was obtained from the Water Information Mapping System (WIMS) on the Oklahoma Water Resources Board (OWRB) website at http://www.owrb.ok.gov/maps/server/wims.php. WIMS is an Internet-based map server that requires a supported web browser. WIMS enables users to create custom maps by selecting an area of interest and map features (layers) to display, such as water resources, political boundaries, geology, and aerial images.

The website contains the latest online update of Oklahoma's rural water supply systems and has information for over 750 Rural Water Districts, including water line sizes, system/municipal boundaries, and the location of facilities such as pumps, wells and tanks. The GIS (*.shp) files can be freely downloaded from the website.

Information regarding the age of the system, problems related to inadequate flows, low water pressure, leakages and bursts water usage patterns and equipment information for pumps was obtained from interviews with the plant operator at Braggs, Oklahoma.

Water usage data were obtained from the Oklahoma Department of Environmental Quality (ODEQ) records. The records included information regarding the total water pumped daily from the treatment plant, the pH and the doses of the different chemicals added to the water prior to distribution over an eight year period from January 2001 to April 2009.

The US Census Bureau and the Oklahoma Department of Commerce records were used to provide the latest information regarding population for Braggs. Census Block information is also available from the Environmental Research Institute (ESRI). However, due to its small size, the entire city of Braggs comprises one census block.

3.2 Creating a model of the pipeline for use with EPANET

Copies of the *.shp files showing the facilities, pipeline drawings and system boundaries for the entire State of Oklahoma were opened in GIS and the boundary for Braggs was used to trim the information for all the other rural water districts leaving the only the system attributes of the Braggs RWD. The shp2epa utility converter was used to extract data from the *.shp files to create *.inp network files for EPANET. This program allows the user to assign prefixes to junctions and pipes and obtain a working model of the system in EPANET. However the nodes have no elevations, the pipes have no diameters, many of the pipes are not joined and there may be duplicate pipes in the system.

A point file with X and Y coordinates of the junctions and vertices was created using the shp2epa program. This file was loaded into an elevation file for Muskogee County in Global Mapper in order to determine the elevations at the junctions. The vector data from Global Mapper was exported as a simple text file of the form *.xyz.

The three files of the system *.shp, *.inp and *.xyz were loaded into a macro enabled Excel spreadsheet for editing. The spreadsheet was used to carry out the following tasks:

- Assign elevations to the nodes and replace X, Y coordinates with the re-projected ones
- Add pipe diameters and missing pipe lengths
- Join pipes with the same nodes and eliminate duplicate nodes
- Find and eliminate pipes with zero length
- Count the number of times a node is used
- Find and eliminate duplicate pipes
- Locate unconnected pipes
- Attach pipes that cross but have no connecting nodes or where the node is present but pipes are unconnected
- Create a new *.inp files for EPANET

The new *.inp was opened in EPANETZ, a modification of the EPANET from Zonum solutions. The software can be obtained free from their website athttp://www.zonums.com/epanetz.html. EPANETZ allows the pipelines to be viewed over internet-based maps. The network was compared to the hand drawn maps of the system availed by the operator to confirm accuracy of the information obtained from the OWRB *.shp files. Modifications to the system were then made using EPANETZ to include the information from the hand drawn maps which was missing *.shp files.

After a working model of the system was generated in EPANET, the next step was to assign demand to the different nodes in the system. Information regarding water demand is based on population to be served. In order to determine the demand at each junction, it was necessary to determine the population that could be served from the junction. The households that are served by the system were manually located from a photomap of Muskogee County obtained from the National Resource Conservation Service's Geospatial Data Gateway. Using ArcMap and clicking on households in the photo map, a record of their coordinates was created and a *.shp file was generated. The *.shp file for the households created was edited in Global Mapper to create an *.xyz file for the households. This *.xyz file was loaded into the macro enabled Excel spreadsheet which then assigned demands to the junctions in the system. The Excel spreadsheet compares coordinates of the junctions to the households and assigns the demand from a particular household to the nearest node.

The macro enabled spreadsheet requires input of the average consumption in gallon per capita per day, the number of household meters and the population served in order to assign demands to the junction. The per capita consumption was determined by comparing the water pumped daily over a period of three years from April 2006 to April 2009 to the population served, which was obtained from the US Census Bureau data for Braggs, Muskogee County in Oklahoma.

3.3 Hydraulic modeling using EPANET

The process of modeling a network using EPANET involves input of the parameters or variables that most closely describe the operation of the actual system. These parameters include the shape of the tanks, the pump curve which describes the operation of the pump and an infinite reservoir. Other input parameters required for the model to run include the maximum and minimum water levels and an initial water level in the tank. The default shape for tanks in EPANET is cylindrical which allows the user to input the diameter and height of water in the tank. However, EPANET can model odd shaped tanks. In this case, a volume curve that shows how volume changes with water level in the tank would have to be included in the input parameters. The three water tanks at Braggs are all cylindrical in shape so it was not necessary to include volume curves.

The pump curve shows the relationship between the head and volume delivered by the pump. A set of controls that shows how the pump operates can be input to model when the pump is running. For example, the pump can be set to start when the water level in the tank drops below a certain level and off when it exceeds another preset level. This ensures that the pump is not constantly running, which increases its operational life while ensuring that the towers never run dry. There are three identical pumps at Braggs, each delivering 150gpm at 208ft of head. The pumps operate in parallel delivering the same head and are set to sequentially come on line in order to meet increasing flow requirements for the system. The pumps were modeled according to the information received from the system operator. Usually a single pump is switched on when the pressure drops below 65psi and is switched off when the pressure is exceeds 80psi. Therefore, rule based controls were set to ensure that the first pump was switched on when the pressure dropped below 65psi and switched off when the pressure increased to 80psi. Pump 2 was modeled to switch on if the pressure dropped further as would be the case in the event of a fire. Pump 3 was treated as a standby for the system in case pumps 1 or 2 failed to operate and was not included in the hydraulic modeling process.

The EPANET hydraulic model requires pipe roughness coefficients in order to calculate the friction losses as water moves within the distribution system. Pipe roughness coefficients can be calculated using the Bernoulli's equation if the water flow and pressure at the ends of a straight uninterrupted pipe are known. However this method is often not practical and the pressure drops that occur are usually too small to be measured by typical pressure gauges. Simulating differences in diameter between new and old pipes caused by a buildup of scale can also be used to estimate the roughness coefficients for old pipes. EPANET has a preset roughness coefficient of 100 as an estimate for old cast iron pipes. However, roughness coefficients are available in literature for new and used pipes of different materials.

The greatest percentage of the pipes at Braggs was installed in 1982 when the currently existing PVC pipes were installed to replace deteriorated cast iron pipes that had been previously installed in the 1940's. Therefore, most of the pipes are almost 30 years old. The operator noted that they had not replaced any pipes recently.

From review of existing literature, it was established that the roughness coefficient for new PVC pipe is in the range of 140 to 150. It was also determined that after 25 years of service, the roughness coefficient of PVC pipe is still about 140. This figure does not drop below 130 even in excess of 50 years of service. This is because PVC is smooth, does not corrode and there is rarely a significant build up of scale to constrict the pipe diameter. In order to be conservative, the roughness of 130 was used to calculate the friction losses in the hydraulic simulation of the water distribution system at Braggs.

According to the Oklahoma Department of Commerce, the population of Braggs, Oklahoma, was 308 in 2007. This corresponded to a 2.3% increase from the population recorded in the 2000 census. Population projection is very important in determining the future water supply requirements for a given area. The Braggs water distribution system was modeled in order to determine improvements to its operation under current conditions. Therefore all recommendations simulations were done for the current population of 308 for the city and a total population served of 1,030 people.

CHAPTER IV

FINDINGS

This chapter details the observations made after running simulations of the model of the Braggs water distribution system using EPANET. A simulation of the current conditions was done for year 2009 and another simulation was carried out to examine conditions after the implementation of proposed changes. During the simulations, changes in selected parameters such as flow velocities, water pressure at nodes and water age were observed and are discussed in the following sections.

4.1 Water distribution system information for Braggs

The city of Braggs obtains its water from artesian wells. The water is good quality and the only treatment is chlorination using sodium hypochlorite as well as addition of caustic soda to increase the pH. From the wells, water is pumped to three water towers.

The total water storage at Braggs is 200,000 gallons which is split between three water towers. There is an 85,000 gallon tank located in the center of the city, a 65,000 gallon tank to the north and a 50,000 gallon tank to the south of the city. The plant operator availed the diameters and storage capacity of the tanks. The heights of the tanks were calculated using this information. Table 4.1 shows the storage capacity, the diameter and calculated height of the three water towers at Braggs.

Table 4.1: Water Storage Facilities at Braggs, Oklahoma

Storage Tank	North Tower Tank 3	Central Tower Tank 1	South Tower Tank 2
Capacity (gallons)	65,000	85,000	50,000
Diameter (ft)	10.5	12.0	10.5
Height (ft)	100.5	100.5	77.2
Difference in elevation from well house (ft)	92.0	20.0	88.0

There are a few 6 inch and 4 inch diameter main pipes; however the distribution system consists mainly of 2 inch pipelines. The material used for the distribution system piping is PVC. The operator noted that they sometimes experience problems associated with low pressure at certain points within the system. However, they had not had to replace any pipes due to build up of scale, leakages or pipe bursts recently.

Figure 4.1 shows the EPANET model of the pipeline for Braggs Rural Water System that was developed by updating information from OWRB *.shp files with the system operator's hand drawn maps. The numbers adjacent to the nodes with the prefix BrJ are the identifiers used to refer to different nodes discussed in the following sections of this text. The area to the right of Figure 4.1, where the junctions clustered closely together, is the center of the City of Braggs, which is enlarged and shown in Figure 4.2.

4.2 Hydraulic simulation of existing conditions

A flow rate of 75,600 gallon per day, which represents an average daily demand of 52.5gpm, was used to carry out a simulation of the current conditions. This figure was obtained from the well house records and averaged over three years from April 2006 to April 2009. These records include all the water pumped into the distribution system and therefore account for unmetered water and all water that is lost due to leakages in the system.

