EFFECTS OF SYSTEM SIZE ON THE SUSTAINABILITY OF WATER SUPPLY INFRASTRUCTURE

by

NATHAN ANDREW HESTER

(Under the Direction of Ke Li)

ABSTRACT

In water supply systems, capital costs are often the primary factor in design selection. However, due to increasing energy prices and a growing concern for the environment, increased attention to operational energy consumption has become a central issue. This research evaluates the economic and environmental impacts of water supply systems of varying size. This study found that total life cycle costs are dominated by drinking water treatment and the economies of scale provide that the unit life cycle cost is decreased from \$0.82/m^3 to \$0.42/m^3 from the smallest to largest systems. However, unit distribution system capital costs were found to increase with system size, and unit pumping-energy requirements also increased from 0.15 kWh/m^3 to 0.24 kWh/m^3. While the total unit cost of water was found to decrease with system size, this trend may be reversed through a more complete representation of the distribution system.

INDEX WORDS: Drinking water, Distribution systems, Sustainable infrastructure, Energy efficiency, Life cycle assessment, Life cycle cost

EFFECTS OF SYSTEM SIZE ON THE SUSTAINABILITY OF WATER SUPPLY INFRASTRUCTURE

by

NATHAN ANDREW HESTER

B.S., Southern Polytechnic State University, 2009

A Thesis Submitted to the Graduate Faculty of The University of Georgia in Partial Fulfillment of the Requirements for the Degree

MASTER OF ENVIRONMENTAL ENGINEERING

ATHENS, GEORGIA

2012

EFFECTS OF SYSTEM SIZE ON THE SUSTAINABILITY OF WATER SUPPLY INFRASTRUCTURE

by

NATHAN ANDREW HESTER

Major Professor:	Ke Li
Committee:	William Tollner Todd Rasmussen

Electronic Version Approved:

Maureen Grasso
Dean of the Graduate School
The University of Georgia
May 2012

DEDICATION

I dedicate this research to my loving wife Jana Davis Hester and the rest of my family whose encouragement and unwavering support made this research possible.

ACKNOWLEDGEMENTS

I would like to extend my appreciation to Mr. Chris Adams, Mr. Sam Mark, Ms. Melody Melton, and Mr. Harold Jones of Jacobs Engineering - Athens, GA office and Mr. Glenn Coleman, Mr. William Cottrell, Mr. Gary Duck, Mr. Edward Focht, and Mr. Jeff Knight of the Athens-Clarke County, GA Department of Public Utilities for their input in the design of my project. I am also very appreciative for the support of my graduate committee: Professors Bill Tollner, Todd Rasmussen, and major professor Dr. Ke Li.

TABLE OF CONTENTS

Page

CHAPTER 1

INTRODUCTION

1.1 Problem Overview

The current practice in the urban water supply industry is to pump water from the raw water source to a centralized Drinking Water Treatment Plant (DWTP) where raw water is treated to drinking water standards and distributed to the community for consumption. A wastewater collection system then sends sewage to a Water Treatment Plant (WWTP) where it is treated for discharge into the environment. As part of the Resilient and Sustainable Infrastructure (RESIN) research effort, this work focuses on sustainable water treatment and distribution (collectively referred to as water supply). The primary goal of any potable water supply system is to reliably provide safe water to its customers at a reasonable cost. In addition, global concern for the environment has influenced many water authorities to become increasingly concerned with energy consumption and sustainability. In the past, low energy prices enabled utility planners to primarily consider the initial capital cost of water infrastructure while largely disregarding operation and maintenance costs. As energy prices increased, so did the attention given to water system energy-efficiency and the resulting operational costs. In the last few decades, concern for the environment has brought issues like Ozone depletion and natural resource depletion to the forefront. Therefore, careful consideration

of capital costs, operational costs, and environmental impacts are needed to meet our future water needs with sustainable solutions.

1.2 Energy, Infrastructure, and the Environment

Growing concern for the environment has prompted an increased interest in energy-efficiency. At the request of Congress, the U.S. Department of Energy (U.S. DOE) reported on the interrelationship between water and energy production (DOE 2006). The primary finding of the report is that energy is required to supply water and water is required to generate energy. This concept is shown in Figure 1.1. More specifically, energy generation accounts for roughly 40% of all freshwater withdrawals in the U.S., and roughly 4% of the electricity produced is used for the treatment and transportation of water. The authors also note that often the majority of this energy is devoted to pumping. As a result, energy consumption is a critical factor in the design of water systems, and a great deal of research has been conducted on the subject.

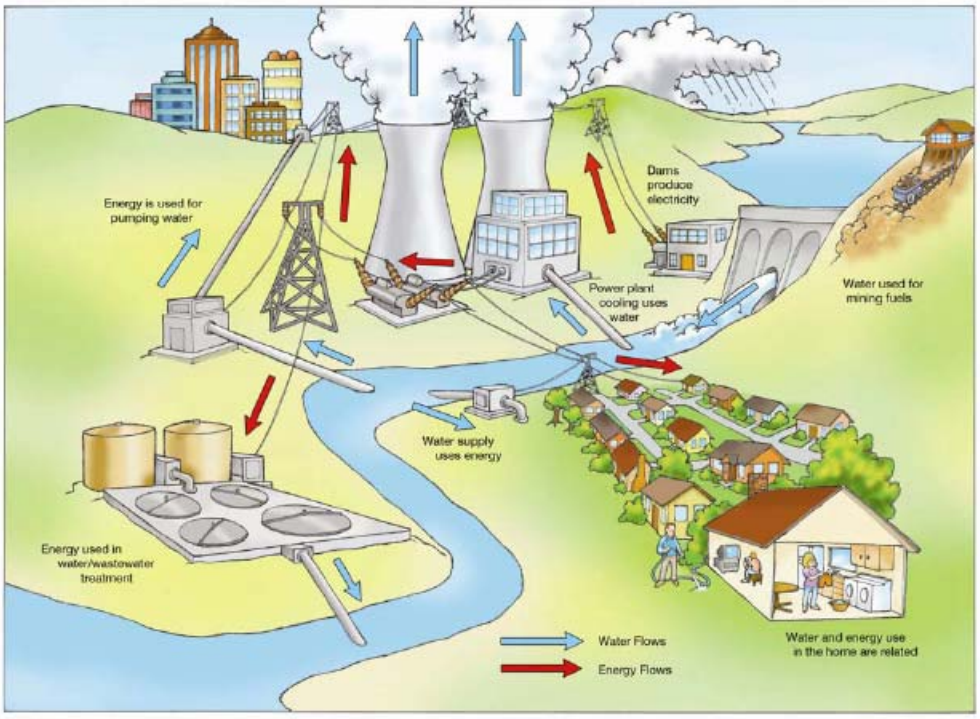

Figure 1.1: Energy/Water Interactions (DOE 2006)

1.3 Life-Cycle Assessment (LCA) in the Water Industry

In the last 20 years, Life-Cycle Assessment (LCA) has become a valuable tool for understanding the interrelationship between water, energy, and the environment. LCA is used evaluate the total environmental impacts of a product or system. Decision makers can utilize LCA to aid in the selection of the most sustainable design options and to target the most polluting steps (Friedrich 2002). However, most LCAs are site specific and potential comparisons from one LCA to another are limited by significant variations in scope definition, chosen LCA methodology, and local characteristics, etc (Vince, Aoustin et al. 2008). Despite these limitations, valuable information has been derived from LCAs of the water industry from around the world.

The scope of LCAs in the water industry are typically limited in three ways; by limiting the level of detail being modeled, by limiting assessment to the operational phase of the lifecycle, and limiting the number of water system components studied. It is obvious that the accuracy of a LCA is dependent on the level of detail being considered. However, time and finance may not permit that every nut and bolt be considered. This is especially true when studying a large subject such as a water supply system. Therefore, establishing and reporting limitations regarding design detail are important in any LCA.

Another way to limit the scope of a LCA in the water industry is to consider only the operational phase of the lifecycle. LCA is commonly described as "cradle to grave"; however, many LCAs in the water industry only consider the operational phase of the lifecycle and neglect any impacts from the construction and decommissioning phases. This assumption is not always practical. For an item like a chair, there are no operational impacts, and this assumption would render the LCA worthless. However, Freidrich, Pillay et al. (2007) reviewed LCAs in the water treatment industry and found that the operational phase of the lifecycle represents 95% of the total environmental impacts. The construction and decommissioning phase of the lifecycle represented 4% and 1% of the environmental impacts. The overall conclusion of this work is that energy consumption can be used as an environmental indicator when comparing design options in the absence of an LCA. In fact, a common theme in all of the reviewed water industry LCAs is that municipal energy consumption is a significant contributor to environmental impacts. According to Boulos and Bros (2010), the energy consumed in the U.S. water and wastewater industry accounts for 34 million tons of carbon dioxide equivalent greenhouse gas emissions each year. On the other hand, Herz and Lipkow (2002) found

the Life Cycle Impacts resulting from the construction of water and sewer mains can be significant.

Finally, the scope of a LCA in the water industry can be limited regarding the number of water system components studied. In fact many are limited to the water treatment process and few include distribution. Only one of the LCAs studied considered the entire water cycle, i.e. water treatment, distribution, consumer use, collection, and wastewater treatment. Arpke and Hutzler (2006) reviewed relevant literature to establish energy consumption ranges for each phase of the entire water lifecycle including consumer use (see Table 1.1). The authors found that the building use phase of the life cycle dominates environmental impacts. The majority of these impacts are due to the energy intensive nature of water heating. This fact illustrates the need for more efficient water heating technologies (solar methods, tankless water heaters), and changes in human behavior.

Table 1.1: Water System Energy-Use Ranges Arpke and Hutzler (2006)

	Energy consumption		Percentage of total energy consumption	
	(kWh/m^3)	(kWh/m^3)	(%)	(%)
Life Cycle Phase	Min	Max	Min	Max
Water Treatment and Distribution	0.11	0.66	1.66%	1.76%
Consumer-Use	6.3	36	95.17%	96.18%
Wastewater Collection and Treatment	0.21	0.77	3.17%	2.06%
Total	6.62	37.43	100.00%	100.00%

In a study of Sydney, Australia's water system Lundie, Peters et al. (2004) found that water distribution and wastewater treatment had more environmental effects than water treatment. Freidrich, Pillay et al. (2007) reviewed LCA's in the water industry and found similar results adding that pumping energy for potable water distribution was the largest contributor to environmental impacts. Table 1.2 displays the distribution pumping-energy requirements found in literature.

Table 1.2: Comparison of Distribution Pumping-Energy

Author(s)	Average value kWh/m^3	Minimum kWh/m^3	Maximum kWh/m^3
Vince, Aoustin et al. 2008	0.25	0.2	0.8
Arpke and Hutzler 2006	0.315	n/a	n/a
Elliot, Zeier et al. 2003	0.399	n/a	n/a
Wilkinson 2000	0.178	n/a	n/a

In conclusion, innovative solutions are needed to limit the environmental impacts of energy consumption associated with water supply. When the use phase of the lifecycle is excluded, potable water distribution was found to be the most environmentally taxing phase of the lifecycle. These findings highlight the need for optimizing distribution system efficiency.

1.4 Drinking Water Regulations in the U.S.

In the U.S., drinking water regulations promulgated by the U.S. EPA play a significant role in determining the fiscal and environmental costs of water supply. This section provides a brief overview of relevant drinking water regulations.

In 1974, the Safe Drinking Water Act (SDWA) required that the EPA establish National Primary Drinking Water Regulations (NPDWRs) to include Maximum Contaminant

Levels (MCL) and MCL goals (MCLGs) for a variety of contaminants. The SDWA was amended in 1986 amendments to give the EPA more regulatory power and require regulations be set for 83 emerging contaminants. The SDWA saw further revision in 1996 that required customer confidence reports with statistics regarding the quality of the water in the system, utility operators to be licensed, and called for additional protection from microbial risks and disinfection byproducts.

The Clean Water Act (CWA) of 1977 is primarily concerned with water pollution but overall it is significant to drinking water in the form of source water protection. The CWA prohibits any wastewater discharge without a permit from the federal government (National Pollutant Discharge Elimination System (NPDES) permits).

The Surface Water Treatment Rule (SWTR) was created in 1989 and modified several times to include the Interim Enhanced SWTR (IESWTR) of 1998, the Long Term 1 Enhanced SWTR (LT1ESWTR) of 2002, and the Long Term 2 Enhanced SWTR (LT2ESWTR) of 2002(EPA 2005). Initially, the SWTR established National Primary Drinking Water Regulations (NPDWRs) for surface water systems and groundwater systems under the direct influence of surface water with the primary goal of reducing the risk associated with Giardia, viruses and Legionella. In addition, the SWTR required a disinfectant residual, and established additional treatment requirements for some systems. The IESWTR 0f 1998, only applies to surface water systems and groundwater systems under the direct influence of surface water that serve more than 10,000 customers. It included new monitoring requirements and new removal requirements for Cryptosporidium. The LT1SWTR of 2002, extended the IESWTR to systems serving less than 10,000 customers. The Long Term 2 Enhanced SWTR (LT2ESWTR) required

a two-year Cryptosporidium survey and systems with 0.075 oocysts per liter or more are required to provide additional treatment.

The Information Collection Rule (ICR) was promulgated in 1996 and required system monitoring for microorganisms and DBPs. The information collected is used to appropriately improve the SWTR.

The Stage 1 Disinfection Byproducts Rule (DBPR) was promulgated in 1998 and established rules regarding the monitoring and reporting of DBPs (EPA 1998). In addition, the stage one DBPR established maximum residual disinfectant levels (MRDLs) and MRDL goals (MRDLGs) for chlorine, chlorinamines, and chlorine dioxide as well as total trihalomethanes (TTHMs), five haloacetic acids (HAA5), bromate, and chlorite. The stage 2 DBPR retained the MCLs for TTHM and HAA5 but changed the method of sample collection. Under the stage 1 DBPR, TTHM and HAA5 monitoring was based on an annual average of all data points in the system for the entire year. The stage 2 DBPR requires systems to perform an initial analysis of the distribution system to determine which areas are at greatest risk and annual averages for each sampling location will be provided.

The common trend in drinking water regulations is that water quality requirements have become increasingly stringent with time. Often times, compliance with these regulations require additional treatment and energy inputs.