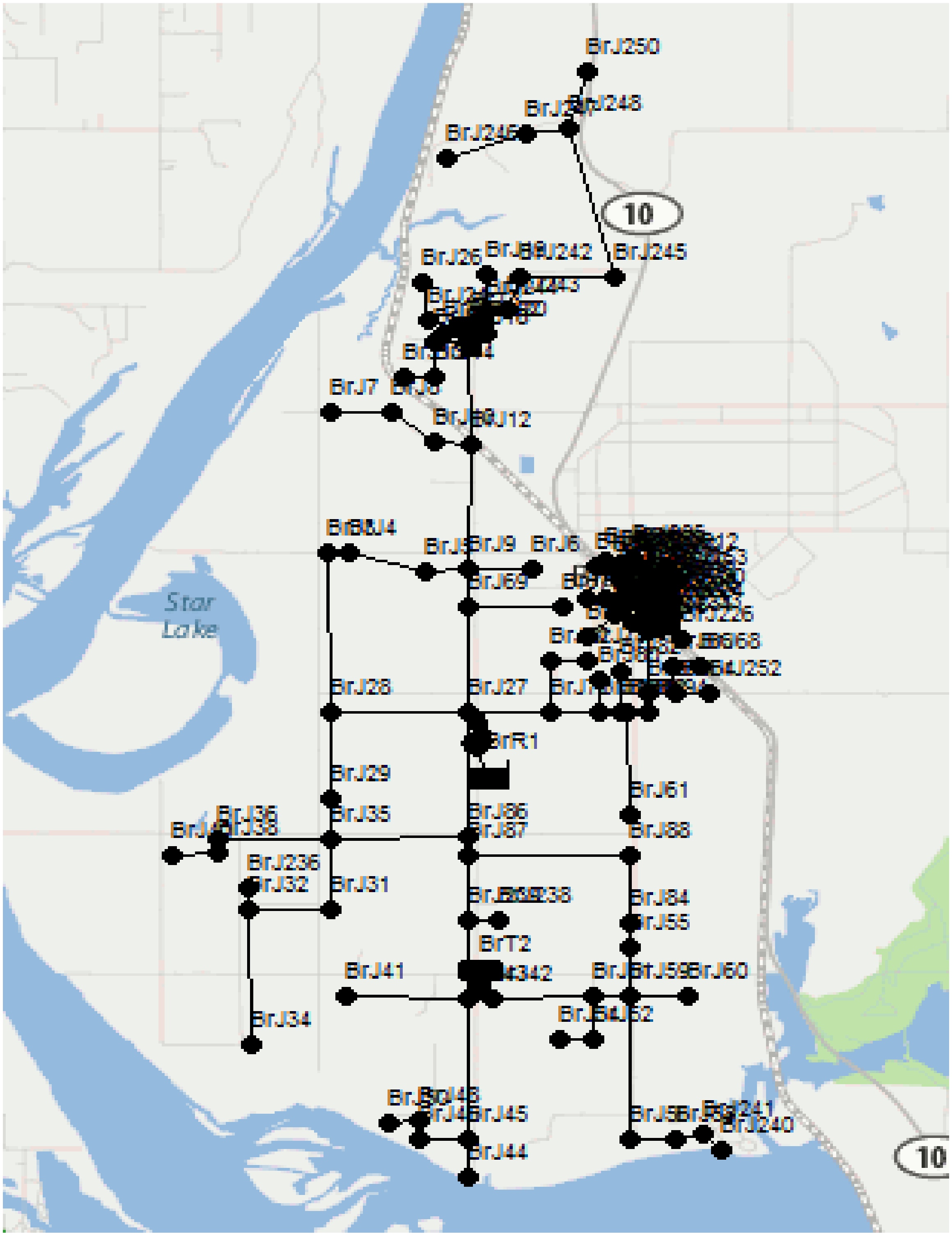

Figure 4.1: EPANETZ model of Braggs Rural Water System (Node identification)

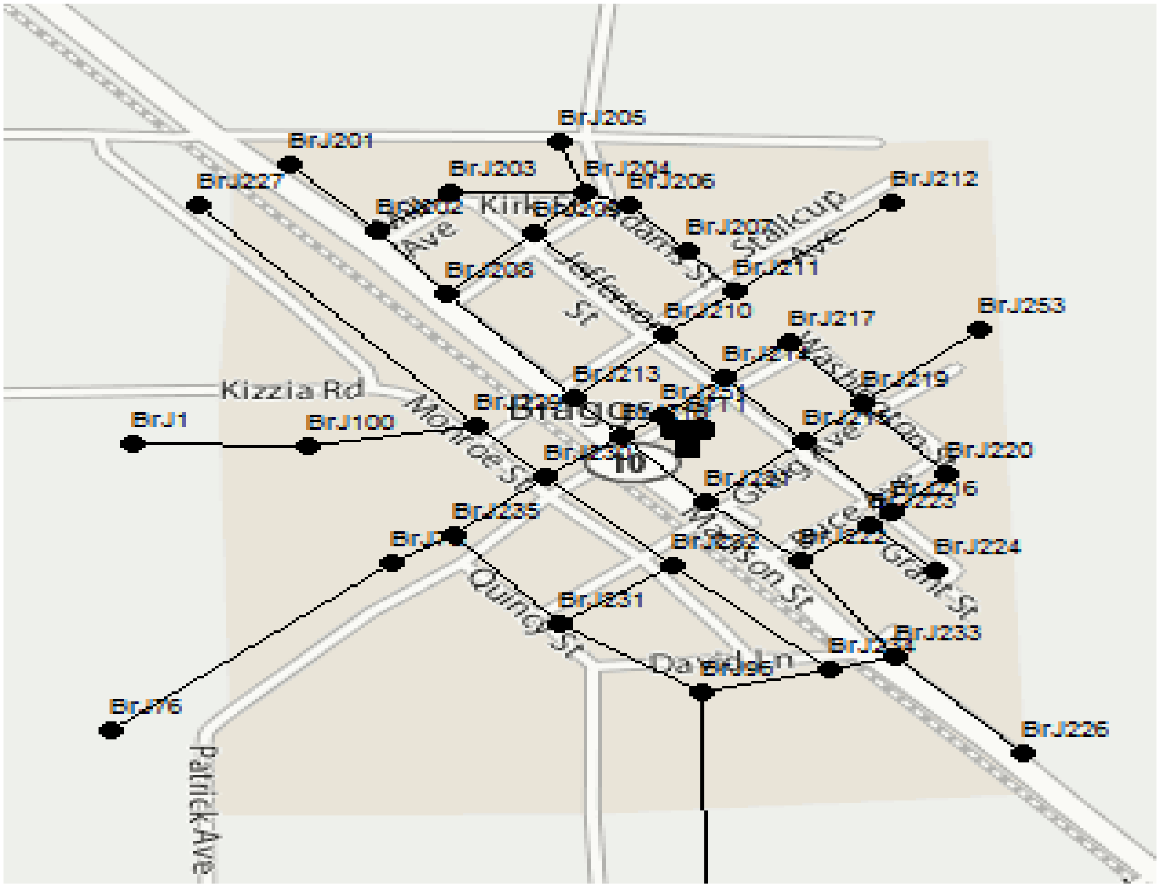

Figure 4.2: Layout of pipelines within the city of Braggs

The highest demand of the system occurs at node BrJ206 to which a school with 270 students is connected. The system was modeled with three demand patterns. The first pattern was assigned to all the nodes except BrJ206 and had 2 hour peaks between 7 and 9am and 5 and 7pm. The pattern allocated a demand that is 2.25 times the average daily demand during these periods. A separate demand pattern which assumed that demand at the school only exists between 9 and 4pm was assigned to BrJ206 to which the school is connected. A third pattern was defined for the nodes to which fire hydrants would be connected. The operation of the fire hydrants was tested under conditions of peak demand; therefore this pattern only assigned flow to the fire hydrants for two hours during conditions of peak demand.

There are three identical pumps at the pump station adjacent to the treatment plant each delivering 150gpm at 208ft while operating at 74 percent efficiency. The pumps are set to start in sequence to increase flow according to the demands in the system. The curve which shows how flow delivered by the pump varies with head is shown in Figure 4.3. The curve was generated by inputting information read from the hard copy obtained from the operator into EPANET.

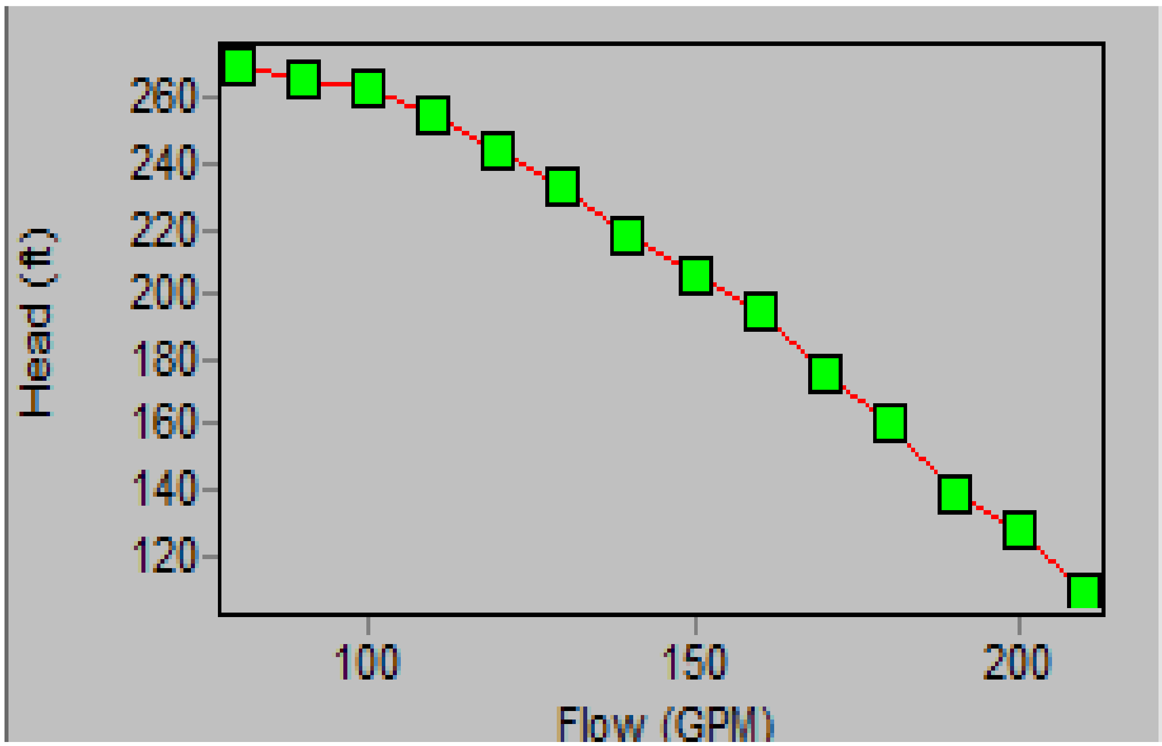

Figure 4.3: Operating curve for the pumps at Braggs

Hickey (2008) reveals that domestic water supplies are typically fed from the top 1 to 25 feet of the storage in elevated tanks and the rest held in reserve as fire storage. He notes that as the level falls below the top 25 percent of the total storage, tank controls activate high service pumps in order to satisfy the system demand and refill towers. This assumption was made for the system at Braggs and set as a rule based control in addition to the cut off pressures for operation of the pumps that were availed by the operator.

The minimum level in the tanks was set at 40ft to ensure a minimum pressure to drive water within the distribution system. The initial water level in the tanks was also set at 50ft to ensure that tanks would fill up at the beginning of the simulation. The maximum water level in the tanks was set at half a foot from the top of the tank.

The system operator mentioned that the last pressure testing of the system had been done in June 2009 at a point one mile to the east of the northern tower. The pressure observed there was 36psi. By taking measurements from the map, it was concluded that this point was at junction BJ245. A plot of the pressure variation the junction shown in Figure 4.4 was used to estimate the accuracy of the results generated by the model. From the figure, it is observed that pressure at the junction varies between 34 and 34.68psi during periods of regular demand and falls to 28.5psi during peak demand. The water level in tank 3, which serves node 245, varies from 70.5ft during periods of low demand and drops to 56ft during periods of high demand.

EPANET evaluates various system characteristics such as velocity of flow in pipes, water age and pressure at various nodes as a function of time. The system characteristics stabilize at different times. The tanks fill up at the beginning of the simulation, and it takes 4, 5 and 8 hours for the water level to reach the maximum height in tanks 1, 2 and 3 respectively. Tank 3 takes the longest to fill up because the smaller 4 inch mains from the well house to the tank deliver a significantly smaller quantity of water than the 6 inch mains that lead to the other two tanks. At the set operating condition, each of the pumps delivers 150gpm at 208ft. Tanks 1 and 3 are both100.5ft high while tank 2 is 77.2ft high.