1.5 Disinfection and Disinfection Byproducts

Drinking water disinfection with chlorine began in the early 1900s and is considered by many to be one of the most important discoveries of the century. Disinfection is the process of removing pathogens from water. Chlorine is effective in

the inactivation of pathogens and the most cost effective disinfectant currently available. As a result, chlorine is the most commonly used disinfectant around the world.

Today, current federal policy requires a minimum disinfectant residual in drinking water. However, in other parts of the world (particularly Europe) a disinfectant residual isn't always required. Trussell (1999) explored the differences in potable water distribution with, and without a disinfection residual. Chlorination is the most common method of disinfection in the world and the disinfection process can be divided into primary disinfection and residual maintenance. Primary disinfection is performed at the water plant, and is intended to eliminate pathogens in the water. UV and other alternative methods of disinfection can provide primary disinfection without a residual and the resulting DBPs (Shaw 2002). Residual maintenance is provided in-line and has several purposes; overcome possible contamination, prevent opportunistic pathogens, prevent coliform regrowth, prevent biofilm formation, and stabilize water quality in general.

Providing drinking water without a residual requires careful distribution system design and maintenance. In order to provide safe water without a residual, contamination of the distribution system must be avoided through; maintaining an adequate minimum pressure in the system, proper backflow prevention, better system maintenance, more frequent replacement of old water mains, more frequent system flushing, conservative distribution system design, and better initial water treatment. The most common cause of distribution system contamination is caused by inadequate system pressure and back-siphoning (Trussell 1999). The author concluded that while it is possible to provide safe drinking water without a residual, it is not recommended and not allowed in the U.S.

From the 1970s to the present, the formation of Disinfection ByProducts (DBPs) has been of great concern in the water industry (Trussell 1999; Sadiq and Rodriguez 2004). As chlorine reacts with Natural Organic Matter (NOM) in the water supply disinfection occurs and DBPs are formed. The formation of DBPs is primarily dependant on; chlorine dose, Hydraulic Retention Time (HRT), NOM concentration, pH level, and temperature (EPA 2001). Sadiq and Rodriguez (2004) reviewed existing models to predict DBP formation and developed a second-order model based on pH, temperature, initial chlorine dose, Dissolved Organic Carbon (DOC, a substitute measure of NOM), bromide concentration, and alkalinity. The authors concluded that higher pH, temperature, and NOM concentration expedite the formation of DBPs.

Chlorine demand is directly related to DBP formation and the models presented to estimate it are generally based on the same factors as the DBP prediction model presented by Sadiq and Rodriguez (2004). The only additional factor considered for estimating chlorine demand is UltraViolet spectral Absorbance (UVA, a method to classify the specific structure of NOM) (Clark and Sivaganesan 2002). A detailed review of chlorine chemistry and DBP formation is provided by the U.S. EPA (EPA 2001). DBPs are still being discovered and linked to potential human health effects. We now know that thousands of DBPs can be formed when chlorine reacts with Natural Organic Matter (NOM) in water. As a result, drinking water regulations have become increasingly stringent regarding DBP control. According to the literature, there are four methods to reduce DBPs;

1. Switch to alternative disinfectants (Diamant 1996; Adams and Sutherland 2000; EPA 2001; Das 2002; EPA 2005; Hijnen and Medema 2005; Kerwick, Reddy et al. 2005; Bichai and Barbeau 2006; Casson and Hunter 2008),
2. Remove additional NOM through advance water treatment technologies (Hong and Elimelech 1997; EPA 2001; Bolto, Dixon et al. 2002; Chiang, Chang et al. 2002; Van der Bruggen and Vandecasteele 2003; EPA 2005; Less 2011)
3. Remove DBPs once they are formed in the distribution system (EPA 2001; Chae and Wang 2002; EPA 2005; Norton and Weber 2006; Arnal, García-Fayos et al. 2009)
4. Limit the extreme water age (size) of the distribution system (Weber Jr and Norton 2008).

Options 1 – 3 have been studied in great detail over the last thirty years and the U.S. EPA currently suggests options 1 and 2 for DBPR compliance (EPA 2001). Limiting DBP formation with alternative disinfectants can be an effective option. Potential alternatives to conventional chlorination include; chlorinamines, chlorine dioxide, ozone, hydrogen peroxide, iodine species, bromine species, permanganate, ionizing radiation, silver, ultraviolet irradiation (UV), copper, titanium photocatalysis, photodynamic disinfection, high voltage pulsed electric fields, and ultrasonication. The most commonly used disinfection alternatives are chlorinamines, chlorine dioxide, UV and ozone. Alternative disinfectants generally produce less DBPs than chlorine; however DBPs are not the only factor to be considered in disinfectant selection. Kerwick, Reddy et al. (2005) developed a method to evaluate the performance of a disinfectant in the Netherlands. The authors found that the most critical criteria in disinfection selection are

inactivation efficiency, DBP formation, toxicity, aesthetic water quality, cost, scalability, and residual maintenance.

The relationship between various classifications of NOM and DBP formation has been an area of extensive study in recent years. NOM can be difficult to remove and the performance of various treatment technologies regarding NOM removal has been studied in great detail. The enhanced removal of DBP precursors (NOM) through advanced treatment is the EPA's recommendation for DBPR compliance. The intent of the DBPR is that 90% of all water systems in the U.S. should be able to comply with little or no modification to the treatment process. The remaining 10% will be required to limit DBP formation with alternative disinfectants and/or advanced treatment techniques like enhanced coagulation, filtration with Granular Activated Carbon (GAC), and Nanofiltration (NF) (EPA 2001; EPA 2005). Obviously, other treatment technologies are able to remove NOM, but the EPA's recommended methods seem to be the most commonly considered.

Removal of DBPs after they are formed must occur at multiple points in a distribution system and is considered impractical and difficult to implement. This method involves the installation of small DWTPs to remove DBPs where DBPR compliance is an issue (Weber Jr 2002; Weber Jr 2004; Norton and Weber 2006). These DWTPs can be large enough to serve several customers (point of use treatment) or as small enough to provide safe water for a single household. This method is commonly used in developing countries where poor water quality is common and additional treatment is required for consumption (Arnal, García-Fayos et al. 2009; Peter-Varbanets, Zurbrugg et al. 2009).

According to the EPA, it is easier to remove NOM than DBPs. Advanced treatment methods, such as membrane filtration and Granular Activated Carbon (GAC) are capable of treating almost any water source into exceptional drinking water. However, these advanced water treatment methods are generally more expensive than conventional treatment due to increased energy, chemical, and capital requirements.

As the size of a water treatment system increases, economies of scale reduce the treatment cost per unit of water produced. On the other hand, water quality in the distribution system degrades with age as chlorine reacts with Natural Organic Matter (NOM) to form harmful Disinfection By-Products (DBPs). DBPs are regulated under the U.S. EPA's Disinfection By-Products Rule (DBPR), and as a result, their formation can limit the acceptable extreme water age in the distribution system. In some cases energy-intensive advanced treatment technologies may be required to reduce NOM concentrations and extend the acceptable extreme water age (EPA 2001). Therefore, DBP formation can affect the fiscal and environmental performance of water treatment and potentially limit the acceptable size of a distribution system. Under conventional thought, limiting the size of a water supply system is considered impractical and has received limited attention in the research community. However, a growing interest in distributed water supply systems seems to validate this concept.

1.6 Water Supply System Components

The primary objective of a water supply system is to reliably provide adequate supplies of safe drinking water to the customer. The major components of a water supply system are briefly discussed in the following sections.

1.6.1 Raw Water Source

Every water supply system begins with a source. Sources are typically classified as groundwater, brackish groundwater, rivers and streams, reservoirs, and ocean water. The quality of the water has a considerable impact on system design, DWTP process selection, and water system performance.

1.6.2 Raw Water Delivery System

Raw water is collected from the source and unless the local topography enables gravity flow, intake pumping is required to deliver raw water from the source to the DWTP.

1.6.3 Drinking Water Treatment Plant

Drinking Water Treatment Plants (DWTPs) vary considerably in size and function. Modern DWTPs are custom designed for the existing local conditions and an almost limitless combination of processes is possible in DWTP design. Site-specific conditions such as raw water quality and site footprint dictate each design. A conventional DWTP includes chemical addition (typical chemicals include Potassium Permanganate, Alum or Ferric Salts, polymers, Floride, and Chlorine), flocculation, sedimentation, filtration, and disinfection. Although every DWTP is different, the majority of DWTPs in the US are a variation of conventional treatment, and advanced water treatment methods, like membrane filtration and ultraviolet irradiation, are used less frequently but gaining popularity.

1.6.4 Pumps

Pumps are an essential component of every water treatment system as they are required to move raw and finished water through the system. Adequate head is required

to overcome friction losses and elevation changes. The most common type of pump in water supply is the centrifugal pump. In a centrifugal pump, a motor spins an impeller to increase the head in the system.

1.6.5 Junctions

Junctions are connections between pipes. In hydraulic modeling software water demand is exerted at the nodes only.

1.6.6 Pipes

Pipes are enclosed conduits for transferring flow and come in a variety of sizes and materials. Pipes are constructed of many materials including Polyvinyl Chloride (PVC), cast iron, Ductile Iron Pipe (DIP), cement, and spiral-welded steel. Variations in diameter, material, and velocity all affect the friction losses incurred.

1.6.7 Storage Tanks

Storage tanks are designed to hold excess water in times of low flows and discharge water to the system during high flows. They are used to provide emergency storage for fire protection and additional flow during peak hours or system failures. Tanks can be constructed of various materials and can be designed to operate under pressure or as “floating” tanks where the pressure in the system dictates the tank level.

1.6.8 Valves

A valve is a component that can be open closed to some degree. Valves can be classified as isolation valves, directional valves, attitude valves, air release valves, and control valves. Isolation valves are used to completely stop flows through a pipe and are commonly used to isolate sections of pipe for maintenance. Types of Isolation valves include gate valves, butterfly valves, globe valves, and plug valves. Directional valves,

or check valves, allow flows to occur in only one direction. Altitude valves are commonly used in tanks and are opened and closed based on the upper and lower limits of the tank. Air release valves are typically located at highpoints to release trapped air from the line. Finally, control valves (pressure reducing valves, pressure sustaining valves, flow control valves, and throttle control valves) are used to regulate flows and pressures.

1.6.9 Service Connections

All water supply systems are designed to provide water to a customer. The service connection marks the end of the distribution system and the beginning of a privately owned water line.

1.6.10 Controls

As water supply systems become more sophisticated, a complex system of sensors and controls are commonly used to monitor and control system components. These Supervisory Control and Data Collection (SCADA) systems can be used to remotely view real-time data and control elements of the system.

1.7 Factors Affecting the Performance of Water Supply Systems

Optimization is a methodology to find the most effective or functional solution, and typically one or more factors are minimized or maximized. Traditionally, water system design selection has been based on economics, safety, and reliability with little attention given to environmental impacts. Although economic factors still dominate design selection, recent research regarding energy-use in the water industry emphasizes the need for energy optimization to conserve natural resources and protect the environment. Due to the energy-intensive nature of water supply, there is opportunity for

significant energy savings through innovative engineering. This research effort will focus on optimizing the fiscal and environmental costs of water supply systems.

Optimizing the fiscal and environmental performance of a water system is a site-specific problem that is dependent on a number of factors including; system size, urban form, topography, population density, socioeconomic factors, location of the water source, raw water quality, drinking water regulations, and many others. These factors are explored in the following sections.

1.7.1 System size

In the U.S., most water supply systems are small ones. In fact, the smallest 63% of water systems serve only 2.4% of the population. On the other hand, the largest 5.4% of water systems supply water to 78.5% of the population (EPA 1998). It is evident that water systems vary greatly in size and that larger water supply systems provide the majority of drinking water in the U.S.

As cities grow in size, the water supply systems serving them increase in size. The conventional thought is that economies of scale enable increases in DWTP size to decrease the unit cost of water treatment (Clark, Sivaganesan et al. 2002; EPA 2005). However, recent research regarding water quality (EPA 1998) and distribution pumping costs (Rao 2011) seem to suggest there is a size limit to the effectiveness of centralized water supply. In an evaluation of decentralized water supply systems, Chung, Lansey et al. (2008) report that economies of scale afford a large DWTP will be the most cost effective option in many situations. However, the authors also propose that in situations with sparsely distributed populations or significant topographical change, increased pumping and piping costs may provide that multiple decentralized DWTPs would be

more cost effective. Arple and Hutzler (2006) also report that the energy requirements of a system are dependent on size, and found that an average value of 0.37 kWh/m^3 is required for water supply with roughly 0.315 kWh/m^3 of this energy being required for distribution pumping alone. The affect of system size and population density on unit pumping costs are shown in Figure 1.2 (Rao 2011).