The difference in elevation between the water treatment plant and the base of the tank is approximately 20ft for tank 1 compared to approximately 88ft and 92ft for tanks 2 and 3 respectively. Therefore, tank 1 fills up completely while the water in tanks 2 and 3 reaches a maximum of 76ft and 71ft respectively. The top 30ft of tank 3 is not filled with water and as a result, the north of the city is very dependent on the operation of the pump for reliability of water pressure.

The hydraulic conditions of the system were examined under steady state conditions to determine the junctions that had unusually high or low pressure. Tanks 2 and 3 continue to fill up after tank 1 is full and tank controls shut off water supply to the tank. As a result, pressure increases are observed to the south of the water treatment plant for approximately one hour until the water level in tank 2 reaches its maximum height. During this period, a number of junctions experience sustained pressures in excess of 100psi. Salvato (1992) noted that for most water distribution pipeline materials, sustained pressures in the region of 100 psi can cause leakages and even system failure. The nodes with high pressure were of key interest because this can result in pipe bursts, especially in older systems. Figure 4.5 shows the sixteen junctions that have pressure above 90psi for short intervals while the pumps are in operation.

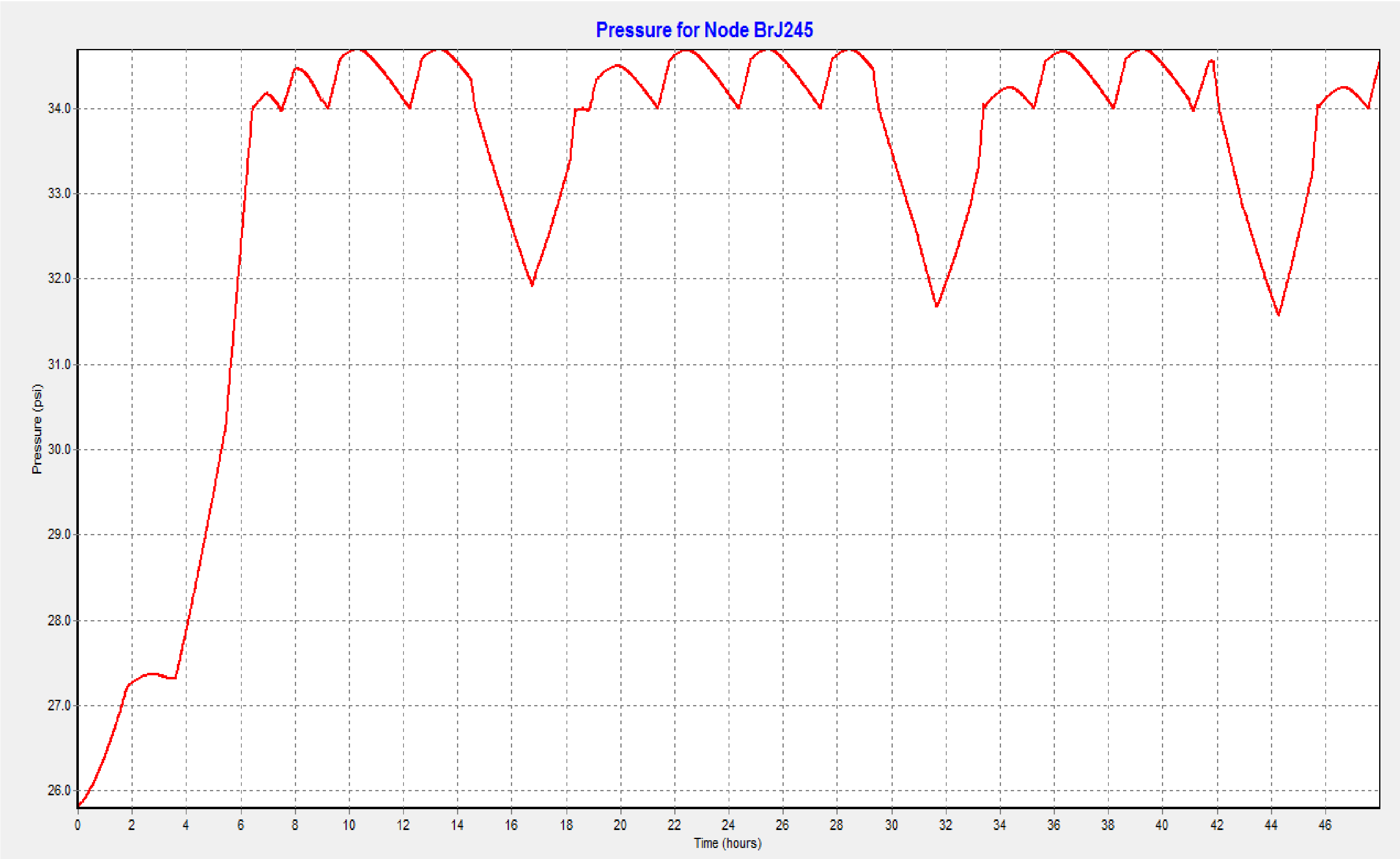

Figure 4.4: Variation of pressure with time at Node BrJ245

Node ID	Pressure psi
Junc BrJ3	108.75
Junc BrJ4	100.31
Junc BrJ7	93.32
Junc BrJ27	100.26
Junc BrJ34	107.53
Junc BrJ36	91.38
Junc BrJ38	103.34
Junc BrJ40	118.25
Junc BrJ44	104.90
Junc BrJ45	100.74
Junc BrJ46	92.02
Junc BrJ50	92.62
Junc BrJ56	102.22
Junc BrJ58	99.83
Junc BrJ240	109.15
Junc BrJ241	109.45

Figure 4.5: Nodes that experience pressure above 90psi

All except four of these junctions are located to the south of the city and some of the highest pressures are observed at dead ends within the system. The junctions experience a large variation in pressure while tank 2 is filling up and emptying. For example, the pressure at node BrJ38 is 103.7psi while the tank 2 is approaching its maximum level with the pump switched on, and the pressure drops to 60psi as the water reaches the lower operating level with the pump turned off.

It could be assumed that city center is fed from the top 29.7ft of water, while the south and the north are fed from the top 22.7ft and 12.5ft respectively since all attempts to simulate a water level below these values resulted in negative pressures at certain points within the system, prompting warning messages from the program.

The system is characterized by many long pipelines with low demand. For example, there is a 3 inch diameter, 2 mile long pipeline to the north of the city that splits into two pipelines including one that continues for another mile to feed two households. As a result, this part of the system experiences low flow velocities, especially during periods of low demand when the pumps are turned off. The highest flow velocities are observed in the pipeline that leads to tank 1 during periods of peak demand when the pump is switched on. Flow velocity in pipe Br61 for example reaches a maximum of 4.13ft/s. During this period, the velocity in pipes BrP59 and BrP28 reach their maximum values of 2.27ft/s and 1.5ft/s respectively. It should be noted that these are main pipelines. However, many of the distribution pipes experience velocities less than 0.5ft/s even during the times when the pump is running and this situation is common to more than 80 percent of all the pipes during peak demand conditions.

4.3 Location and performance of existing fire hydrants

The location of fire hydrants was obtained from the drawings of the system availed by the operator. There is a fire hydrant adjacent to the north tower to serve the northern part of the distribution and another adjacent to the south tower that serves the surrounding areas. There are also fire hydrants adjacent to the water treatment plant and node BrJ71 that serve adjacent areas. Fire hydrants in the city should be located every 500ft or at the end of each block. However, there are only 9 fire hydrants around the city of Braggs. There is a fire hydrant adjacent to the central water tower, another adjacent to the school and another seven located at major road intersections.

Simulations of the system was carried out to ensure that the these hydrants would be able to deliver the minimum flow of 250gpm for two hours under conditions of maximum hourly demand without dropping the pressure at any of the nodes to below 20psi. The nodes to which the fire hydrants were connected were assigned a separate demand pattern in order to simulate flow for 2 hours at the same time that the rest of the system experienced maximum demand. The assumption was that if the system was able to supply the fire hydrants under these conditions, then the hydrants would be able to perform satisfactorily under normal flow conditions. It was observed that the pressure at nodes shown in Table 4.2 below had fallen below 20psi after the simulation.

Table 4.2: Junctions with pressure below 20psi after 2 hours of testing fire hydrants

Node (BrJ)	**Pressure after testing of fire hydrant at junction (BrJ)**								
	201	**221**	**216**	**222**	**234**	**207**	**235**	**231**	**251**
61	19.79	19.56	19.84	19.91	19.88	19.71	19.93	19.75	19.61
242	19.07	18.54	18.44	18.34	18.40	18.52	18.33	18.46	18.46
247	19.30	18.80	18.70	18.59	18.60	18.77	18.59	18.71	18.71
250							19.97		

Nodes BrJ61, BrJ242 and BrJ247 have sustained pressure below 20psi when any the fire hydrant in the city is being tested. However, the pressure for node BrJ250 only drops below 20psi during the test at node BrJ235.

Testing of the fire hydrant at BrJ71 reveals that it is unable to produce water at 20psi under peak conditions since the pressure drops to 16psi at the onset of peak demand. A simulation of the fire demand conditions carried out for the fire hydrants located adjacent to the north and south water towers reveals that during the operation of these fire hydrants, that pressure at many of the adjacent junctions drops rapidly to below 20psi. By the end of the first 35 minutes, the junctions that are most severely affected are experiencing high negative pressures and this causes the program to generate warning messages.

4.4 Energy cost for running pumps

EPANET can be used to determine the percentage utilization of the pumps and give an approximate daily cost for operating the pumps under the described conditions. This is done by filling in the unit cost of power and the efficiency of the curve under the energy options of the browser. In turn EPANET generates a report that can be accessed by selecting the energy option under report from the main menu. The unit cost of 9.7 cents per kWh for Oklahoma and 74 percent efficiency from the pump curve were input to the program. An example of an energy report is shown in Figure 4.6

Pump	Percent Utilization	Average Efficiency	Kw-hr /Mgal	Average Kwatts	Peak Kwatts	Cost /day
BrPm1	44.91	74.00	710.00	7.32	7.73	7.66
BrPm2	3.03	74.00	636.23	7.06	7.25	0.50
BrPm3	0.00	0.00	0.00	0.00	0.00	0.00
Total Cost						8.15
Demand Charge						0.00

Figure 4.6: Energy report for 48 hour simulation

From the figure above, it is determined that under the described operating conditions, the utilzation for pumps BrPm1 and BrPm2 is approximately 45% and 3%, respectively, and that the daily cost running the pumps is 7.66 and 0.5 dollars. The daily cost of running pump BrPm2 is only 0.5 dollars because the head to which it is set to open is lower than the operating range of the system. It is only switched on at the beginning of the simulation to assist in filling up the tanks. The cost of opeating pump 3 is zero because it was treated purely as a stanby for the system. Simulations of longer periods of operation give lower daily operating costs that can be used to approximate the annual cost of running the pumps. Information obtained from the city of Braggs reveals that the cost of power for the wellhouse for the month of September 2009 was $344.77. The cost of power for the running the pumps at the well house that was obtained from the energy analysis in EPANET is 42 percent less than the actual paid for the month of September 2009. This could be a result of the controls that were assumed in the model for the operation of the pumps or a higher unit cost for energy than 9.7 cents per kWh, the average for Oklahoma, which was used for the simulation. It could also be caused by the pumps operating at a lower efficiency than is assumed in the modeling process.