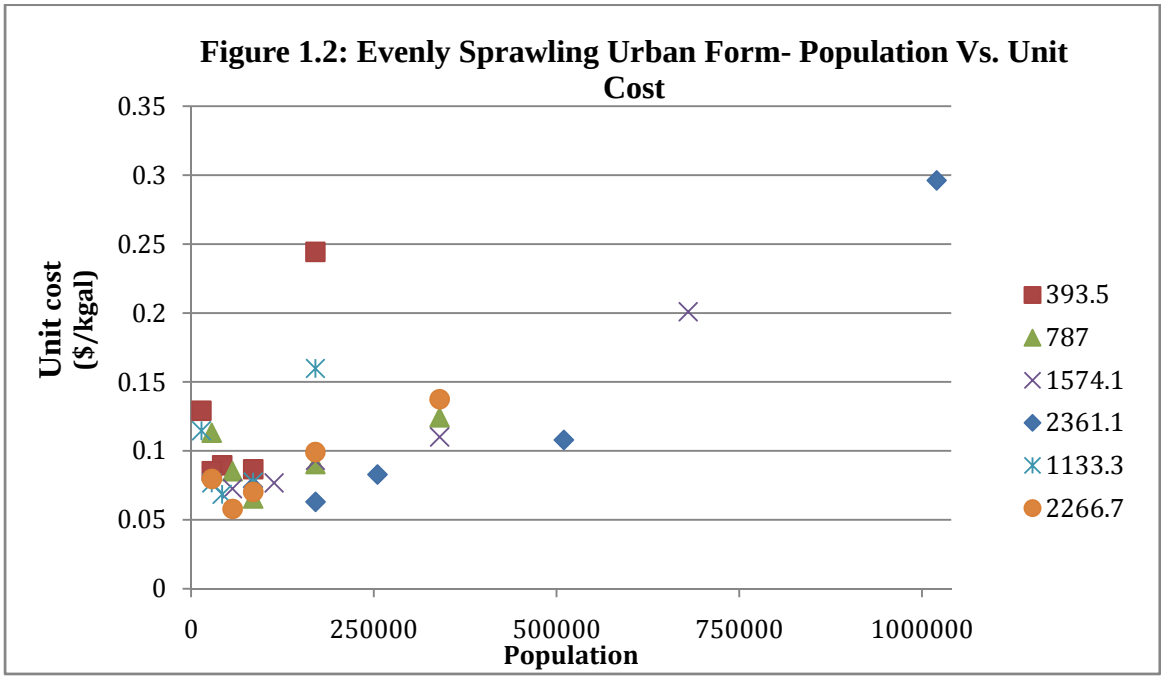

Figure 1.2: Evenly Sprawling Urban Form- Population Vs. Unit Cost (Rao 2011)

Limited research has been conducted on the effect of system size on pumping energy. Rao (2011) modeled five hypothetical distribution systems with a population range of 54,440 to 680,000 capita and a population density of 1574.1 capita/km^2, and found that distribution pumping costs increased from $0.01918 to $0.05309 $/kgal. However, model assumptions regarding the level of service delivered limit the validity of the results.

1.7.2 Topology, Topography, and Socioeconomics

Topology is an important factor in the design of a water supply system. For example, the location of raw water supplies can be the most important factor regarding the fiscal and environmental costs of water supply. In situations where there is a limited supply of water, energy intensive alternatives like water transfer from another basin and saltwater desalination can potentially increase fiscal costs and energy-use 300%-500% when compared to conventional treatment (Raluy, Serra et al. 2005). Similar results were found in Stokes and Hovrath (2006).

Rao (2011) found that urban form and population density can have a considerable effect on the energy efficiency of a distribution system. The authors modeled theoretical cities of various urban forms and found that the population density, urban form, and number of location of water sources all had a significant effect on pumping costs.

Local socioeconomic factors affect water supply systems in a number of ways. These factors are too numerous to comprehensively discuss and are considered beyond the scope of this study.

1.7.3 Water quality

One of the most important considerations in water supply is raw water quality. It has been shown that even waters of the poorest quality can be treated to drinking water standards with advanced water treatment methods, however the advanced treatment methods with these capabilities are often more energy and cost intensive than conventional treatment. Therefore, raw water quality directly affects the fiscal and environmental costs of water supply.

As previously discussed, local drinking water regulations can affect the level of treatment required at the DWTP and DBP formation can limit the effective size of the distribution system. Therefore, local drinking water regulations are an important consideration in optimizing water supply systems.

1.7.4 Water Supply System Design Considerations

The design of a distribution system can greatly affect the performance. Pipe sizing, pipe material, pipe routing, storage tank sizing, pump selection, and pump scheduling directly influence the energy needs of the system. In practical application, water supply systems are designed based on current consumer needs, forecasted growth patterns, and site-specific conditions such as topography and topology. For example, it is common practice for distribution piping to be installed in the right-of-way of existing roads, thus pipe routing is directly related to existing transportation infrastructure.

1.8 Objectives

The first objective is to determine a relationship between water system size and unit pumping energy. The second objective is to investigate the impact of system size on the overall economic and environmental costs of potable water supply systems.

1.9 Summary

The literature has shown that there is a potential conflict between the unit cost of water treatment and the unit cost of distribution. In other words, there may be a point where cost savings derived from the economies of scale in large DWTPs are negated by the increased operational costs of the distribution system. This factor indicates that there may be an optimal size for water supply systems, and in a large metropolitan area, two

smaller systems may be more efficient than one large system. This concept would be of particular importance in the field of distributed (or decentralized) water supply.

Therefore, this research will explore the validity of Rao (2011) and attempt to find the optimal size of a water supply system. Hydraulic modeling software will be used to evaluate the fiscal and environmental performance of water supply systems of various sizes.

CHAPTER 2

WATER SUPPLY SYSTEM DESIGN

2.1 Project ScopeThe primary focus of this work is to evaluate the effect of system size on the economic performance and energy efficiency of a water supply system. The site specific nature of these systems provides that no two systems are exactly the same. In order to study this topic on a comparable basis, modeling techniques will be used in conjunction with a number of generalizations and assumptions about the operating parameters of each system. For each modeled scenario the scope will include a DWTP, primary pumps, booster pumps, storage tanks and distribution piping.

Optimizing the fiscal and environmental performance of a water supply system is a site-specific problem that is dependent on a number of factors including; system size, urban form, topography, population density, socioeconomic factors, location of the water source, raw water quality, drinking water regulations, and many others. The design parameters that must be considered for this project are summarized in the following sections.

2.2 Selection of Hydraulic Modeling Technique

The methodology of this research project was based on that of Weber and Norton (2008). Many of the assumptions chosen by the authors are duplicated in this work to allow for comparison of results, however, the hydraulic modeling technique presented by the authors is not able to simulate energy consumption and energy-use in the distribution system will be a major component of this research. Additional inspiration was derived

from Rao (2011) who studied the effect of system size on pumping energy and found that pumping energy requirements were increased in larger water systems. To build on these research efforts, hydraulic modeling software will be used to provide detailed results. EPANET modeling software was selected for its ability to model WSS system components and accurately estimate pumping energy requirements (Rossman 2000). EPANET was created by the U.S. EPA to simulate drinking water distribution systems and aid in the understanding of DBP formation. The mechanics of the model have been validated by others and the software is readily available as a free download.

EPANET modeling software will be utilized to complete the stated objectives. EPANET was created by the U.S. EPA to simulate drinking water distribution systems (Rossman 2000), and was selected for application in the current research because it is capable of estimating the two critical factors needed for analysis; pumping energy and water age. EPANET is capable of calculating friction losses through pipes by three different methods: Hazen-Williams, Darcy-Weisbach, and Chezy-Manning. The Hazen-Williams method will be used in this work as it is most commonly used method in the US.

2.3 Site Conditions

In practical application, a variety of site conditions must be considered when selecting the appropriate water system design. The factors considered in this study are detail in the following sections.

2.3.1 Design Population, Population Density, and Urban Form

The population of a community directly affects the amount of water consumed. There is a great deal of variation in the population characteristics of cities across the U.S.

As the focus of this study is on large water supply systems, design populations were selected for six scenarios doubling in size from 48,600 to 1,555,200 capita. The design populations are listed along with corresponding cities of roughly the same population in Table 2.1: Design Population Comparison.

Table 2.1: Design Population Comparison

Case #	Design Population (capita)	Comparable City	2010 Population of Comparable City (capita)
1	48,600	Valdosta, GA	54,518
2	97,200	Macon, GA	91,351
3	194,400	Augusta, GA	195,844
4	388,800	Oakland, CA	390,724
5	777,600	Austin, TX	790,390
6	1,555,200	Philadelphia, PA	1,526,006

In addition to the population of a city, factors such as urban form, amount and type of industry, local climate and socioeconomic factors all affect the water demand and will be simplified and represented in the selection of per capita demand. The effect of urban form, local zoning, and amount of industry are beyond the scope of this study, and in this research effort the population is considered evenly distributed without consideration of zoning and the presence of industry. The population density in this work is held constant at 1,350 capita/km^2. This assumption was used in Weber and Norton (2008) and represents the average urban population density from 2005 US Census data.

2.3.2 Topography

Although elevation change can be a significant factor in WSS performance the effect of topography is beyond the scope of this study and the elevation at any point in the system is assumed to be 100 ft above sea level.

2.4 Design Parameters

The design of each scenario in this study is based on several design assumptions and parameters. Each of these parameters is discussed in the following sections.

2.4.1 Demand

The local water demand in the US varies greatly from one region to another due to site-specific conditions like climate, land-use type, availability of raw water, water quality, system pressure, watering restrictions, and socioeconomic factors like standard of living and type of residences. Shammas and Wang (2011) provide a typical demand values for total water demand in the US ranging from 227.1 – 1324.9 L/capita/day (60 - 350 gal/capita/day). A base value of 757.1 L/capita/day (200 gal/capita/day) was used to estimate the average daily demand, or normal flow, of each scenario. This value includes domestic, commercial, industrial, and public consumption, as well as unaccounted for water. The total daily demand is displayed for each scenario in Table 2.2, and is calculated by,

$$TDD = P * D$$

Where, TDD = Total Daily demand (L/d)

P = Population (capita)

D = Per capita demand (L/capita/day)

Table 2.2: Average Daily Flows			
Case #	Population (capita)	Total Daily Flow L/capita/day	Total Daily Flow MGD
1	48,600	36,795,060	9.72
2	97,200	73,590,120	19.44
3	194,400	147,180,240	38.88
4	388,800	294,360,480	77.76
5	777,600	588,720,960	155.52
6	1,555,200	1,177,441,920	311.05

2.4.2 Demand Pattern

The total daily demand figure displayed in Table 2.2 is very useful for planning purposes such as determination of DWTP capacity but does not adequately represent the variation in demand through the course of the day. Under normal operating conditions, consumer demand fluctuates from low flows at night to high flows during the day. To account for hourly demand variation, demand factors for each hour are used in conjunction with normal flow values to create a 24 hour demand pattern. The largest demand factor is known as the peaking factor and represents the peak daily flow in the system. The hourly demand is calculated by,

$Q * D_f = Q_h$

Where, Q = Normal flow (L/capita/day)

D_f = Demand factor (none)

Q_h = Hourly flow (L/capita/day)

Demand patterns appropriately ration the system flows throughout the day, and are a critical to any modeling effort of water supply systems. In this study, a simplified demand pattern is used for all scenarios and is represented in Table 2.3 and Figure 2.1.

Table 2.3: Allocation of Daily Demand

Time	Demand Factor	Time	Demand Factor
12:00:00 AM	0.4	12:00:00 PM	1.4
1:00:00 AM	0.4	1:00:00 PM	1.4
2:00:00 AM	0.4	2:00:00 PM	1.4
3:00:00 AM	0.4	3:00:00 PM	1.6
4:00:00 AM	0.4	4:00:00 PM	1.6
5:00:00 AM	1	5:00:00 PM	1.8
6:00:00 AM	1	6:00:00 PM	1.6
7:00:00 AM	1.3	7:00:00 PM	1
8:00:00 AM	1.3	8:00:00 PM	0.4
9:00:00 AM	1.3	9:00:00 PM	0.4
10:00:00 AM	1.3	10:00:00 PM	0.4
11:00:00 AM	1.4	11:00:00 PM	0.4

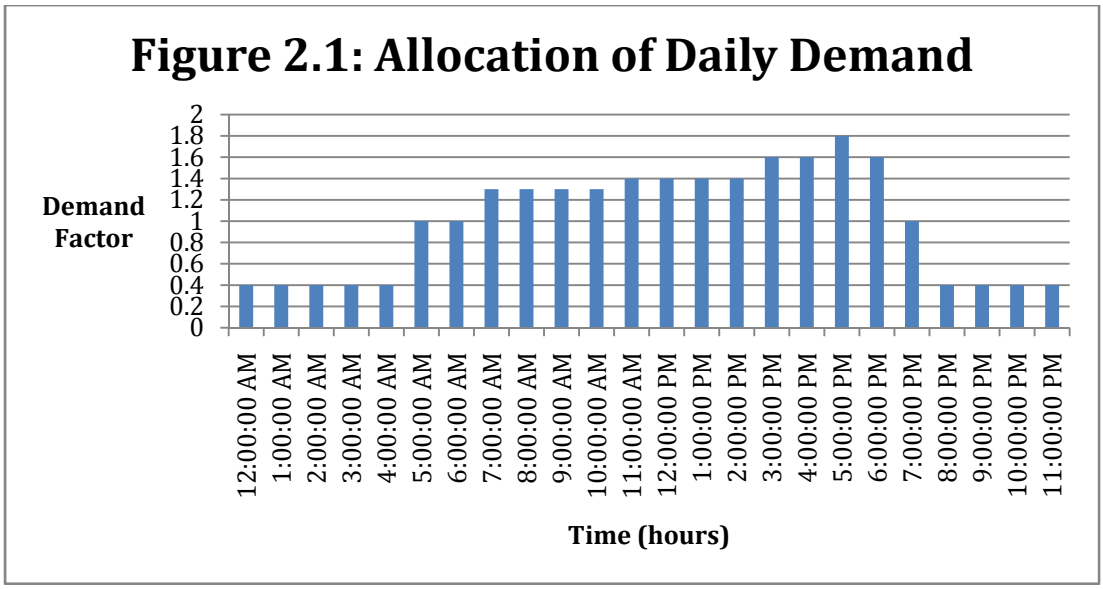

Figure 2.1: Allocation of Daily Demand

2.4.3 Fire Flow

Several methods of fire demand estimation were developed by the Insurance Services Office Inc., the Illionois Institute of Technology Research Institute, and Iowa State University (Shammas and Wang 2011). These methods are often based on building type (multifamily residential, commercial, etc) and the distance between buildings. In practical design these equations should be used to assure adequate fire flow is achieved, however this high level of detail isn't required for this project. The National Board of Fire Underwriters provides a population-based method of fire flow estimation suitable for use in this project. Design fire flows can be found in Table 2.4 and are calculated by,

$Q_f = 3860.7*(P_k)^{0.5}[1-0.01*(P_k)^{0.5}]$

Where, Q_f = Flow (L/min)

P_k = Population in thousands

Table 2.4: Fire Flow Requirements						
Case #	Population in thousands	Normal flow	Fire flow	Fire flow	Fire flow	Ratio of normal flow/ fire flow
	(capita / 1000)	(L/min)	(L/min)	(L/s)	(MGD)	
1	48.6	25551.5	25038.1	417.3	9.5	1.0
2	97.2	51103.1	34310.1	571.8	13.1	1.5
3	194.4	102206.1	46323.5	772.1	17.6	2.2
4	388.8	204412.2	61114.9	1018.6	23.2	3.3
5	777.6	408824.5	77636.7	1293.9	29.5	5.3
6	1555.2	817648.9	92209.1	1536.8	35.1	8.9

It should be noted that this method of estimating fire flows provides a larger percentage of fire flow for smaller systems than larger ones (i.e. the ratio of normal flow / fire flow is much lower in the smaller scenarios, see Table 2.4). This concept is of particular importance regarding system design as fire flows are used for pipe sizing and will be discussed in the following chapters.

2.4.4 Flow Summary

In addition to Normal flows and fire flows, it is important to know the maximum flow that could occur in the system for design purposes (i.e. peak flow + fire flow). The flow data for each scenario are summarized in Table 2.5. In each scenario a peaking factor of 1.8 is used to determine the peak flow by,

$Q * P_f = Q_p$

Where, Q = Normal flow (L/capita/day)

P_f = Peaking factor (none)

Q_p = Peak flow (L/capita/day)

Table 2.5: Flow Summary								
Case #	Normal flow (NF)	Normal flow (NF)	Peak flow (PF)	Peak flow (PF)	Fire flow (FF)	Fire flow (FF)	PF + FF	PF + FF
	(Lps)	(MGD)	(Lps)	(MGD)	(Lps)	(MGD)	(Lps)	(MGD)
1	425.9	9.7	766.5	17.5	417.3	9.5	1,183.8	27.0
2	851.7	19.4	1,533.1	35.0	571.8	13.1	2,104.9	48.0
3	1,703.4	38.9	3,066.2	70.0	772.1	17.6	3,838.2	87.6
4	3,406.9	77.8	6,132.4	140.0	1,018.6	23.2	7,150.9	163.2
5	6,813.7	155.5	12,264.7	279.9	1,293.9	29.5	13,558.7	309.5
6	13,627.5	311.0	24,529.5	559.9	1,536.8	35.1	26,066.3	594.9

2.4.5 Raw Water Source

Every water supply system begins with a source. Sources are typically classified as Groundwater, Brackish Groundwater, rivers and streams, reservoirs, and ocean water. Obviously, the quality of the water will have a considerable impact on system design, DWTP process selection, and water system performance. For all scenarios in this study, the source and DWTP are located 2 km (1.243 miles) to the north of the center of the distribution system.

2.4.6 Raw Water Delivery System

Intake pumping is required to deliver raw water from the source to the DWTP. In each scenario, raw water delivery is not considered in the hydraulic analysis as the source is in the same location and at the same elevation as the DWTP.

2.4.7 Water Treatment Plant

For each scenario, a conventional DWTP is modeled to include chemical addition (typical chemicals include Potassium Permanganate, Alum or Ferric Salts, Polymers, and

Chlorine), flocculation, sedimentation, filtration, and disinfection. Although every DWTP is different, the majority of DWTPs in the US are a variation of conventional treatment. The performance of this DWTP is not the focus of this study and it is included for cost estimation purposes only.

2.4.8 Pumps

Pumps are an essential component of every water treatment system as they are required to move raw and finished water through the system. Adequate head is required to overcome friction losses and elevation changes. The most common type of pump in water supply is the centrifugal pump and this is the type of pump modeled in this study. Primary pumps will be used to distribute water from the DWTP to the system, and booster pumps will be strategically placed to counteract low pressures during high flows.

2.4.9 Junctions

Junctions (nodes) are connections between pipes. In hydraulic modeling software nodes are used to model connections between pipes, pumps, and tanks. In addition, water demand is exerted at the nodes.

2.4.10 Pipes

Pipes are enclosed conduits for transferring flow and come in a variety of sizes and materials. Pipes are constructed of many materials including PolyVinyl Chloride (PVC), cast iron, Ductile Iron Pipe (DIP), cement, and spiral-welded steel. Variations in diameter, material, and velocity all affect the friction losses incurred through a pipe. With a plethora of pipe materials available, it was decided that DIP would be used for water lines with diameters from 8” to 24” and cement pipe would be used for diameters from 30” to 144”. The range of pipe diameters used is presented in Table 2.6.

Table 2.6: Pipe Diameters			
(inches)	(mm)	(inches)	(mm)
8	203	66	1676
10	254	72	1829
12	305	78	1981
14	356	84	2134
16	406	90	2286
18	457	96	2438
20	508	102	2591
24	610	108	2743
30	762	114	2896
36	914	120	3048
42	1067	126	3200
48	1219	132	3353
54	1372	138	3505
60	1524	144	3658

In this study, mainlines will be modeled to deliver water to nodes in each section of the distribution system where water is withdrawn for consumer use. Accordingly, service lines used to deliver water to each customer are not included in the hydraulic analysis. EPANET will estimate friction losses with the Hazen-Williams method,

$h_L = (C_f * L * Q^{1.852}) / (C^{1.852} * D^{4.87})$

Where, h_L = Head loss due to friction (m)

C_f = Unit conversion factor (10.7 for S.I. units)

L = Pipe length (m)

Q = Flow rate (m^3/s)

C = Hazen-Williams friction factor

D = Pipe diameter (m)

For simplicity, all of the pipes in this study are considered to have a C factor of 140. In addition, minor losses at valves, tees, bends, and other appurtenances are ignored. This simplification would not be suitable for real-world design application; however this assumption provides an adequate amount of accuracy for this research.

2.4.11 Storage Tanks

After careful consideration, it was decided that each scenario should have the same number of tanks per area rather than providing one tank of increasingly large size. Therefore, all of the tanks in this study are the same size and height while the number of tanks doubles from one scenario to the next. Further, the bottom elevation of each tank is at 127 m above sea level (the elevation of all other components is 100 m above sea level). The total storage volume required is set to approximately 25% of normal daily flow for each scenario. Every tank in this study is connected to the distribution system with a 50 meter long pipe. See Table 2.7 and Table 2.8 for additional details.

Table 2.7: Tank Specifications

Item	Value	Units
Tank Shape	Cylindrical	(shape)
Tank Height	13	(m)
Tank Diameter	30	(m)
Tank Volume	9189.2	(m^3)
Elevation of Tank Base	127	(m)
Tank Drawdown	33	(%)
Minimum Level	8.7	(m)
Maximum Level	13	(m)

Table 2.8: Number of Tanks Required						
Case #	Normal Flow	Required Tank Volume (25% of Normal Flow)	Volume of Each Tank	Number of Tanks	Total Storage Provided	% of Required Storage Provided
	(m^3/d)	(m^3)	(m^3)	(#)	(m^3)	(%)
1	36794	9198.6	9189.2	1	9189.2	99.90%
2	73588	18397.1	9189.2	2	18378.3	99.90%
3	147177	36794.2	9189.2	4	36756.6	99.90%
4	294354	73588.4	9189.2	8	73513.3	99.90%
5	588707	147176.8	9189.2	16	147026.5	99.90%
6	1177414	294353.6	9189.2	32	294053.1	99.90%

2.4.12 Valves

Control valves are used in each tank to restrict outward flow when the tank level is less than 8.7 m and prevent inward flow when the tank level reaches 13 m. In practical application, isolation valves would be placed throughout the system to allow for maintenance and repairs.

2.4.13 Controls

As water supply systems become more sophisticated, a complex system of sensors and controls are commonly used to monitor and control various system components. In this study, controls are used to control primary pumps, booster pumps, and tank levels.

2.5 Water Supply System Design

The water supply system components described in the previous sections are combined to form the six scenarios of this study. The methodology for system design is described in the following sections.

2.5.1 Distribution System Layout

The main focus of this work is to study the effect of system size on energy requirements and fiscal costs. Accordingly, a great deal of effort was made to design each scenario according to the same design standards to ensure the validity of the results. It was decided that each scenario would be a designed as a square grid of equally spaced nodes. The assumptions that the node spacing, daily demand, and population density remain constant throughout each scenario allows for each node to have the same demand. Therefore, each node in the system serves an area 2 km x 2 km (1.24 miles x 1.24 miles) with a normal daily flow of 47.3 Lps (624.27 gpm). Some difficulty was encountered when laying out each scenario as it proved impossible to simultaneously hold node spacing and demand constant while providing a perfectly square grid. It was decided that the first scenario would be exactly square and each subsequent scenario would have additional nodes added to the perimeter of the system or removed at random to account for additional/excess flows. As each scenario is exactly twice the size of the last, this afforded that scenarios 1, 3, and 5 would be perfect squares while scenarios 2, 4, and 6 require adjustment. See Table 2.9 for details.

Table 2.9: Distribution System Layout

Case #	Total Population (capita)	Number of Nodes in Grid (#)	Number of Additional Nodes Needed (#)	Total Number of Nodes (#)
1	48,600	9	0	9
2	97,200	16	2	18
3	194,400	36	0	36
4	388,800	64	9	73
5	777,600	144	0	144
6	1,555,200	289	-1	288

2.5.2 Pipe Routing

Each scenario was designed as a square grid of equally spaced nodes with a complete grid of pipes connecting them. Each scenario was designed to be square in shape in the event that a perfect square was not applicable, nodes are either added or removed (See Figure 2.2, Figure 2.3, Figure 2.4, Figure 2.5, Figure 2.6, and Figure 2.7).

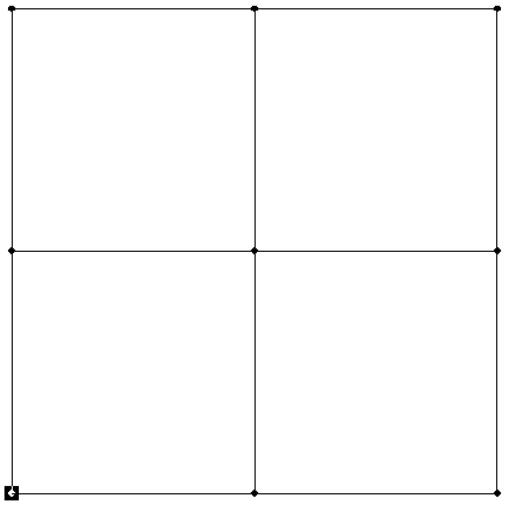

Figure 2.2: Initial Pipe Layout for Scenario #1

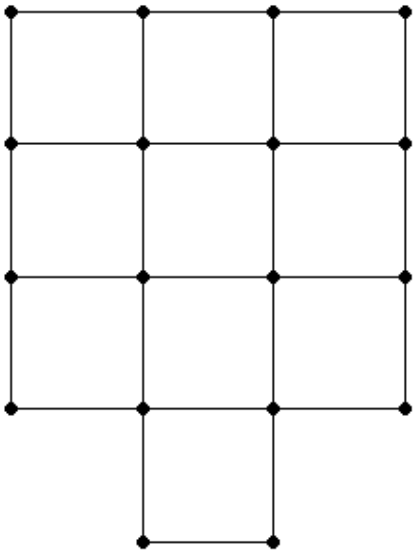

Figure 2.3: Initial Pipe Layout for Scenario #2

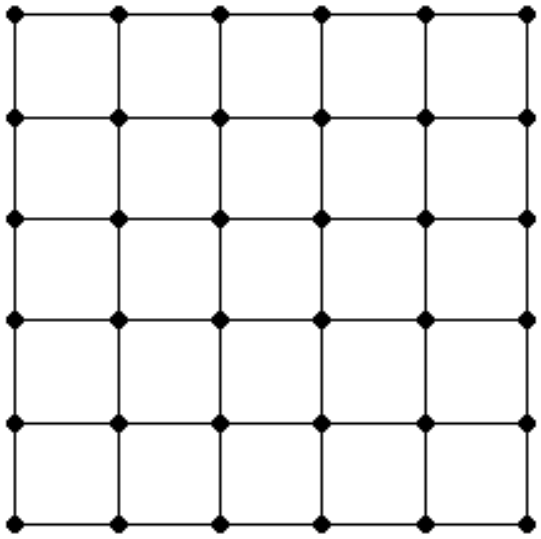

Figure 2.4: Initial Pipe Layout for Scenario #3

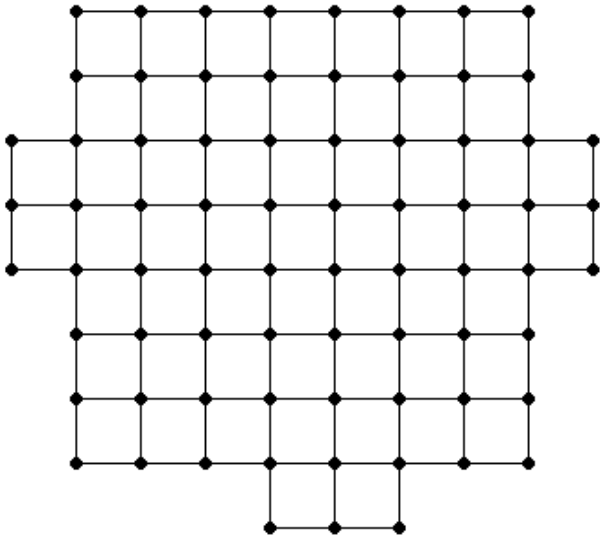

Figure 2.5: Initial Pipe Layout for Scenario #4

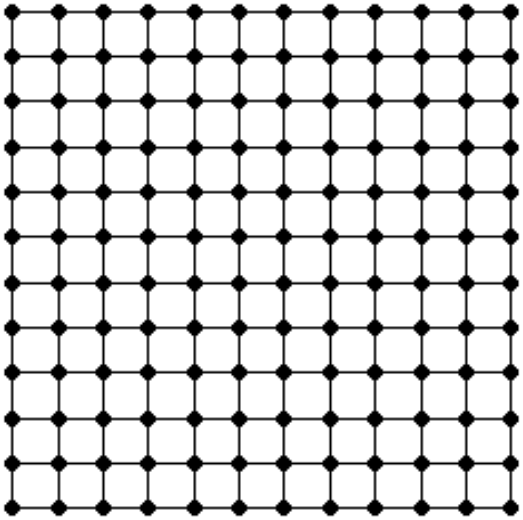

Figure 2.6: Initial Pipe Layout for Scenario #5

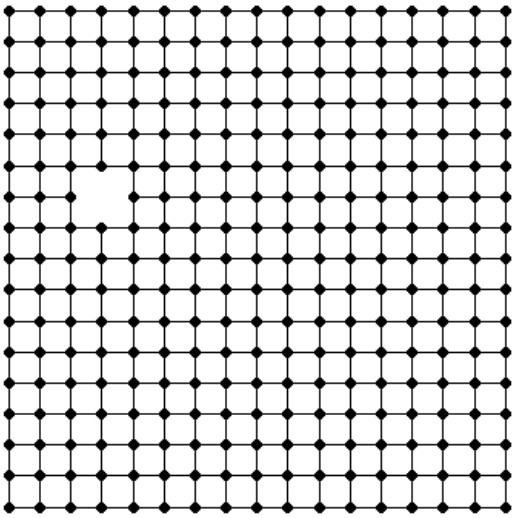

Figure 2.7: Initial Pipe Layout for Scenario #6

While redundancy is important in any distribution system, it was decided that the level of redundancy provided in these complete grids was unnecessary and impractical. To remedy this issue in an impartial manner, a genetic algorithm was used to find a

layout that minimized the number of pipes while assuring that each node could be supplied from at least two directions. Simply put, a genetic algorithm is a stochastic search procedure based on the evolutionary mechanisms of natural selection and genetics. The solutions to an optimization problem are encoded as chromosomes, which evolves toward better solution by mating and mutation according to the guidance of a fitness function. A large number of successful applications of genetic algorithms in a variety of engineering optimization domains have proved its ability of identifying valid solutions.