4.5 Water quality simulation under existing conditions

The decay of chlorine within a distribution system is assumed to be first order whereby the concentration decreases exponentially according to equation (i) below (Boccelli et al, 2003; Hua et al, 1999; Clark et al, 1994)

$$C = C_o * e^{-kt} \quad \text{(i)}$$

Where

C is the concentration at time t

C_o is the initial concentration

k is the decay constant

t is the time elapsed

The decay constant k is considered to be the sum of k_b, the bulk decay constant of free chlorine and k_w, the wall decay constant due to the reaction of chlorine with biofilm at pipe wall or the pipe wall itself. Fang Hua et al. (1999) studied the effects of water quality parameters on the bulk decay constant of free chlorine and determined the empirical relationship between the initial concentration and the bulk decay constant at a fixed temperature shown in equation (ii) below.

$k_b = (0.018/ C_o) – 0.024$ ………………………………………………………………… (ii)

The applied chemical dose at the Braggs well house which, averages 1.0mg/l, was used as the initial concentration. This figure is a three year average of the daily dose obtained from the monthly operational reports submitted by the plant to ODEQ. This initial concentration yields a k_b value of 0.009/hr when used in the equation above.

Hallam et al. (2003) conducted experiments on different types of pipe material and determined their effect on the wall chlorine decay constant k_w. A summary of their findings is shown in Table 4.3.

Table 4.3: The effect of pipe material on the wall chlorine decay constant k_w

Pipe Material	Cast iron	Spun iron	Cement lined ductile iron	Medium density polyethylene	Polyvinyl chloride
Wall decay constant hr^{-1}	0.67	0.33	0.13	0.05	0.09

Since the distribution system at Braggs consists of PVC pipes, the k_w for PVC was selected for input the model. The values of - 0.009 and - 0.09 were input for the global bulk coefficient and global wall coefficient, respectively, under the reactions section found by following the options link from EPANET's data browser. Chlorine concentration was selected as the quality parameter to be modeled and the initial quality at the source was set as 1.2 mg/l.

ODEQ standards stipulate that the chlorine residual at the furthest point should not drop below 0.2mg/l. EPANET requires input of the initial chlorine concentration at individual nodes. This information was not available. It was assumed that the concentration at the nodes was zero and water quality simulation was carried out to show how the concentration of chlorine from water treatment plant changed within the system. Among the factor investigated was how long it would take to reach the furthest nodes and what the concentration at the nodes would be. According to the simulation, chlorine from the water treatment plant took over 75 hours to reach node BrJ245 that was previously tested for pressure and by that time, the concentration had dropped from 1.2 to 0.34mg/l.

After 75 hours of simulation, there are 27 nodes that have a chlorine concentration less than 0.2mg/l. This includes the north water tower and ten nodes located at dead ends within the system that have no demand. These points are also associated with high water age which is responsible for decay of the chlorine residual. However there are also nodes with associated demand that have chlorine residual less than 0.2mg/l. These junctions are shown in Figure 4.7 and the corresponding water age is shown in Figure 4.8

Node ID	Demand GPM	Chlorine mg/L
Junc BrJ7	0.13	0.17
Junc BrJ13	0.13	0.14
Junc BrJ18	0.13	0.15
Junc BrJ19	0.26	0.09
Junc BrJ22	0.26	0.12
Junc BrJ26	0.13	0.05
Junc BrJ44	0.38	0.14
Junc BrJ50	0.38	0.12
Junc BrJ51	0.13	0.17
Junc BrJ52	0.51	0.15
Junc BrJ54	0.38	0.12
Junc BrJ56	0.64	0.16
Junc BrJ240	0.13	0.12
Junc BrJ241	0.13	0.19
Junc BrJ246	0.13	0.00
Junc BrJ250	0.38	0.00

Figure 4.7: Nodes with non zero demand and chlorine residual less than 0.2mg/l

An important point to note is that the results above assume no residual at any of the junctions at the beginning of the simulation and no chlorine addition within the distribution system. It is known however that the city of Braggs actually has chemical boosters to raise the concentration of chlorine within the distribution system and that a total of 1.0mg/l is added to the treated water at these locations.

Node ID	Demand GPM	Age hours
Junc BrJ7	0.13	73.10
Junc BrJ13	0.13	74.28
Junc BrJ18	0.13	75.11
Junc BrJ19	0.26	76.80
Junc BrJ22	0.26	75.62
Junc BrJ26	0.13	77.12
Junc BrJ41	1.02	70.57
Junc BrJ44	0.38	73.55
Junc BrJ50	0.38	73.55
Junc BrJ51	0.13	70.57
Junc BrJ52	0.51	71.33
Junc BrJ54	0.38	72.74
Junc BrJ207	0.64	70.67
Junc BrJ240	0.13	72.97
Junc BrJ246	0.13	77.50
Junc BrJ250	0.38	77.50

Figure 4.8: Water age at nodes with demand and chlorine residual less than 0.2mg/l

As noted previously, the simulated low chlorine residual at various junctions is due to chlorine decay associated with high water age. It is observed from Figure 4.8 above that the water age at the junctions that have chlorine residual less than 0.2mg/l is above 70 hours. High water age was also observed at junctions served by long pipelines and characterized by low demand. Since water demands are only fed from the top 20 – 25 percent of water stored in the towers, the water in these tanks has a high residence time and residuals decay in the tanks. The nodes are observed to receive water of varying quality as a result of receiving fresh water from the wells while the pump is running and the tank is being filled up, and older water when the nodes are fed from storage.

From the drawings of the water distribution system, it was observed and later confirmed with the plant operator that some of the would-be dead ends in the system had recently been tied or looped together. Looping provides reliability and redundancy and is considered a good way to keep water flowing within the system and minimize water age at would-be dead ends. For example, BrJ84 and BrJ555, which were originally dead ends, had recently been connected by a 2400ft long 2 inch diameter pipe that is now recorded as BrP274 in the EPANET model of the system. Similarly, BrJ4 and BrJ5 had been connected by a 3200ft long 2 inch diameter pipe that appears as BrJ275. This information was not captured in the OWRB updates and forms part of the revisions that were made to the *.shp files that were obtained from their records.

The effects of adding these sections of pipe on pressure and water age were studied by running models of the system before and after the additions. Table 4.4 shows the water age and pressure at the respective junctions before and after alterations were made to the system. By running simulations of the system before and after the modifications, it was observed that the extra lengths of pipe had negligible effect on the pressure at the respective nodes. However, there was a marked improvement in the water age at three of the four nodes. For example, the simulated water age at BrJ5 prior to looping varies from a minimum of 57 hours when the pump was operational and increases when the junction is supplied with old water from the storage tank. However, after the looping is done, the simulated water age reduces tremendously and varies within 19 and 31 hours.

Table 4.4: Effect of looping on water distribution system

Junction	Before		After	
	Pressure	Age	Pressure	Age
BrJ55	33.0 – 34.0	57+	34.0 – 36.0	19 – 31
BrJ84	35.0 – 39.0	70 – 82	35.0 – 38.0	15 – 28
BrJ5	57.0 – 61.0	36 – 50	57.0 – 61.0	14 – 26
BrJ4	75.0 – 81.0	17 – 24	75.0 – 80.0	26 – 40

From Table 4.4 above, it is shown that BrJ84, BrJ85 and BrJ5 had decreased water age after looping. The effect of looping BrJ55 and BrJ84 resulted in tremendously decreased water age at both junctions. The water age at BrJ84 dropped from 70 – 82 hours to 15 – 38 hours after looping to junction BrJ55 as shown in Figures 4.9 and 4.10 respectively. However, joining BrJ4 and BrJ5 resulted in increased water age at BrJ4. Justification for the increase in water age at BrJ4 could be made from the observation that a significant decrease in water age at BrJ5 is achieved by looping the two junctions together and that the increased water age at BrJ4 after looping is still much lower than what it was at BrJ5 before the two junctions were looped.

It was also observed that looping the system only affects the water age of the nodes adjacent to the site of the looping and does not affect other dead ends within the system. BrJ246 and BrJ250 to the north of the city and all the three water towers are affected by extremely high water age. However, looping of BrJ55 to BrJ84 and BrJ5 to BrJ4 had no effect on the water age at these nodes.