In this study, a mainline was selected to remain untouched, and the genetic algorithm was run for each scenario to determine the pipe routing used. The modified pipe routing for each scenario are presented in Figure 2.8, Figure 2.9, Figure 2.10, Figure 2.11, Figure 2.12, and Figure 2.13. In each figure the bold line represents the mainline selected to remain untouched by the algorithm.

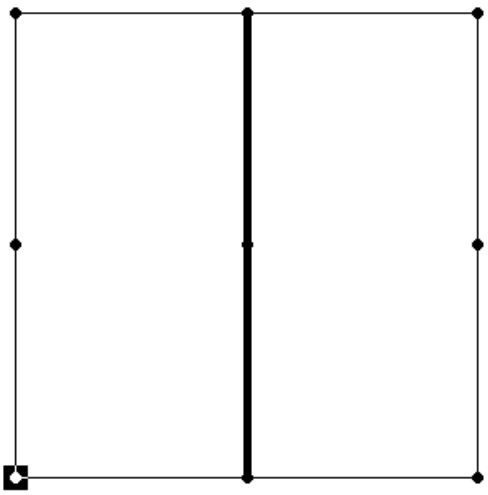

Figure 2.8: Modified Pipe Layout for Scenario #1

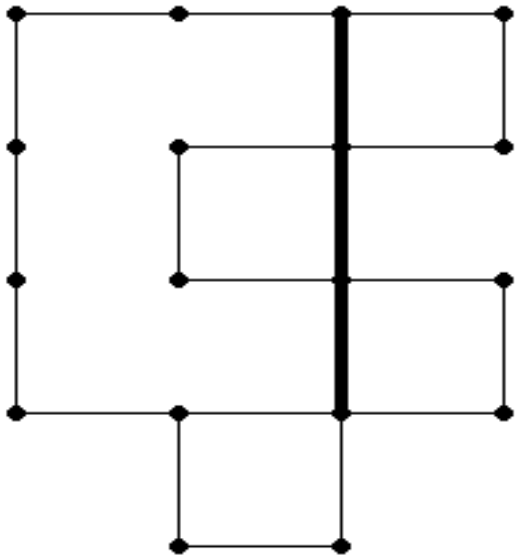

Figure 2.9: Modified Pipe Layout for Scenario #2

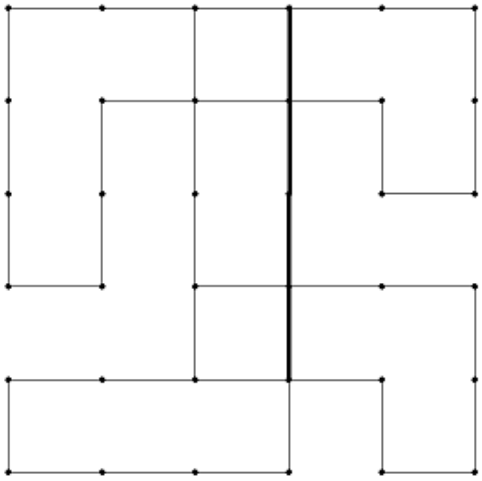

Figure 2.10: Modified Pipe Layout for Scenario #3

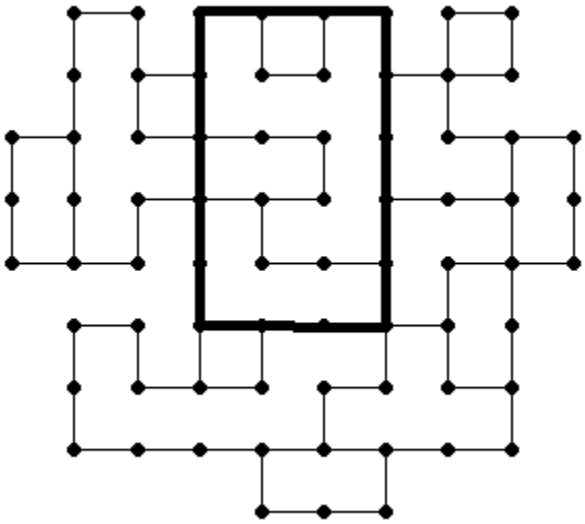

Figure 2.11: Modified Pipe Layout for Scenario #4

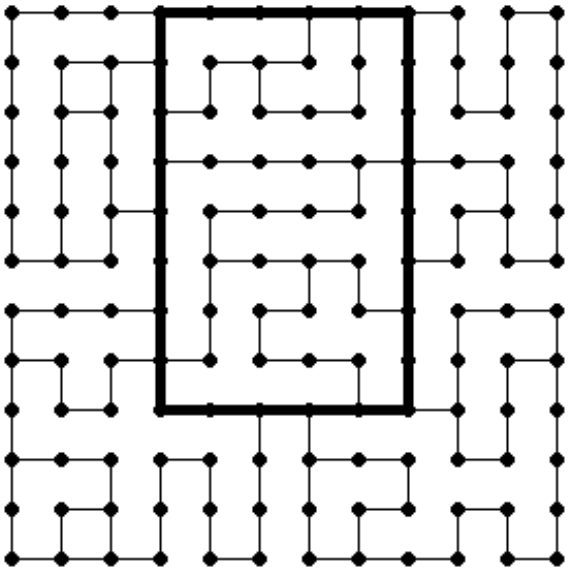

Figure 2.12: Modified Pipe Layout for Scenario #5

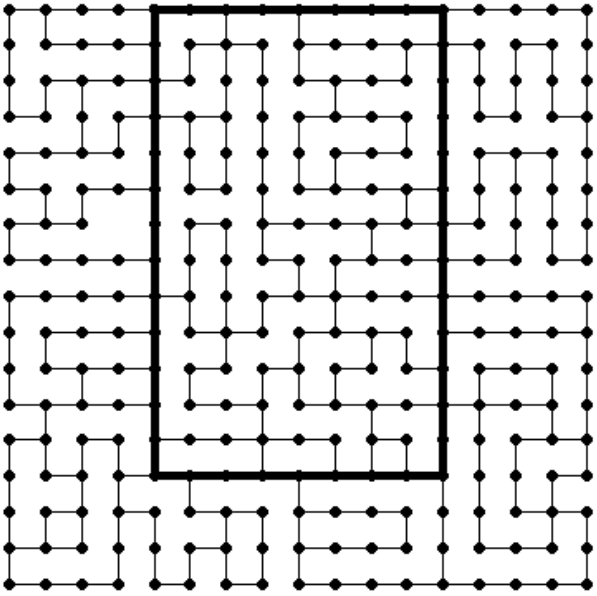

Figure 2.13: Modified Pipe Layout for Scenario #6

It should be noted that the genetic algorithm used is computationally intensive and functions well for systems with less than 120 pipes. However, in scenarios 5 and 6 the systems were too large for use with the genetic algorithm and had to be split into 5 sections that were individually run then combined to form the finished pipe routing. Figure 2.14 and Figure 2.15 display the method of division used for genetic algorithm input.

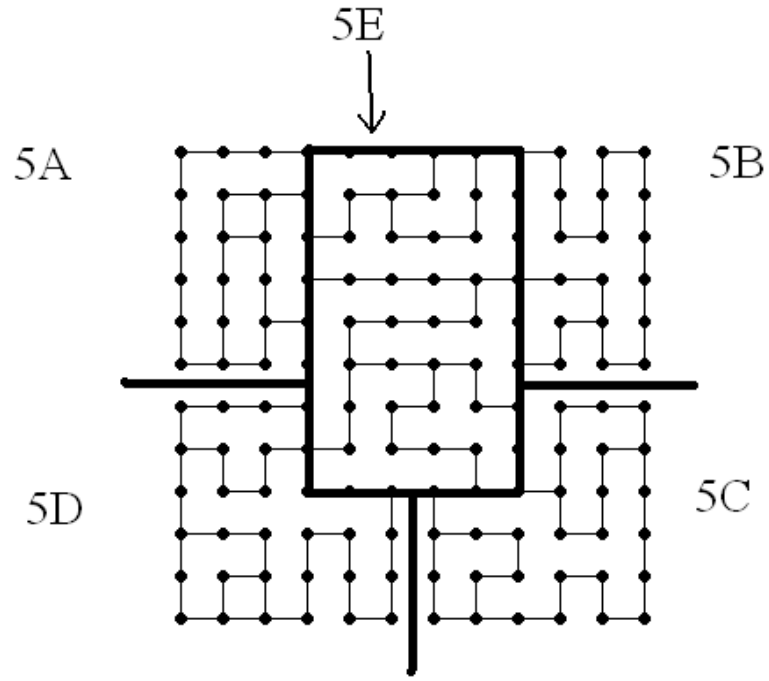

Figure 2.14: Division of Scenario # 5

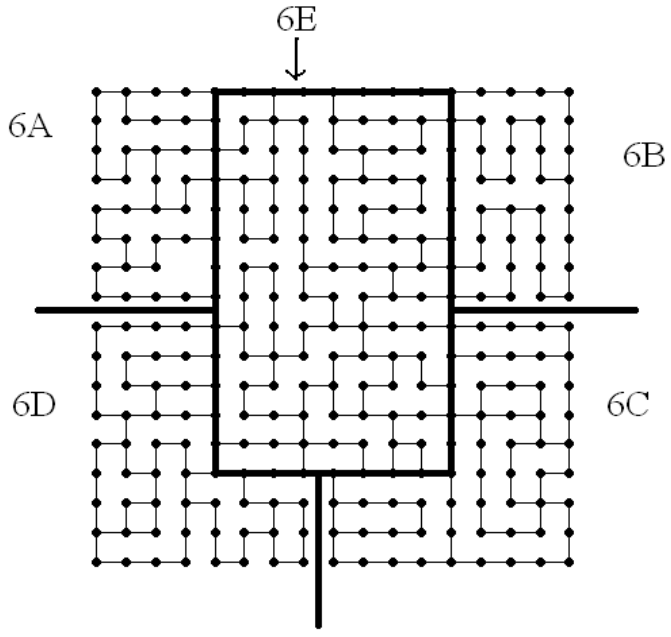

Figure 2.15: Division of Scenario # 6

While the same algorithm was run on each scenario, a larger number of pipes were eliminated in the larger systems. The results of the pipe route optimization procedure for each scenario are presented in Table 2.10.

Table 2.10: Effect of Pipe-Routing Optimization			
Case #	Number of Pipes in Compete Grid	Number of Pipes in Modified Grid	Percent Reduction
1	12	10	16.67%
2	27	23	14.81%
3	60	44	26.67%
4	127	88	30.71%
5	264	164	37.88%
6	532	337	36.65%

2.5.3 System Requirements

The design of a distribution system can greatly affect the performance. Pipe sizing, pipe material, pipe routing, storage tank sizing, pump selection, and pump scheduling directly influence the energy needs of the system and these factors have been carefully considered during the design of each scenario. The distribution systems will consist of 4 km^2 square grids arranged to form square distribution systems. These lines are considered to be major transmission lines and the smaller service lines that branch from them are not considered. System pressures and velocities will be maintained throughout the system according to the Ten States Standards (Managers 2007) and other design recommendations (Shammas and Wang 2011) regarding pressures and velocities to ensure fair comparison among scenarios. These design standards are presented in Table 2.11.

Table 2.11: Pressure and Velocity Requirements		
Item	Value	Unit
Absolute Minimum Pressure	20	(psi)
Absolute Minimum Pressure	138	(kPa)
Absolute Minimum Pressure	14	(m)
Minimum Operating Pressure	35	(psi)
Minimum Operating Pressure	245	(kPa)
Minimum Operating Pressure	25	(m)
Maximum Pressure	100	(psi)
Maximum Pressure	687	(kPa)
Maximum Pressure	70	(m)
Maximum Velocity	9.8	(ft/s)
Maximum Velocity	3	(m/s)

2.5.4 Primary Pumping

The primary pumping station in this study consists of a series of four pumps capable of providing a range of flows from half the normal flow to the sum of peak flow and fire flow. In practical application, additional pumps would be provided for redundancy in the case that one or more pumps must be taken out of commission for maintenance/repair. In this study only these four pumps will be modeled for each scenario and will be designed in parallel to enable multiple pumps to function at once. Figure 2.16 illustrates the layout of the primary pumps for all scenarios.

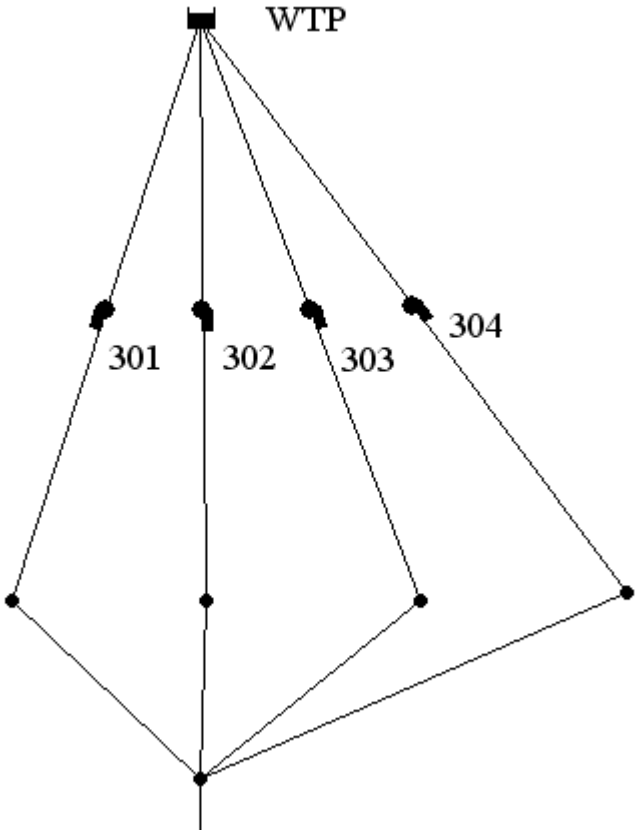

Figure 2.16: Primary Pump Configuration

For each scenario, the desired head and flow of each pump are input into EPANET and pump curves are created for each. See Table 2.12 for additional information regarding the pump design characteristics of each scenario. In addition, a sample pump curve is provided in Figure 2.17.

Table 2.12: Pump Curve Characteristics								
Pump (I.D.#)	Design Flow	Head (m)	Case #1 (Lps)	Case #2 (Lps)	Case #3 (Lps)	Case #4 (Lps)	Case #5 (Lps)	Case #6 (Lps)
301	Half normal flow	40	212.9	425.9	851.7	1,703.4	3,406.9	6,813.7
302	Normal flow	40	425.9	851.7	1,703.4	3,406.9	6,813.7	13,628
303	Half normal flow	40	212.9	425.9	851.7	1,703.4	3,406.9	6,813.7
304	Peak flow + fire flow	70	1,183.8	2,104.9	3,838.2	7,150.9	13,559.0	26,066.0

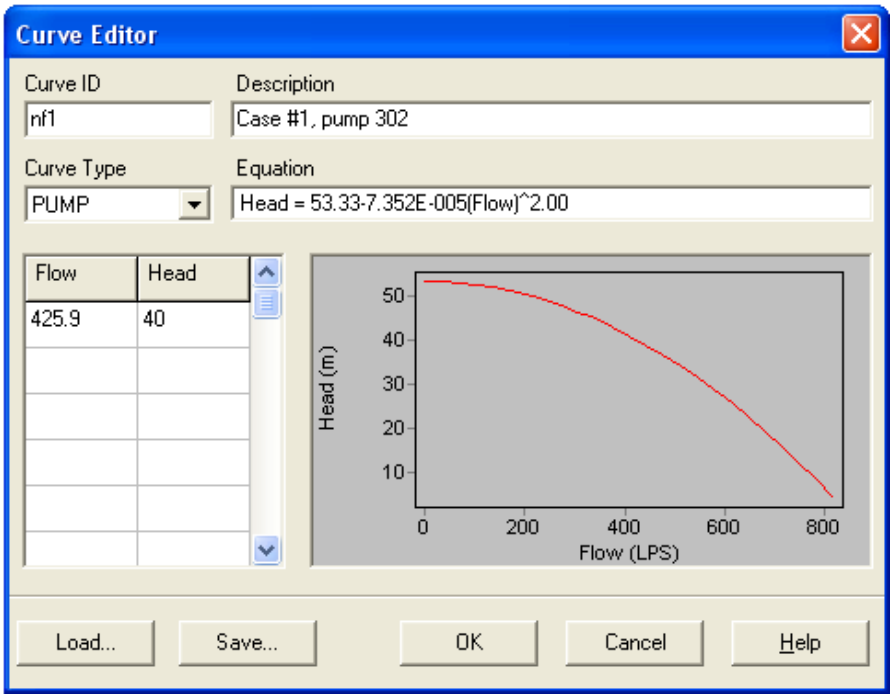

Figure 2.17: Pump Curve for Case #1, Pump 302

2.5.5 Pipe Sizing

In practical application, pipe sizing decisions are based on current conditions and anticipated growth patterns. In many situations city planners dictate where growth will occur and design the system accordingly. As the demand has already been established in this study, pipe sizing decisions are based on the worst case flow condition (peak flow + fire flow). For each scenario, the appropriate demand is allocated to each node in the system and a steady-state analysis is utilized in EPANET to size pipes while maintaining appropriate system pressures and flows. During steady-state analysis, a primary pump (#304) is sized to provide worst case flows, while pumps 301, 302, and 303 are all turned off. During steady state analysis the affect of storage tanks and demand patterns are neglected. Next, pipe sizes are adjusted to maintain velocities between 2 and 3 m/s (especially in mainlines) using an iterative process, and booster pumps are strategically placed at points in the system where pressures drop below the absolute minimum pressure of 138 kPa (20 psi). This process yields a system that is designed to provide minimum pressures throughout the system even under worst case flow conditions.

2.6 Final Distribution System Design

With the system layout and pipe sizing complete, the next step is to add storage tanks and define pump schedules/controls to establish the final designs. Each scenario is simulated in EPANET using an extended period analysis to maintain system performance requirements under daily demand variations.

2.6.1 Primary Pump Scheduling

In an effort to provide each scenario with similar inputs for valid comparison, it was decided that the primary pumps in each system will provide 40 meters (131.23 ft) of

head. The efficiency of all pumps in this study is assumed to be 75% and the primary pumps are designed to operate in parallel to provide the necessary flows dictated by the daily demand pattern. To provide the same level of service, all of the primary pumps in this study will operate under the same pumping schedule with the exception of scenario #1 where the controls are largely the same but pump #303 is turned off at 3am, 5pm and 9pm. The primary pump schedules are described in Table 2.13.

Table 2.13: Primary Pump Schedules

Time	Pump # 301	Pump # 302	Pump # 303	Pump # 304
12:00:00 AM	+	-	-	-
1:00:00 AM	+	-	-	-
2:00:00 AM	+	-	-	-
3:00:00 AM	+	-	+	-
4:00:00 AM	+	-	+	-
5:00:00 AM	+	-	+	-
6:00:00 AM	+	-	+	-
7:00:00 AM	+	-	+	-
8:00:00 AM	+	+	-	-
9:00:00 AM	+	+	-	-
10:00:00 AM	+	+	-	-
11:00:00 AM	+	+	-	-
12:00:00 PM	+	+	-	-
1:00:00 PM	+	+	-	-
2:00:00 PM	+	+	-	-
3:00:00 PM	+	+	-	-
4:00:00 PM	+	+	-	-
5:00:00 PM	+	+	+	-
6:00:00 PM	+	+	+	-
7:00:00 PM	+	-	+	-
8:00:00 PM	+	-	+	-
9:00:00 PM	+	-	+	-
10:00:00 PM	+	-	-	-
11:00:00 PM	+	-	-	-

2.6.2 Booster Pumps

In the event that the head provided by the primary pumps is insufficient to maintain minimum system pressures, additional head will be provided by booster pumps. If required, each booster pump is controlled by either a pump schedule or a tank level in the system. For example, the majority of the booster pumps in each scenario are linked with a tank on the downstream side of the pump. When the tank level reaches a minimum level the corresponding booster pump is turned on, and when the tank level reaches the desired level the booster pump is turned off. This type of booster pump control is entered into EPANET in the following format, "link 64 open if node 89 below 8.8, link 64 closed if node 89 above 12.9", where; "link 64" is a booster pump, "node 89" is a storage tank, and head levels (8.8 and 12.9) are tank levels measured in meters above the tank bottom. The booster pump controls for each scenario are detailed in the following sections. Each booster is individually sized to provide the necessary flows and pressures without exceeding the maximum allowable pressure of 687 kPa (100 psi). Bypass piping is provided for each booster pump to allow flow past the pump when it is not in operation. A directional valve is included in the bypass piping to prevent circular flows when the pump is in operation, and the combined length of each set of bypass piping is 20 meters (10 meters for each pipe) for all booster pumps. The layout of booster pumps and bypass piping is presented in Figure 2.18.

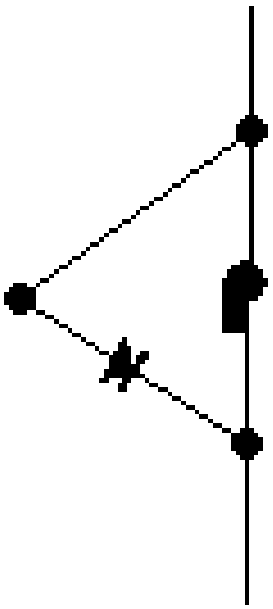

Figure 2.18: Booster Pump Configuration

2.7 Design Summary

Using the methodology provided in the preceding sections, the six scenarios have been designed in a consistent manner to facilitate a valid comparison. The completed distribution system layouts and booster pump characteristics are presented below, where booster pumps are labeled b(pump #) and tanks are labeled t(tank #).

2.7.1 Case #1

In case #1, the 40 meters of head provided by the primary pumps was enough to maintain minimum allowable pressures without the need of booster pumps. Thus, there is no booster pump curve or controls needed for this case. Figure 2.19 represents the completed first scenario.

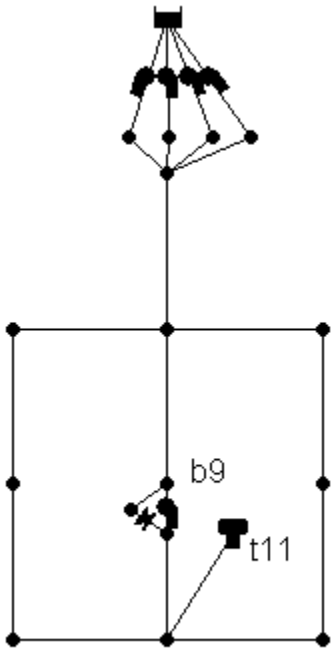

Figure 2.19: Case #1 – Final Layout

2.7.2 Case #2

Similar to Case #1, Case #2 did not require any additional energy input from the booster pumps. Accordingly, no details regarding booster pump curves or controls is provided. Figure 2.20 represents the completed second scenario.

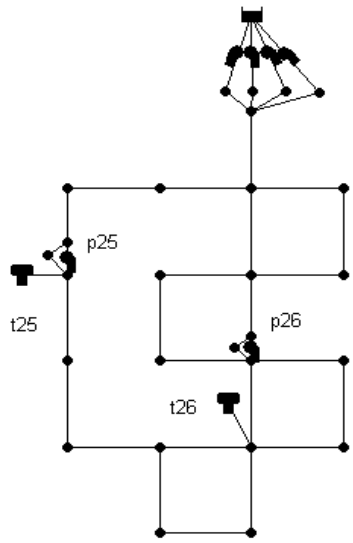

Figure 2.20: Case #2 – Final Layout

2.7.3 Case #3

Case # 3 is the first scenario where the initial head provided by the primary pumps is not enough to maintain minimum pressures without additional energy input from booster pumps. Figure 2.21 illustrates the completed third scenario, while Table 2.14 and Table 2.15 provide information regarding booster pump sizing and controls.

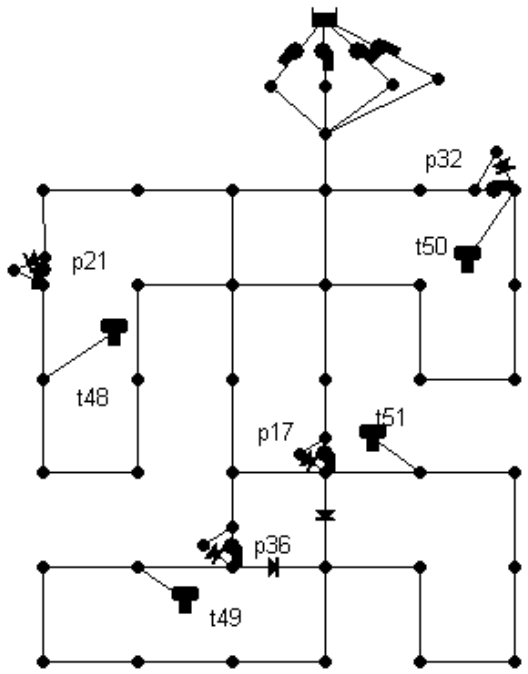

Figure 2.21: Case #3 – Final Layout

Table 2.14: Case # 3 Booster Pump Specifications		
	Flow	Head
Pump #	(Lps)	(m)
17	1,600	7
21	600	15
32	400	20
36	1,100	15

Table 2.15: Case # 3 Booster Pump Controls	
Pump #	Control Statement
17	Link 17 open if node 51 below 8.8, Link 17 closed if node 51 above 11
21	Link 21 open if node 48 below 8.8, Link 21 closed if node 48 above 12.9
32	Link 32 open if node 50 below 8.8, Link 32 closed if node 50 above 12.9
36	Link 36 open if node 49 below 8.8, Link 36 closed if node 49 above 12

2.7.4 Case #4

In case #4, booster pumping is required to maintain adequate system pressures, and the final design is presented Figure 2.22. Table 2.16 and Table 2.17 provide information regarding booster pumps sizing and controls.