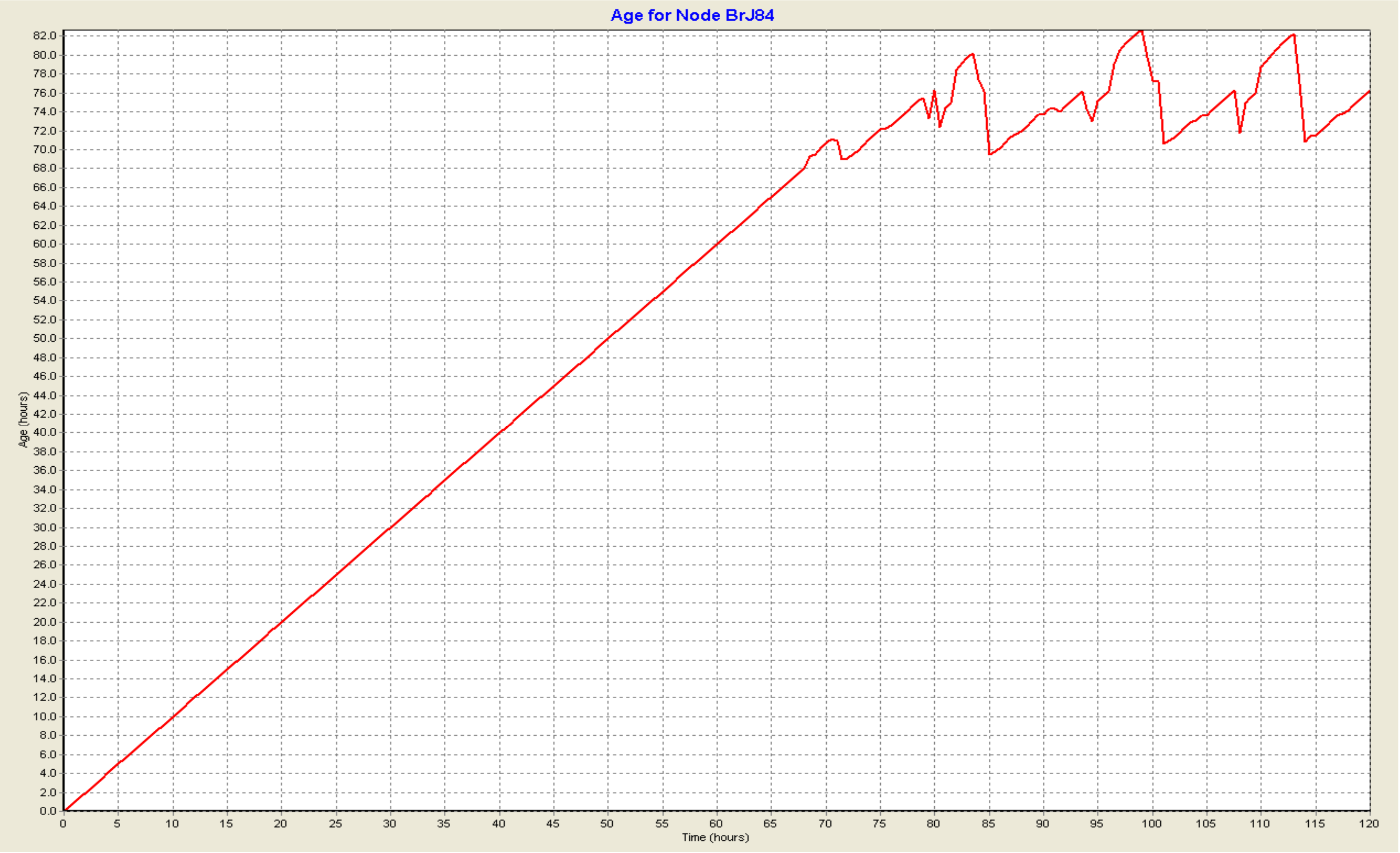

Figure 4.9: variation of water age with time at BrJ55 before looping to BrJ84

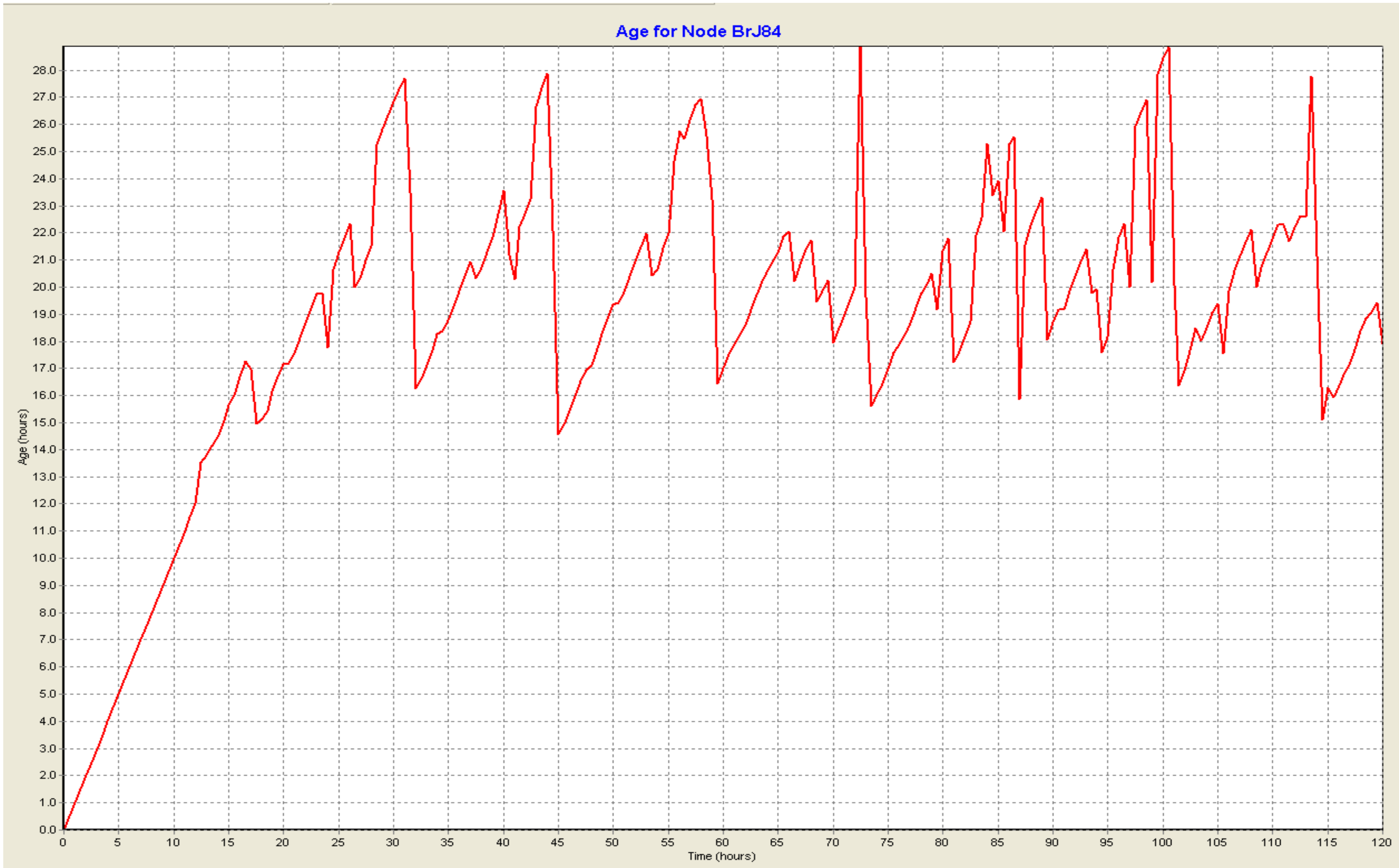

Figure 4.10: Variation of water age with time at BrJ84 after looping to BrJ55

Trial and error simulations were carried out to determine the effect that looping the existing dead ends in the system would have on water age. Looping would also be beneficial to the system as it would provide more reliability within the system for otherwise dead ends. Table 4.5 shows the links that were simulated to join dead ends within the system. The effect that introducing these pipes had on the respective nodes is shown in Table 4.6.

Table 4.5: Simulated loops to control water age

Connecting Pipe ID	Length (ft)	Diameter (in)	Junctions linked
1	3,282	2	BrJ40 and BrJ236
2	4,162	2	BrJ34 and BrJ241
3	1,267	2	BrJ61 and BrJ88
4	4,183	2	BrJ41 and BrJ50
5	2,965	2	BrJ13 and BrJ26
6	3,810	2	BrJ246 and BrJ19
7	2,639	2	BrJ26 and BrJ19
8	4,300	2	BrJ7 and BrJ3
9	6,292	2	BrJ246 and BrJ250

The length of the connecting pipes was obtained using EPANET's auto length feature and pipe diameters were selected based on the diameter of the pipes feeding the nodes to be connected. As observed from Table 4.5 above, many of the dead end nodes within the system are located several thousand feet apart. This is a typical scenario in many rural water systems and the high water age problem is compounded by low demand at these nodes. Therefore, many of the loops were found to have insignificant impact towards improving the water age at the desired locations. In some situations, the loops aggravated the water age problem, as shown in Table 4.6 below.

Table 4.6: The effect of looping on water age

Junction ID	Water age after 60hours of simulation (hrs)	
	Before looping	After looping
BrJ19	55.79	59.71
BrJ26	55.93	44.80
BrJ13	60.00	59.76
BrJ7	60.00	60.00
BrJ3	24.01	35.07
BrJ4	44.36	22.07
BrJ40	60.00	32.76
BrJ28	7.95	7.80
BrJ236	60.00	47.13
BrJ34	43.64	48.07
BrJ41	54.90	55.20
BrJ50	58.40	59.38
BrJ88	16.64	13.57
BrJ61	18.63	13.69
BrJ60	47.21	51.99
BrJ246	60.00	60.00
BrJ250	60.00	60.00
BrT1	46.20	56.03
BrT2	51.90	51.76
BrT3	59.00	59.07

The simulation of connecting BrJ88 to BrJ61 produced a negligible reduction the water age at both junctions but increased the water age in tank BrT1 by almost ten hours and the water age at an adjacent node BrJ60 increased by five hours. However, since nodes BrJ250 and BrJ246 are so far from any other nodes and have very low demand, none of the simulations of proposed loops achieved the desired goal of reducing water age at these junctions. The only significant change in water age was a 22 hour reduction achieved at BrJ40 after joining to BrJ236. The simulations revealed that looping had maximum reduction on water ager where demands were higher and the junctions to be connected were not very far apart, that is several hundred feet as opposed to several thousand feet.

The three water tanks are mounted on the ground and a high water level is required to maintain distribution system pressure when the pumps are not active. The tanks experience high water age which is observed to increase with the duration of the simulation. The water age in tank BrT1 reaches its equilibrium range of 85 – 99.5 hours after approximately 300 hours of simulation. The water age in tank BrT2 reaches its equilibrium operating range of 125 – 130 hours after approximately 375 hours of simulation. The water age in tank 3 was still rising even after 480 hours of simulation.

CHAPTER V

CONCLUSIONS

The main problem facing Braggs is high water age in the tanks and at a number of dead ends within the distribution system. The high water age in the tanks results from using nearly 75% of storage to maintain pressure at various demand nodes. Simulations of lower water levels in the tanks, in an attempt to lower water age predicted negative pressures at various nodes. However, in addition to maintaining pressure, the volume in storage is maintained 75% to provide fire fighting reserves for the region.

The high water age in the tanks is also likely to cause loss of disinfectant residual, bacterial re-growth and taste and odor problems as well as formation of disinfectant by-products. These potential problems should be further examined. Hickey (2008) advises that this situation requires the water in the tanks be recycled weekly in order to prevent excessive aging and sedimentation. He recommends that the tank be drained and a flow meter is used to measure the water drained. An equivalent amount of fresh water should be added at the top, maintaining a 90% fill during the operation. AWWA (2006) advises that water age at storage facilities can be reduced by creating an artificial demand through programmed hydrant flushing to generate flow from the tank so that it can refill with fresh water.

The water distribution system at Braggs consists of several long pipelines serving low demands at dead ends. Looping to connect dead ends is a recommended method to increase reliability of the system and to provide a constant flow of water in the pipes, which can greatly reduce water age. However, in the case of Braggs, the existing dead ends have very low demand and are so far apart that simulation of the proposed looping was not seen to have any significant impact on water quality. Modifications that were expected to improve the operation of the system revealed little impact on the current conditions such as flow velocity. In some cases, the situation was predicted to worsen. Therefore, the north end of the city requires frequent monitoring for chlorine residual, which is likely to be low due to the high water age.

Simulations of the existing conditions revealed that the pumps do not have enough head to get water to the top of Tank 3. A larger impeller cannot be installed in the existing pumps since the curves show that the pumps are run by the largest impeller that can be used with that particular model of pump. A larger pump would have to be installed to be able to fill Tank 3. However, there is low water demand at the north of the city. While filling tank 3 would improve supply pressure in the adjacent regions, it would also exacerbate the problem of high water age that is already prevalent in this area of the network.

The cost of power for the running the pumps at the well house that was obtained from the energy analysis in EPANET is 42 percent less than the actual paid for the month of September 2009. This could be a result of the controls that were assumed in the model for the operation of the pumps or a higher unit cost for energy than 9.7 cents per kWh, the average for Oklahoma, which was used for the simulation. It could also be caused by a lowering of the efficiency of the pumps.

The process of working with *.shp files, EPANET and a number of other software packages took several months before a workable model of the distribution system at Braggs was obtained for analysis. EPANET has many modeling capabilities that are not well discussed in the user's manual. The software is not very user-friendly, and there is no support network where users can share ideas and solutions to problems encountered while using the software. Many useful functions were only discovered after several months of working with the software. It would be a reasonable assumption that many of the target rural water systems do not have the resources in terms of hardware and skilled personnel to accomplish this type of assignment. Few operators are trained in this type of software and they usually have a wide range of assignments outside the daily operation of the water system.

A valuable lesson that can be derived from working with the model of Braggs is the usefulness of software in evaluating the impacts of planned/proposed modifications to water distribution systems. For example, looping in this case did not improve water age or flow velocities within the system, as was expected prior to modeling. In fact, it frequently made the current condition worse. This analysis would save resources of rural water systems by prioritizing projects that will result in the most benefit while eliminating those that will not improve the operation of the system.