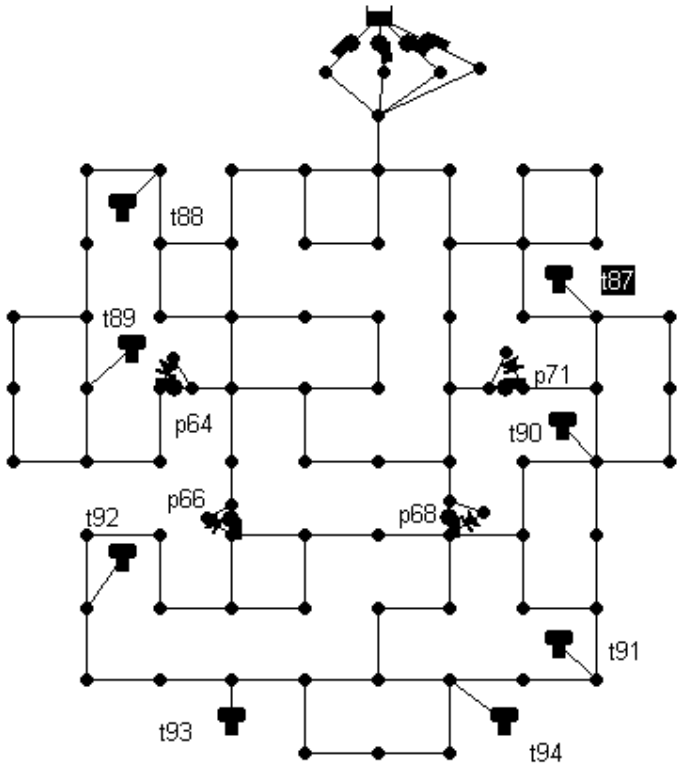

Figure 2.22: Case #4 – Final Layout

Table 2.16: Case # 4 Booster Pump Specifications		
	Flow	Head
Pump #	(Lps)	(m)
64	700	35
66	1,100	20
68	1,200	25
71	600	27

Table 2.17: Case # 4 Booster Pump Controls	
Pump #	Control Statement
64	Link 64 open if node 89 below 8.8, Link 64 closed if node 89 above 12.9
66	Link 66 open if node 93 below 8.8, Link 66 closed if node 93 above 12.9
68	Link 68 open if node 91 below 8.8, Link 68 closed if node 91 above 12.9
71	Link 71 open if node 90 below 8.8, Link 71 closed if node 90 above 12.9

2.7.5 Case #5

In case #5, booster pumping is required to maintain adequate system pressures, and the final design is presented in Figure 2.23. Table 2.18 and Table 2.19 provide information regarding booster pumps sizing and controls.

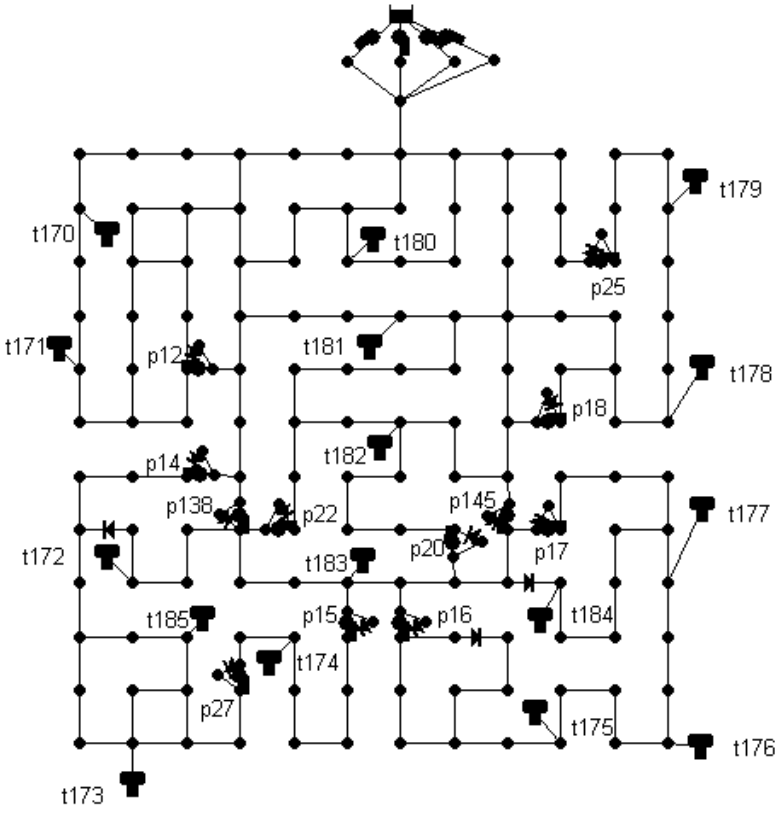

Figure 2.23: Case #5 – Final Layout

Table 2.18: Case # 5 Booster Pump Specifications

Pump #	Flow (Lps)	Head (m)
12	1,500	25
14	1,500	15
15	1,500	25
16	2,000	20
17	1,500	20
18	1,400	25
20	1,500	25
22	1,500	20
25	900	30
27	1,300	30
138	2,500	30
145	2,500	30

Table 2.19: Case # 5 Booster Pump Controls	
Pump #	Control Statement
12	Link 12 open if node 171 below 8.8, Link 12 closed if node 171 above 12.9
14	Link 14 open if node 185 below 10, Link 14 closed if node 185 above 12.9
15	Link 15 open if node 185 below 10, Link 15 closed if node 185 above 12.9
16	Link 16 open if node 176 below 10, Link 16 closed if node 176 above 12.9
17	Link 17 open if node 176 below 8.8, Link 17 closed if node 176 above 12.9
18	Link 18 open if node 178 below 8.8, Link 18 closed if node 178 above 12.9
20	Link 20 open if node 182 below 8.8, Link 20 closed if node 182 above 12.9
22	Link 22 open if node 182 below 10, Link 22 closed if node 182 above 12.9
25	Link 25 open if node 179 below 10, Link 25 closed if node 179 above 12.9
27	Link 27 open if node 185 below 8.8, Link 27 closed if node 185 above 12.9
138	Link 138 closed from 12 am to 2 pm, Link 138 open from clocktime 3 pm to 6 pm, Link 138 closed from clocktime 7 pm to 11 pm
145	Link 145 closed from 12 am to 2 pm, Link 145 open from clocktime 3 pm to 6 pm, Link 145 closed from clocktime 7 pm to 11 pm

2.7.6 Case #6

In case #6, booster pumping is required to maintain adequate system pressures, and the final design is presented in Figure 2.24. Table 2.20 and Table 2.21 provide information regarding booster pumps sizing and controls.

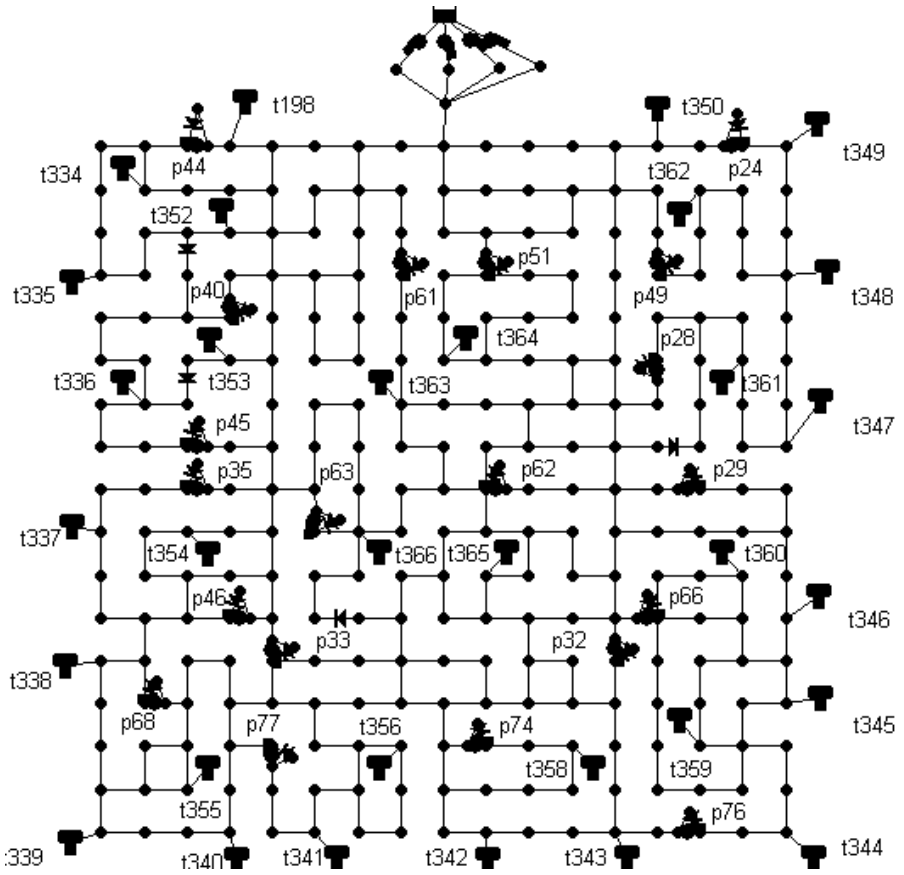

Figure 2.24: Case #6 – Final Layout

Table 2.20: Case # 6 Booster Pump Specifications

	Flow	Head
Pump #	(Lps)	(m)
24	800	40
28	900	28
29	1,500	17
32	3,000	35
33	3,000	45
35	700	25
40	600	22
44	900	30
45	700	35
46	700	33
49	700	40
51	900	45
61	800	35
62	2,800	30
63	2,400	40
66	1,300	28
68	800	10
74	700	20
76	1,200	25
77	400	30

Table 2.21: Case # 6 Booster Pump Controls	
Pump #	Control Statement
24	Link 24 open if node 348 below 8.8, Link 24 closed if node 348 above 12.9
28	Link 28 open if node 347 below 8.8, Link 28 closed if node 347 above 12
29	Link 29 open if node 359 below 8.8, Link 29 closed if node 359 above 12.9
32	Link 32 open if node 342 below 8.8, Link 32 closed if node 342 above 12.9
33	Link 33 open if node 340 below 8.8, Link 33 closed if node 340 above 12
35	Link 35 open if node 337 below 8.8, Link 35 closed if node 337 above 12.9
40	Link 40 open if node 336 below 8.8, Link 40 closed if node 336 above 12.9
44	Link 44 open if node 335 below 9, Link 44 closed if node 335 above 12
45	Link 45 open if node 336 below 8.8, Link 45 closed if node 336 above 12
46	Link 46 open if node 339 below 8.8, Link 46 closed if node 339 above 12.9
49	Link 49 open if node 348 below 8.8, Link 49 closed if node 348 above 12
51	Link 51 open if node 364 below 8.8, Link 51 closed if node 364 above 12
61	Link 61 open if node 363 below 8.8, Link 61 closed if node 363 above 12.9
62	Link 62 open if node 365 below 8.8, Link 62 closed if node 365 above 12.9
63	Link 63 open if node 366 below 8.8, Link 63 closed if node 366 above 12.9
66	Link 66 open if node 359 below 8.8, Link 66 closed if node 359 above 12
68	Link 68 open if node 339 below 8.8, Link 68 closed if node 339 above 12.9
74	Link 74 open if node 358 below 8.8, Link 74 closed if node 358 above 12.9
76	Link 76 open if node 359 below 8.8, Link 76 closed if node 359 above 12.9
77	Link 77 open if node 339 below 8.8, Link 77 closed if node 339 above 12

CHAPTER 3

LIFE CYCLE COST (LCC) AND LIFE CYCLE ASSESSMENT (LCA)

The cost of water supply infrastructure is dependent on a great deal of factors beyond the scope of this study. This chapter will establish the scope of the cost estimates in this study and provide a methodology for the estimation of the various parts of each scenario. It is clear from observations of the aging infrastructure in the U.S. that variations in system age, materials used, system maintenance, operating conditions, and local climate can greatly affect both the initial and operational costs of a water supply system. In this study, these factors are simplified through assumptions and limitations in an attempt to find generalizations for application to a variety of situations. It should be noted that highly accurate cost estimates are not needed for this study and the scope of the following cost estimates is limited through several assumptions. For example, all water system components are considered to be new construction, the cost of storage tanks, primary pump stations, booster pump stations, valves, hydrants, service lines, pipe fittings are not considered, and distribution piping is considered to be installed in a typical right-of-way without obstacles, pavement cuts, or boring under roadways.

3.1 Life Cycle Costs (LCCs)

In the past, the initial construction cost of water supply infrastructure was the main consideration in design selection; however, rising energy costs and an increased attention to operation and maintenance costs have prompted utility planners to consider the Life Cycle Cost (LCC) of design alternatives. LCC is a method of economic analysis

that considers the total costs across the life cycle of a product, process, or system including upfront purchase costs, Operation and Maintenance (O&M) costs and end-of-life handling costs. LCC accounts for all present and future costs by summing all relevant project costs over a given study period in present-value terms. A LCC comparison typically involves 6 steps: identification of design alternatives, identification of common assumptions, identification of relevant project costs, conversion of costs to present value, compute and compare LCCs of alternatives, and interpretation of results. As fiscal costs are commonly the primary consideration in water supply system design, this study will involve LCC estimation for each of the six scenarios studied.

3.1.1 DWTP Construction Cost Estimation

For each scenario, a conventional DWTP is modeled to include chemical addition (typical chemicals include Potassium Permanganate, Alum or Ferric Salts, Polymers, and Chlorine), flocculation, sedimentation, filtration, and disinfection according to the specifications of McGivney and Kawamura (2008). The performance of this DWTP is not the focus of this study and it is included for cost estimation purposes only. The lifespan of the DWTP is assumed to be 50 years. The total capital costs of the DWTPs are estimated for each scenario according to the equation provided by McGivney and Kawamura (2008) for a conventional DWTP.

$$Y = 8.7684 * x^{0.5957}$$

Where, Y = Total Capital Costs (millions of U.S. dollars)

x = Design Flow (MGD)

3.1.2 DWTP Operation and Maintenance Cost Estimation

The total O & M costs of the DWTPs are estimated for each scenario according to the equation provided by McGivney and Kawamura (2008) for a conventional DWTP.

$Y = 0.4384 * x^{0.2946}$

Where, Y = Annual O & M Costs (millions of U.S. dollars/year)

x = Design Flow (MGD)

Annual DWTP O & M costs will be converted to current values using an annual discount rate of 7%.

3.1.3 Energy Cost Estimation

Annual energy consumption estimates are derived from each EPANET simulation and combined with the unit energy cost assumption ($0.20 per kWh) to estimate the total annual energy cost by,

$y = e * c$

Where, y = Total annual energy cost

e = Annual energy consumption (kWh)

c = Unit energy cost ($/kWh)

Note, the effect of rising energy costs is not considered in this study. Annual pumping costs will be converted to current values LCC comparison using an annual discount rate of 7%.