REFERENCES

Brière, F. G. (1999) Drinking-water distribution, sewage, and rainfall collection Pages 106 – 107 accessed September 01, 2009 at http://books.google.com/books?id=BVLwhhEBDP0C&dq

Chochezi, V. S. (2006) Rural water systems work hard to deliver quality product; Accessed August 24, 2009 at http://www.swhydro.arizona.edu/archive/V5_N5/SWHVol5Issue5.pdf

Clement, J. et al., (2004) Predictive Models for Water Quality in Distribution Systems: Accessed September 15, 2009 at http://books.google.com/books?id=7uiu_D-QHGEC

General Accounting Office (1980), Water Issues Facing the Nation: An Overview Accessed September 30, 2009 at http://archive.gao.gov/d41t14/118309.pdf

General Accounting Office (2003) Performance and Accountability Report 2002 Accessed September 30, 2009 at http://www.gao.gov/htext/d03305sp.html

Hickey, H (FEMA) Water Supply Systems and Evaluation Methods Volume I: Water Supply System Concepts. Accessed September 30, 20009 at http://www.usfa.dhs.gov/downloads/pdf/publications/Water_Supply_Systems_Volume_I.pdf

Hickey, H (FEMA) Water Supply Systems and Evaluation Methods Volume II: Water Supply Evaluation Methods. Accessed September 30, 20009 at http://www.usfa.dhs.gov/downloads/pdf/publications/Water_Supply_Systems_Volume_I.pdf

Kirmeyer, G. J. (1999) Maintaining water quality in finished water storage facilities Accessed September 15, 2009 at http://books.google.com/books?id=0FCraz0Tr-AC

Kirmeyer, G. J. AWWA Research Foundation, 2002: Guidance manual for Monitoring distribution system water quality. Accessed August 24, 2009 at http://books.google.com/books?id=6UTyP_uWxGwC

Machell, J. et al., (2009) Improved Representation of Water Age in Distribution Networks to Inform Water Quality; Accessed September 13, 2009 at http://scitation.aip.org/getpdf/servlet/GetPDFServlet?filetype=pdf&id=JWRMD5000135000005000382000001&idtype=cvips&prog=normal

Obradović, D. and Lonsdale, P. (1998) Public water supply: data, models and operational management pages 373 – 374. Accessed September 01, 2009 at http://books.google.com/books?id=ZrzOnRyV19oC

ODEQ Public Water Supply Operation Manual; Accessed September 16, 2009 at http://www.deq.state.ok.us/Rules/631.pdf

Oklahoma Water Resources Board (2005) Water Quality Standards. Accessed August 24, 2009 at http://www.owrb.ok.gov/quality/standards/standards.php

Pawlowski, Lucjan et al, (2007) Environmental Engineering; Accessed September 16, 2009 at http://books.google.com/books?id=BsS6itxox-oC

Robinson, R. B (1976) Cost Optimization for Rural Water Systems Abstract; Accessed August 24, 2009 at http://cedb.asce.org/cgi/WWWdisplay.cgi?5012301

Rossman, L. A. (2000) EPANET 2 Users Manual; Accessed August 24, 2009 at http://www.epa.gov/nrmrl/wswrd/dw/epanet/EN2manual.PDF

Salvato, J. A. (1992) Environmental Engineering and Sanitation, Fourth Edition. Accessed August 24, 2009 at http://www.lacsd.org/civica/filebank/blobdload.asp?

Texas Commission on Environmental Quality Determining Maximum Water Age for Selecting DBP Sample Sites; Accessed September 13, 2009 at http://www.tceq.state.tx.us/files/dbp_sites.pdf_4531587.pdf

The Engineering Toolbox; Accessed September 01, 2009 at http://www.engineeringtoolbox.com/hazen-williams-water-d_797.html

The pipe flow e-Handbook; Accessed September 01, 2009 at http://www.geocities.com/SiliconValley/Hills/1829/e-Handbook/Pipeflow.htm

US Environmental Protection Agency (2001) 1999 Drinking Water Infrastructure Needs Survey Fact Sheet accessed September 30, 2009 at http://www.epa.gov/safewater/needssurvey/pdfs/2001/fs_needssurvey_2001.pdf

US Environmental Protection Agency (2009) Safe Drinking Water Act (SDWA); Accessed September 30, 2009 at http://www.epa.gov/safewater/sdwa/basicinformation.html

US Environmental Protection Agency, 1999 Drinking Water Infrastructure Needs Survey Fact Sheet. Accessed August 24, 2009 at http://www.epa.gov/ogwdw/needssurvey/pdfs/2001/fs_needssurvey_2001.pdf

US Environmental Protection Agency, 1999 Drinking Water State Revolving Fund Accessed August 24, 2009 at http://www.epa.gov/safewater/dwsrf/index.html

US Environmental Protection Agency, 2008: Distribution System Research; Accessed ugust 24, 2009 at http://www.epa.gov/NRMRL/wswrd/dw/dsr.html

US Environmental Protection Agency, 2009: Rehabilitation of Wastewater Collection and Water Distribution Systems State of Technology Review Report. Accessed on August 24, 2009 at http://www.epa.gov/nrmrl/pubs/600r09048/600r09048.pdf

US Office of Health, Safety and Security (HSS), 2009 Safe drinking water act. Accessed August 24, 2009 at http://www.hss.energy.gov/nuclearsafety/env/policy/sdwa.html

Walski, T. M. et al., Haestad Methods, 2003: Advanced water distribution modeling and management. Accessed September 01, 2009 at http://www.haestad.com/books/pdf/awdm.pdf

Water and Wastes Digest, 2008: EPA Announces Check Up Program for Small Systems; Accessed August 24, 2009 at http://www.wwdmag.com/EPA-Announces-Check-Up-Program-for-Small-Systems-NewsPiece15746

APPENDICES

Appendix A – EPANET *.inp file for Braggs RWS

shp2epa:
Imported data

[JUNCTIONS]

;ID	Elev	Demand	Pattern	
BrJ1	553.4	0.128044	P1	;
BrJ3	506.6	0.256087	P1	;
BrJ4	524	0.896305	P1	;
BrJ5	567.6	0.256087	P1	;
BrJ6	575.4	0.384131	P1	;
BrJ7	500.8	0.128044	P1	;
BrJ8	523.3	0.384131	P1	;
BrJ10	560.7	0.384131	P1	;
BrJ12	520.7	0.384131	P1	;
BrJ13	514.4	0.128044	P1	;
BrJ14	538.7	0	P1	;
BrJ16	588.3	0	P1	;
BrJ18	627.4	0.128044	P1	;
BrJ19	587.7	0.256087	P1	;
BrJ20	629.7	0	P1	;
BrJ22	601.6	0.256087	P1	;
BrJ24	556.1	0.256087	P1	;
BrJ26	545.9	0.128044	P1	;
BrJ27	543.9	1.408479	P1	;
BrJ28	585.1	0.768261	P1	;
BrJ29	603.5	0.384131	P1	;
BrJ31	600.4	0.768261	P1	;
BrJ32	579.6	0.640218	P1	;
BrJ34	524.8	0.640218	P1	;
BrJ35	602.9	0.512174	P1	;
BrJ36	562.7	0.384131	P1	;
BrJ38	535.1	0.128044	P1	;
BrJ40	500.7	0	P1	;
BrJ41	583.3	1.024348	P1	;
BrJ42	620.9	0.768261	P1	;
BrJ43	620.6	1.152392	P1	;
BrJ44	532.5	0.384131	P1	;
BrJ45	542.1	0.512174	P1	;
BrJ46	562.2	0	P1	;
BrJ48	573.2	0.128044	P1	;

BrJ50	560.8	0.384131	P1	;
BrJ51	620.9	0.128044	P1	;
BrJ52	620.8	0.512174	P1	;
BrJ54	602.4	0.384131	P1	;
BrJ55	621.4	0.128044	P1	;
BrJ56	538.5	0.640218	P1	;
BrJ58	544	0	P1	;
BrJ59	620.8	1.280435	P1	;
BrJ60	584.4	0.384131	P1	;
BrJ61	600.6	0.640218	P1	;
BrJ62	537	0.384131	P1	;
BrJ63	545.7	0.640218	P1	;
BrJ64	555.7	0.128044	P1	;
BrJ66	544.2	0	P1	;
BrJ68	559.5	0.128044	P1	;
BrJ69	579.3	0.768261	P1	;
BrJ70	560.6	0.640218	P1	;
BrJ71	568.6	1.024348	P1	;
BrJ72	568.9	0.512174	P1	;
BrJ74	560.9	0.640218	P1	;
BrJ76	563.2	0.896305	P1	;
BrJ78	543.4	0.384131	P1	;
BrJ79	550.9	0.256087	P1	;
BrJ80	562.7	0.512174	P1	;
BrJ81	533.5	0	P1	;
BrJ82	554	0.512174	P1	;
BrJ84	618.1	0.128044	P1	;
BrJ86	585.8	1.664566	P1	;
BrJ87	583.3	0.640218	P1	;
BrJ88	615.6	0.512174	P1	;
BrJ94	559.5	0	P1	;
BrJ96	561.8	1.152392	P1	;
BrJ100	546.9	1.152392	P1	;
BrJ102	636	0.128044	P1	;
BrJ9	583.1	0.512174	P1	;
BrJ201	560.8	0.384131	P1	;
BrJ202	557.8	0.128044	P1	;
BrJ203	559.9	0.256087	P1	;
BrJ204	561.3	0.384131	P1	;
BrJ205	560.8	0.128044	P1	;
BrJ206	562.6	8.130765	P2	;
BrJ207	562.6	0.640218	P1	;
BrJ208	559.7	0.640218	P1	;

BrJ209	561.1	1.280435	P1	;
BrJ210	560.9	1.280435	P1	;
BrJ211	562.3	0.640218	P1	;
BrJ212	564.8	0.128044	P1	;
BrJ214	561.5	0.640218	P1	;
BrJ215	561.9	0.896305	P1	;
BrJ216	561.9	0.384131	P1	;
BrJ217	562.8	0.384131	P1	;
BrJ218	557.1	0.384131	P1	;
BrJ219	563.8	1.152392	P1	;
BrJ220	561.7	0.768261	P1	;
BrJ221	557.7	1.024348	P1	;
BrJ222	559.7	0.768261	P1	;
BrJ223	561.9	0.384131	P1	;
BrJ224	559.5	0.640218	P1	;
BrJ226	543.7	0	P1	;
BrJ227	560.8	0.256087	P1	;
BrJ229	558.7	0.384131	P1	;
BrJ230	554.5	0.512174	P1	;
BrJ231	545.7	0.640218	P1	;
BrJ232	553.7	0.512174	P1	;
BrJ233	554.4	0.128044	P1	;
BrJ234	556.3	0.384131	P1	;
BrJ235	548.6	0.256087	P1	;
BrJ236	581.1	0	P1	;
BrJ238	611.1	0.384131	P1	;
BrJ239	601	1.152392	P1	;
BrJ240	522.5	0.128044	P1	;
BrJ241	521.8	0.128044	P1	;
BrJ242	645.8	0	P1	;
BrJ243	640.9	0	P1	;
BrJ244	638.5	0.128044	P1	;
BrJ245	620.4	0	P1	;
BrJ246	629.5	0.128044	P1	;
BrJ247	645.2	0	P1	;
BrJ248	639.4	0	P1	;
BrJ251	559	0.512174	P1	;
BrJ213	559.1	0.640218	P1	;
BrJ252	558	0.256087	P1	;
BrJ253	564.8	0.256087	P1	;
BrJ250	642	0.384131	P1	;

[RESERVOIRS]

;ID	Head	Pattern	
BrR1	538		;

[TANKS]

;ID	Elevation	InitLevel	MinLevel	MaxLevel	Diameter	MinVol	VolCurve	
BrT1	558.7	50	40	100	12	0		;
BrT2	626.2	50	40	76.5	10.5	0		;
BrT3	630	50	40	100	10.5	0		;

[PIPES]

;ID	Node1	Node2	Length	Diameter	Roughness	MinorLoss	Status	
BrP2	BrJ3	BrJ4	857.61	2	130	0	Open	;
BrP4	BrJ7	BrJ8	2547.95	2	130	0	Open	;
BrP5	BrJ8	BrJ10	1966.92	2	130	0	Open	;
BrP6	BrJ10	BrJ12	1500	2	130	0	Open	;
BrP7	BrJ13	BrJ14	1206.12	2	130	0	Open	;
BrP8	BrJ14	BrJ16	625	2	130	0	Open	;
BrP9	BrJ16	BrJ18	1427.91	2	130	0	Open	;
BrP12	BrJ22	BrJ24	650	2	130	0	Open	;
BrP13	BrJ24	BrJ26	1420	2	130	0	Open	;
BrP14	BrJ27	BrJ28	5608.78	2	130	0	Open	;
BrP16	BrJ31	BrJ32	2625	2	130	0	Open	;
BrP17	BrJ32	BrJ34	4128.92	2	130	0	Open	;
BrP18	BrJ35	BrJ36	4584.32	2	130	0	Open	;
BrP19	BrJ36	BrJ38	471.06	2	130	0	Open	;
BrP20	BrJ38	BrJ40	1809.45	2	130	0	Open	;
BrP23	BrJ45	BrJ46	2500	2	130	0	Open	;
BrP24	BrJ46	BrJ48	190.5	2	130	0	Open	;
BrP25	BrJ48	BrJ50	500	2	130	0	Open	;
BrP26	BrJ51	BrJ52	1250	2	130	0	Open	;
BrP27	BrJ52	BrJ54	800	2	130	0	Open	;
BrP29	BrJ56	BrJ58	750	2	130	0	Open	;
BrP30	BrJ59	BrJ60	1500	2	130	0	Open	;
BrP31	BrJ61	BrJ62	3102.38	2	130	0	Open	;
BrP32	BrJ63	BrJ64	750	2	130	0	Open	;
BrP33	BrJ64	BrJ66	318.7	2	130	0	Open	;
BrP34	BrJ66	BrJ68	253.3	2	130	0	Open	;
BrP35	BrJ69	BrJ70	3832.8	2	130	0	Open	;
BrP36	BrJ71	BrJ72	1508.76	2	130	0	Open	;
BrP37	BrJ72	BrJ74	1625	2	130	0	Open	;
BrP39	BrJ76	BrJ78	1220	2	130	0	Open	;
BrP40	BrJ79	BrJ80	500	2	130	0	Open	;
BrP41	BrJ81	BrJ82	1400	2	130	0	Open	;

BrP44	BrJ87	BrJ88	5250	3	130	0	Open	;
BrP49	BrJ28	BrJ3	4844.2	2	130	0	Open	;
BrP53	BrJ5	BrJ9	3875	2	130	0	Open	;
BrP54	BrJ6	BrJ9	2561.07	2	130	0	Open	;
BrP55	BrJ18	BrJ102	263.9	4	130	0	Open	;
BrP56	BrJ18	BrJ12	3116.73	4	130	0	Open	;
BrP57	BrJ12	BrJ9	4000	4	130	0	Open	;
BrP58	BrJ9	BrJ69	2625	4	130	0	Open	;
BrP59	BrJ69	BrJ27	3202.03	4	130	0	Open	;
BrP61	BrJ27	BrJ71	3400.83	6	130	0	Open	;
BrP62	BrJ71	BrJ79	1990.21	6	130	0	Open	;
BrP63	BrJ79	BrJ81	750	6	130	0	Open	;
BrP64	BrJ81	BrJ62	304.36	6	130	0	Open	;
BrP65	BrJ62	BrJ94	859.14	6	130	0	Open	;
BrP66	BrJ96	BrJ63	2200	6	130	0	Open	;
BrP67	BrJ63	BrJ94	537.53	6	130	0	Open	;
BrP68	BrJ22	BrJ102	750	2	130	0	Open	;
BrP69	BrJ102	BrJ20	750	3	130	0	Open	;
BrP70	BrJ42	BrJ51	4000	3	130	0	Open	;
BrP71	BrJ51	BrJ59	1250	3	130	0	Open	;
BrP72	BrJ43	BrJ45	5250	2	130	0	Open	;
BrP73	BrJ45	BrJ44	2250	2	130	0	Open	;
BrP74	BrJ29	BrJ35	2500	2	130	0	Open	;
BrP75	BrJ35	BrJ31	2625	2	130	0	Open	;
BrP3	BrJ55	BrJ59	2000	2	130	0	Open	;
BrP11	BrJ59	BrJ56	5250	2	130	0	Open	;
BrP22	BrJ100	BrJ1	550	2	130	0	Open	;
BrP28	BrJ27	BrJ86	3833.74	6	130	0	Open	;
BrP45	BrJ86	BrJ87	250	6	130	0	Open	;
BrP47	BrJ86	BrJ35	5125	3	130	0	Open	;
BrP48	BrJ42	BrJ43	775	2	130	0	Open	;
BrP51	BrJ43	BrJ41	4000	2	130	0	Open	;
BrP60	BrJ88	BrJ84	1500	2	130	0	Open	;
BrP80	BrT2	BrJ43	100	6	130	0	Open	;
BrP201	BrJ201	BrJ202	500.75	6	130	0	Open	;
BrP202	BrJ202	BrJ203	700	6	130	0	Open	;
BrP203	BrJ203	BrJ204	900	6	130	0	Open	;
BrP204	BrJ204	BrJ205	162.61	2	130	0	Open	;
BrP205	BrJ204	BrJ206	215.38	6	130	0	Open	;
BrP206	BrJ206	BrJ207	306.38	6	130	0	Open	;
BrP207	BrJ208	BrJ209	900	2	130	0	Open	;
BrP208	BrJ209	BrJ204	450	2	130	0	Open	;
BrP210	BrJ210	BrJ211	351.36	6	130	0	Open	;

BrP211	BrJ211	BrJ212	760.02	2	130	0	Open	;
BrP212	BrJ207	BrJ211	267.96	6	130	0	Open	;
BrP213	BrJ202	BrJ208	630	6	130	0	Open	;
BrP216	BrJ210	BrJ209	1060	2	130	0	Open	;
BrP217	BrJ210	BrJ214	720	6	130	0	Open	;
BrP218	BrJ217	BrJ214	720	6	130	0	Open	;
BrP220	BrJ221	BrJ218	448.87	6	130	0	Open	;
BrP221	BrJ221	BrJ222	434.7	6	130	0	Open	;
BrP222	BrJ215	BrJ214	720	2	130	0	Open	;
BrP223	BrJ216	BrJ215	475.92	2	130	0	Open	;
BrP224	BrJ221	BrJ215	483.88	2	130	0	Open	;
BrP225	BrJ222	BrJ223	364.25	6	130	0	Open	;
BrP226	BrJ217	BrJ219	347.92	6	130	0	Open	;
BrP227	BrJ219	BrJ220	519.62	6	130	0	Open	;
BrP228	BrJ219	BrJ215	233.02	2	130	0	Open	;
BrP229	BrJ220	BrJ216	273.47	6	130	0	Open	;
BrP230	BrJ216	BrJ223	117.74	6	130	0	Open	;
BrP231	BrJ224	BrJ223	630	2	130	0	Open	;
BrP234	BrJ227	BrJ229	1487.31	2	130	0	Open	;
BrP236	BrJ100	BrJ229	880	2	130	0	Open	;
BrP237	BrJ218	BrJ230	600	6	130	0	Open	;
BrP239	BrJ229	BrJ230	750	2	130	0	Open	;
BrP240	BrJ233	BrJ222	584.14	6	130	0	Open	;
BrP241	BrJ233	BrJ226	610	2	130	0	Open	;
BrP242	BrJ233	BrJ234	292.08	6	130	0	Open	;
BrP243	BrJ234	BrJ96	545.31	6	130	0	Open	;
BrP244	BrJ234	BrJ232	793.77	2	130	0	Open	;
BrP245	BrJ232	BrJ230	643.89	2	130	0	Open	;
BrP246	BrJ78	BrJ235	283.82	6	130	0	Open	;
BrP247	BrJ235	BrJ230	460.89	6	130	0	Open	;
BrP248	BrJ235	BrJ231	574.88	6	130	0	Open	;
BrP249	BrJ96	BrJ231	676.51	6	130	0	Open	;
BrP250	BrJ232	BrJ231	537.88	6	130	0	Open	;
BrP76	BrJ32	BrJ236	8380.94	2	130	0	Open	;
BrP251	BrJ87	BrJ239	2625	6	130	0	Open	;
BrP252	BrJ239	BrJ43	2625	6	130	0	Open	;
BrP253	BrJ239	BrJ238	1000	2	130	0	Open	;
BrP254	BrJ58	BrJ241	1173.35	2	130	0	Open	;
BrP255	BrJ241	BrJ240	757.05	2	130	0	Open	;
BrP256	BrJ19	BrJ244	800.97	3	130	0	Open	;
BrP257	BrJ244	BrJ20	920	3	130	0	Open	;
BrP258	BrJ244	BrJ243	1000	3	130	0	Open	;
BrP259	BrJ243	BrJ242	957.66	3	130	0	Open	;

BrP262	BrJ242	BrJ245	4234	3	130	0	Open	;
BrP265	BrJ248	BrJ247	2700	2	130	0	Open	;
BrP266	BrJ246	BrJ247	3274.05	2	130	0	Open	;
BrP268	BrT3	BrJ18	164.16	6	130	0	Open	;
BrP269	BrJ218	BrJ251	185.03	6	130	0	Open	;
BrP270	BrJ251	BrJ214	311.25	6	100	0	Open	;
BrP271	BrT1	BrJ251	147.73	6	100	0	Open	;
BrP219	BrJ248	BrJ245	5020.77	3	130	0	Open	;
BrP273	BrJ208	BrJ213	648.02	6	130	0	Open	;
BrP232	BrJ218	BrJ213	299.38	6	130	0	Open	;
BrP233	BrJ213	BrJ210	469.4	2	130	0	Open	;
BrP235	BrJ64	BrJ252	1250	2	130	0	Open	;
BrP272	BrJ219	BrJ253	648.27	2	130	0	Open	;
BrP274	BrJ84	BrJ55	2395	2	130	0	Open	;
BrP276	BrJ28	BrJ29	2565.52	2	130	0	Open	;
BrP275	BrJ4	BrJ5	3171.02	2	130	0	Open	;
BrP277	BrJ248	BrJ250	1872.66	3	130	0	Open	;

[PUMPS]

;ID	Node1	Node2	Parameters	
BrPm1	BrR1	BrJ27	HEAD C1	;
BrPm2	BrR1	BrJ27	HEAD C1	;
BrPm3	BrR1	BrJ27	HEAD C1	;

[VALVES]

;ID	Node1	Node2	Diameter	Type	Setting	MinorLoss

[TAGS]

LINK	BrP62	1750

[DEMANDS]

;Junction	Demand	Pattern	Category

[STATUS]

;ID	Status/Setting
BrPm1	Closed
BrPm2	Closed
BrPm3	Closed

[PATTERNS]

;ID Multipliers

;

P1	1	1	1	1	1	1

P1	1	2.25	2.25	1	1	1
P1	1	1	1	1	1	2.25
P1	2.25	1	1	1	1	1
;						
P2	0	0	0	0	0	0
P2	0	0	0	1	1	1
P2	1	1	1	1	0	0
P2	0	0	0	0	0	0
;						
Fire	0	0	0	0	0	0
Fire	0	1	1	0	0	0
Fire	0	0	0	0	0	1
Fire	1	0	0	0	0	0

[CURVES]

;ID	X-Value	Y-Value
C1	80	270
C1	90	266.25
C1	100	262.5
C1	110	255
C1	120	243.75
C1	130	232.5
C1	140	217.5
C1	150	206.25
C1	160	195
C1	170	176.25
C1	180	161.25
C1	190	138.75
C1	200	127.5
C1	210	108.75

[CONTROLS]

[RULES]
RULE 1
IF TANK BrT3 HEAD < 698.875
THEN PUMP BrPm1 STATUS IS OPEN

RULE 2
IF NODE BrJ27 PRESSURE > 80
THEN PUMP BrPm1 STATUS IS CLOSED

```
RULE 3
IF TANK BrT1 HEAD < 629.575
THEN LINK BrP61 STATUS IS OPEN

RULE 4
IF TANK BrT1 HEAD > 658.575
THEN LINK BrP61 STATUS IS CLOSED

RULE 5
IF TANK BrT1 HEAD < 628.875
THEN PUMP BrPm2 STATUS IS OPEN

RULE 6
IF TANK BrT1 HEAD > 638.575
THEN PUMP BrPm2 STATUS IS CLOSED


[ENERGY]
 Global Efficiency          75
 Global Price            0.097
 Demand Charge               0

[EMITTERS]
;Junction            Coefficient

[QUALITY]
;Node                InitQual
 BrR1                      1.2

[SOURCES]
;Node                Type        Quality     Pattern

[REACTIONS]
;Type                Pipe/Tank   Coefficient


[REACTIONS]
 Order Bulk                  1
 Order Tank                  1
 Order Wall                  1
 Global Bulk            -0.009
 Global Wall             -0.09
```

Limiting Potential	0
Roughness Correlation	0

[MIXING]

;Tank	Model

[TIMES]

Duration	60
Hydraulic Timestep	0:01
Quality Timestep	0:30
Pattern Timestep	1:00
Pattern Start	0:00
Report Timestep	0:01
Report Start	0:00
Start ClockTime	12:00 AM
Statistic	NONE

[REPORT]

Status	Full
Summary	No
Page	0

[OPTIONS]

Units	GPM
Headloss	H-W
Specific Gravity	1
Viscosity	1
Trials	40
Accuracy	0.001
CHECKFREQ	2
MAXCHECK	10
DAMPLIMIT	0
Unbalanced	Continue 10
Pattern	1
Demand Multiplier	1
Emitter Exponent	0.5
Quality	Age mg/L
Diffusivity	1
Tolerance	0.001

[COORDINATES]

;Node	X-Coord	Y-Coord

BrJ1	-95.204	35.66365
BrJ3	-95.233	35.66825
BrJ4	-95.2307	35.66833
BrJ5	-95.2221	35.66642
BrJ6	-95.2102	35.6665
BrJ7	-95.233	35.68277
BrJ8	-95.226	35.68264
BrJ10	-95.2212	35.67969
BrJ12	-95.2171	35.67935
BrJ13	-95.2247	35.68621
BrJ14	-95.2214	35.68629
BrJ16	-95.2212	35.68995
BrJ18	-95.2173	35.68976
BrJ19	-95.2155	35.6968
BrJ20	-95.2155	35.69078
BrJ22	-95.2201	35.69078
BrJ24	-95.2221	35.69209
BrJ26	-95.2227	35.69592
BrJ27	-95.2175	35.65205
BrJ28	-95.233	35.6519
BrJ29	-95.2329	35.64325
BrJ31	-95.2329	35.63185
BrJ32	-95.2419	35.63187
BrJ34	-95.2419	35.61793
BrJ35	-95.2329	35.63909
BrJ36	-95.2455	35.6391
BrJ38	-95.2456	35.63751
BrJ40	-95.2505	35.63742
BrJ41	-95.2312	35.62286
BrJ42	-95.2148	35.62282
BrJ43	-95.2175	35.62281
BrJ44	-95.2174	35.60441
BrJ45	-95.2174	35.60843
BrJ46	-95.2229	35.60844
BrJ48	-95.223	35.61014
BrJ50	-95.2263	35.6101
BrJ51	-95.2036	35.623
BrJ52	-95.2036	35.61858
BrJ54	-95.2071	35.61854
BrJ55	-95.1993	35.62789
BrJ56	-95.1995	35.60837
BrJ58	-95.1942	35.60835
BrJ59	-95.1993	35.62292

BrJ60	-95.1928	35.62288
BrJ61	-95.1994	35.64162
BrJ62	-95.1996	35.65208
BrJ63	-95.1973	35.6539
BrJ64	-95.1943	35.65386
BrJ66	-95.1945	35.65671
BrJ68	-95.1916	35.65671
BrJ69	-95.2175	35.66288
BrJ70	-95.207	35.66285
BrJ71	-95.2082	35.65201
BrJ72	-95.2082	35.6571
BrJ74	-95.2042	35.65723
BrJ76	-95.2043	35.65974
BrJ78	-95.201	35.66203
BrJ79	-95.2027	35.65206
BrJ80	-95.2028	35.6552
BrJ81	-95.2004	35.65207
BrJ82	-95.2005	35.65609
BrJ84	-95.1993	35.63039
BrJ86	-95.2175	35.63914
BrJ87	-95.2175	35.63736
BrJ88	-95.1993	35.63735
BrJ94	-95.1972	35.65209
BrJ96	-95.1973	35.66025
BrJ100	-95.202	35.6636
BrJ102	-95.2173	35.69075
BrJ9	-95.2174	35.6665
BrJ201	-95.2022	35.66745
BrJ202	-95.2012	35.66655
BrJ203	-95.2003	35.66708
BrJ204	-95.1987	35.66706
BrJ205	-95.199	35.66774
BrJ206	-95.1982	35.6669
BrJ207	-95.1975	35.66628
BrJ208	-95.2003	35.66568
BrJ209	-95.1993	35.66652
BrJ210	-95.1978	35.66512
BrJ211	-95.197	35.66572
BrJ212	-95.1951	35.66693
BrJ214	-95.1971	35.66453
BrJ215	-95.1962	35.66368
BrJ216	-95.1951	35.66271
BrJ217	-95.1963	35.66503

BrJ218	-95.1983	35.66374
BrJ219	-95.1955	35.66419
BrJ220	-95.1945	35.66322
BrJ221	-95.1973	35.66285
BrJ222	-95.1962	35.66204
BrJ223	-95.1954	35.66254
BrJ224	-95.1946	35.66191
BrJ226	-95.1936	35.65944
BrJ227	-95.2033	35.6669
BrJ229	-95.2	35.66389
BrJ230	-95.1992	35.66319
BrJ231	-95.199	35.6612
BrJ232	-95.1977	35.662
BrJ233	-95.1951	35.66075
BrJ234	-95.1959	35.66056
BrJ235	-95.2003	35.66239
BrJ236	-95.2419	35.63406
BrJ238	-95.214	35.63084
BrJ239	-95.2175	35.63078
BrJ240	-95.1893	35.60733
BrJ241	-95.191	35.60878
BrJ242	-95.2117	35.6965
BrJ243	-95.2131	35.69316
BrJ244	-95.2155	35.69292
BrJ245	-95.2009	35.69648
BrJ246	-95.2199	35.70849
BrJ247	-95.211	35.71099
BrJ248	-95.2063	35.71169
BrJ251	-95.1978	35.66403
BrJ213	-95.1988	35.66427
BrJ252	-95.1904	35.65388
BrJ253	-95.1941	35.66521
BrJ250	-95.2042	35.71744
BrR1	-95.2156	35.64541
BrT1	-95.1975	35.66372
BrT2	-95.2161	35.62477
BrT3	-95.217	35.69032

[VERTICES]

;Link	X-Coord	Y-Coord
BrP26	-95.2036	35.62291
BrP30	-95.1993	35.62292
BrP32	-95.1973	35.6539

```
 BrP36                    -95.2082   35.65201
 BrP41                    -95.2004   35.65207
 BrP44                    -95.2175   35.63736
 BrP44                    -95.1993   35.63735

[LABELS]
;X-Coord        Y-Coord        Label & Anchor Node

[BACKDROP]
                                                          -
 DIMENSIONS               -95.2137   35.65505   95.1655   35.67712
 UNITS                  Degrees
 FILE
 OFFSET                          0          0

[END]
```

VITA

Sara Namanda Senyondo

Candidate for the Degree of

Master of Science

Thesis: USING EPANET TO OPTIMIZE OPERATION OF THE RURAL WATER DISTRIBUTION SYSTEM AT BRAGGS, OKLAHOMA

Major Field: Environmental Engineering

Biographical:

Personal Data: Female; Date of birth: 11/19/1976; Citizenship: Uganda

Education:
Completed the requirements for the Master of Science in Environmental Engineering at Oklahoma State University, Stillwater, Oklahoma in December, 2009

Bachelor of Science in Mechanical Engineering, Makerere University, Kampala, Uganda, October 2000

Experience:
Graduate Research Assistant May 2008 – May 2009
Oklahoma State University, Stillwater

Assistant Mechanical Engineer September 2002 – November 2007
Multi-Konsults Ltd, Kampala, Uganda

Sales Executive/Management Trainee October 2000 – August 2002
Toyota Uganda Ltd, Kampala, Uganda

Professional Memberships:
Chi Epsilon
Uganda Institution of Professional Engineers

Name: Sara Namanda Senyondo Date of Degree: December, 2009

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: USING EPANET TO OPTIMIZE OPERATION OF THE RURAL WATER DISTRIBUTION SYSTEM AT BRAGGS, OKLAHOMA

Pages in Study: 86 Candidate for the Degree of Master of Science

Major Field: Environmental Engineering

Scope and Method of Study: This study was carried out in order to assess the performance of the drinking water distribution system at Braggs, Oklahoma using hydraulic simulation software and to address any improvements required in order to improve quality of service to their customers. The study also aimed to establish how common problems experienced by rural water systems can be detected and addressed using hydraulic simulation software. The main focus of the study was water quality, pressure at different points within the distribution system, fire flow requirements, pipe materials and age of the distribution system.

The study was conducted as part of a larger project funded by the Oklahoma Water Resources Research Institute (OWRRI) that aimed to provide an easily accessible and cost effective way for rural water systems in Oklahoma to evaluate the performance of their distribution networks and plan for improvements. The city of Braggs which is located in eastern Oklahoma, 56 miles south east of Tulsa, was selected because it fits the description of a Rural Water System (RWS). The water distribution system serves 1030 people in the city and surrounding areas.

Water utilities seek to provide customers with a reliable and continuous supply of high quality water while minimizing costs. Due to their nature, distribution networks contain points of vulnerability where contamination can occur. Rural water systems are often small and struggle to meet even the basic requirements of the safe drinking water act (SDWA) since they often collect insufficient revenues to keep their networks operating properly. Distribution system modeling helps to identify points where contamination is likely to occur, identifies required upgrades in advance, and forms a basis for decision support by evaluating possible alternatives.

Findings and Conclusions: The study at Braggs predicted low pressures at certain points with the system, identified areas with insufficient fire flows, and where disinfectant residuals were likely to fall below the ODEQ minimum requirements; as well problems with water age that could not be addressed by conventional methods like looping. These problems however, were predicted in the country and not within the city limits. The study generally revealed the usefulness of hydraulic modeling as a decision support tool.

ADVISER'S APPROVAL: Dr. Dee Ann Sanders