3.1.4 Distribution Piping Cost Estimation

The capital costs of the distribution system will be estimated according to three sources: Clark's method (Clark, Sivaganesan et al. 2002), RSMeans method (Spencer 2007) and industry method (cost data collected directly from industry professionals). O

& M costs of the distribution system are not considered beyond pumping energy requirements. In addition, the cost of storage tanks, primary pump stations, booster pump stations, valves, hydrants, service lines, pipe fittings are not included in the LCC comparison, and distribution piping is considered to be installed in a typical right-of-way without obstacles, pavement cuts, or boring under roadways. Installation of distribution mains will be estimated to include: excavation, bedding, material cost, installation cost, and backfill. Material and installation costs vary for or all three methods of estimation, while the cost of excavation, bedding, and backfill are estimated for each method according to the methodology of Clark, Sivaganesan et al. (2002).

3.1.4.1 Clark's Method of Pipeline Cost Estimation

Clark's method provides a detailed methodology for estimating the cost of excavation, bedding, installation, and backfill (Clark, Sivaganesan et al. 2002). The parameter values used for each portion of pipeline installation are presented in Table 3.1. The unit cost of each item is calculated by,

$$y = a + b * x^c + d * u^e + f * x * u$$

Where, y = unit cost ($/ft)

u = indicator variable

a, b, c, d, e, and f = parameter values in table 3.1

x = pipe diameter (inches)

Table 3.1: List of Parameter Values						
Variable	Pipe Installation - Ductile Iron Pipe (DIP)	Pipe Installation - Conc. Cyl. Pipe (CCP)	Pipe Installation - Prestressed Conc. Cyl. Pipe (PCCP)	Excavation - Sandy Gravel Soil, 1:1 Side slope	Backfill - Sandy Native Soil, 1:1 Side Slope	Bedding - Ordinary
a	-44	11.7	7.9	2.9	-0.094	1.6
b	0.33	0.51	1.3	0.0018	-0.062	0.0062
c	1.72	1.38	1.25	1.9	0.73	1.83
d	2.87	0	0	0.13	0.18	-0.2
e	0.74	0	0	1.77	2.03	1
f	0	0	0	0	0.02	0.07
u	50	0	0	Trench Depth (ft)	Trench Depth (ft)	0

For excavation cost estimates, it is assumed that the native soil is “sandy gravel soil” with a side slope of 1:1, the required cover above the top of each pipe is 1.067 meters, and the indicator variable is the trench depth in feet. Required trench depths are calculated by,

$y = d + c$

Where, y = Trench depth (ft)

d = Pipe diameter (ft)

c = Required cover above each pipe (ft)

To estimate the cost of pipeline bedding, it is assumed that the bedding type is “ordinary” as specified by Clark, Savaganesan et al. (2002). To estimate the cost of pipeline placement (installation), it is assumed that pipes with diameters ranging from 8"-

20" are DIPs, pipes with diameters ranging from 24"-48" are Concrete Cylinder Pipes (CCPs), and pipes with diameters ranging from 54"- 144” are Pre-stressed Concrete Cylinder Pipe (PCCP). It is assumed that all DIP pipes are class 50. To estimate the cost of backfill it is assumed that the soil is “sandy native soil” with 1:1 side slope.

By combining these estimates with material cost estimates from (Spencer 2007), the total unit cost for each size pipeline are estimated and the results are presented in Table 3.2.

Table 3.2: Clark's Estimation Method				
Diameter (inches)	Unit Material Cost (from Means) ($/ft)	Unit Cost for Installation ($/ft)	Unit Cost for Excavation, Bedding, and Backfill ($/ft)	Total Cost ($/ft)
8	$18.20	$6.00	$4.69	$28.89
10	$24.50	$7.69	$4.81	$36.99
12	$30.50	$9.63	$4.93	$45.06
14	$38.50	$11.82	$5.07	$55.40
16	$42.00	$14.25	$5.22	$61.48
18	$53.00	$16.91	$5.38	$75.30
20	$62.00	$19.80	$5.55	$87.35
24	$86.50	$16.05	$5.92	$108.47
30	$96.75	$20.55	$6.55	$123.85
36	$107.00	$25.41	$9.98	$142.39
42	$126.50	$30.59	$10.84	$167.93
48	$146.00	$36.05	$11.79	$193.84
54	$193.00	$41.79	$12.81	$247.60
60	$240.00	$68.58	$17.54	$326.12
66	$292.50	$76.95	$18.79	$388.23
72	$300.00	$85.51	$20.11	$405.62
78	$345.00	$94.26	$21.50	$460.76
84	$390.00	$103.17	$27.51	$520.68
90	$485.00	$112.25	$29.11	$626.36
96	$580.00	$121.48	$30.79	$732.27
102	$687.50	$130.85	$32.54	$850.89
108	$795.00	$140.36	$39.81	$975.17
114	$985.00	$150.01	$41.76	$1,176.77
120	$1,175.00	$159.78	$43.79	$1,378.57
126	$1,243.75	$169.68	$52.17	$1,465.60
132	$1,312.50	$179.70	$54.40	$1,546.60
138	$1,381.25	$189.83	$56.70	$1,627.77
144	$1,450.00	$200.07	$59.06	$1,709.13

3.1.4.2 RSMeans Method of Pipeline Cost Estimation

The *RSMeans Heavy Construction Cost Data* manual (Spencer 2007) is commonly used in the construction industry to estimate the cost of a wide variety of projects. In the RSMeans estimation method, unit costs for pipeline material and installation are derived from Spencer (2007), while the unit cost for excavation, bedding, and backfill are derived from Clark, Savaganesan et al. (2002). For this method, it is assumed that pipes with diameters ranging from 8"-20" are DIPs, pipes with diameters ranging from 24"-48" are Concrete Cylinder Pipes (CCPs), and pipes with diameters ranging from 54"- 144 are Pre-stressed Concrete Cylinder Pipes (PCCPs). The total unit cost estimates for each size pipeline are presented in Table 3.3.

Table 3.3: RSMeans Estimation Method				
Diameter (inches)	Unit Material Cost (from RSMeans) ($/ft)	Unit Cost for Installation (from RSMeans) ($/ft)	Unit Cost for Excavation, Bedding, and Backfill (Clark, Sivaganesan et al. 2002) ($/ft)	Total Cost ($/ft)
8	$18.20	22.8	$4.69	$45.69
10	$24.50	27.5	$4.81	$56.81
12	$30.50	30	$4.93	$65.43
14	$38.50	32.5	$5.07	$76.07
16	$42.00	43	$5.22	$90.22
18	$53.00	46	$5.38	$104.38
20	$62.00	56	$5.55	$123.55
24	$86.50	35.5	$5.92	$127.92
30	$96.75	43.25	$6.55	$146.55
36	$107.00	51	$9.98	$167.98
42	$126.50	63.5	$10.84	$200.84
48	$146.00	76	$11.79	$233.79
54	$193.00	83	$12.81	$288.81
60	$240.00	90	$17.54	$347.54
66	$292.50	102.5	$18.79	$413.79
72	$300.00	95	$20.11	$415.11
78	$345.00	115	$21.50	$481.50
84	$390.00	135	$27.51	$552.51
90	$485.00	155	$29.11	$669.11
96	$580.00	175	$30.79	$785.79
102	$687.50	202.5	$32.54	$922.54
108	$795.00	230	$39.81	$1,064.81
114	$985.00	315	$41.76	$1,341.76
120	$1,175.00	400	$43.79	$1,618.79
126	$1,243.75	406.25	$52.17	$1,702.17
132	$1,312.50	412.5	$54.40	$1,779.40
138	$1,381.25	418.75	$56.70	$1,856.70
144	$1,450.00	425	$59.06	$1,934.06

3.1.4.3 Industry Method of Pipeline Cost Estimation

The industry method of pipeline cost estimation was created using unit cost estimates for materials and installation obtained from industry professionals. It should be noted that the unit material and installation costs are subject to a great deal of variation due to factors such as: the cost of raw materials, pipe specifications, length of the pipeline, and site conditions.

For this method, it is assumed that pipes with diameters ranging from 8"-54" are DIP, and diameters ranging from 54"- 144 are spiral welded steel. These materials differ from the previous two methods of unit pipeline cost estimation which utilize DIP, concrete cylinder pipes, and pre-stressed concrete cylinder pipes. Estimates of unit installation costs presented in Table 3.4 were plotted and fitted with an exponential trend-line in Figure 3.1.

Table 3.4: Installation Estimate from Industry

Diameter (inches)	Unit Cost ($/ft)
8	$23.00
20	$38.00
48	$70.00
72	$140.00
96	$260.00
120	$600.00

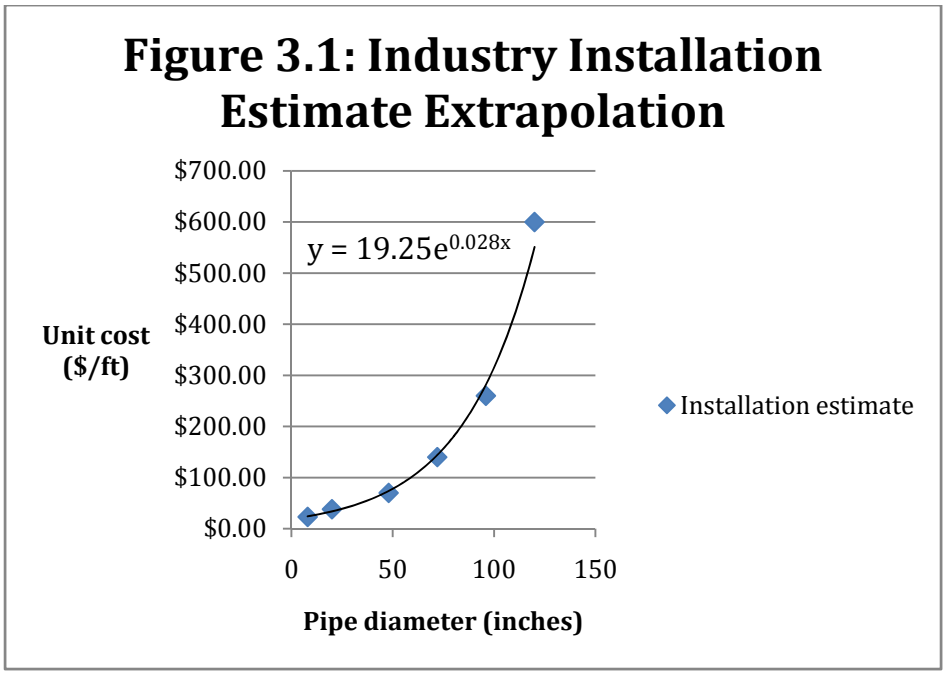

Figure 3.1: Industry Installation Estimate Extrapolation

The trend-line presented in Figure 3.1 was then used to extrapolate the installation costs across all of the diameters included in this study (see table 3.5) by,

$y = 19.25 * e^{(0.028x)}$

Where, y = unit installation costs ($/ft)

x = pipe diameter (inches)

Table 3.5: Industry Estimation Method				
Diameter (inches)	Unit Material Cost (from Industry) ($/ft)	Unit Cost for Installation (from Industry) ($/ft)	Unit Cost for Excavation, Bedding, and Backfill (from Clark et al., 2002) ($/ft)	Total Cost ($/ft)
8	$33.08	$24.08	$4.69	$61.85
10	$46.90	$25.47	$4.81	$77.18
12	$54.38	$26.94	$4.93	$86.25
14	$79.06	$28.49	$5.07	$112.62
16	$81.58	$30.13	$5.22	$116.93
18	$107.84	$31.87	$5.38	$145.09
20	$111.96	$33.70	$5.55	$151.21
24	$140.50	$37.69	$5.92	$184.12
30	$205.94	$44.59	$6.55	$257.08
36	$277.22	$52.75	$9.98	$339.95
42	$229.73	$62.40	$10.84	$302.97
48	$293.17	$73.81	$11.79	$378.77
54	$368.07	$87.31	$12.81	$468.19
60	$422.61	$103.29	$17.54	$543.44
66	$320.63	$122.18	$18.79	$461.60
72	$350.00	$144.53	$20.11	$514.64
78	$379.37	$170.97	$21.50	$571.84
84	$426.89	$202.25	$27.51	$656.65
90	$480.14	$239.25	$29.11	$748.50
96	$535.59	$283.02	$30.79	$849.40
102	$594.86	$334.79	$32.54	$962.19
108	$656.23	$396.04	$39.81	$1,092.07
114	$721.51	$468.49	$41.76	$1,231.76
120	$788.79	$554.19	$43.79	$1,386.77
126	$860.09	$655.57	$52.17	$1,567.83
132	$934.42	$775.50	$54.40	$1,764.33
138	$1,010.60	$917.37	$56.70	$1,984.67
144	$1,090.95	$1,085.19	$59.06	$2,235.20

3.1.5 Comparison of Pipeline Unit Cost Estimation Methods

The results for unit material costs of each method are presented in Table 3.6 and displayed in Figure 3.2.

Table 3.6: Pipeline Material Unit Cost Comparison

Diameter (inches)	Clark ($/ft)	Means ($/ft)	Industry ($/ft)
8	$18.20	$18.20	$33.08
10	$24.50	$24.50	$46.90
12	$30.50	$30.50	$54.38
14	$38.50	$38.50	$79.06
16	$42.00	$42.00	$81.58
18	$53.00	$53.00	$107.84
20	$62.00	$62.00	$111.96
24	$86.50	$86.50	$140.50
30	$96.75	$96.75	$205.94
36	$107.00	$107.00	$277.22
42	$126.50	$126.50	$229.73
48	$146.00	$146.00	$293.17
54	$193.00	$193.00	$368.07
60	$240.00	$240.00	$422.61
66	$292.50	$292.50	$320.63
72	$300.00	$300.00	$350.00
78	$345.00	$345.00	$379.37
84	$390.00	$390.00	$426.89
90	$485.00	$485.00	$480.14
96	$580.00	$580.00	$535.59
102	$687.50	$687.50	$594.86
108	$795.00	$795.00	$656.23
114	$985.00	$985.00	$721.51
120	$1,175.00	$1,175.00	$788.79
126	$1,243.75	$1,243.75	$860.09
132	$1,312.50	$1,312.50	$934.42
138	$1,381.25	$1,381.25	$1,010.60
144	$1,450.00	$1,450.00	$1,090.95

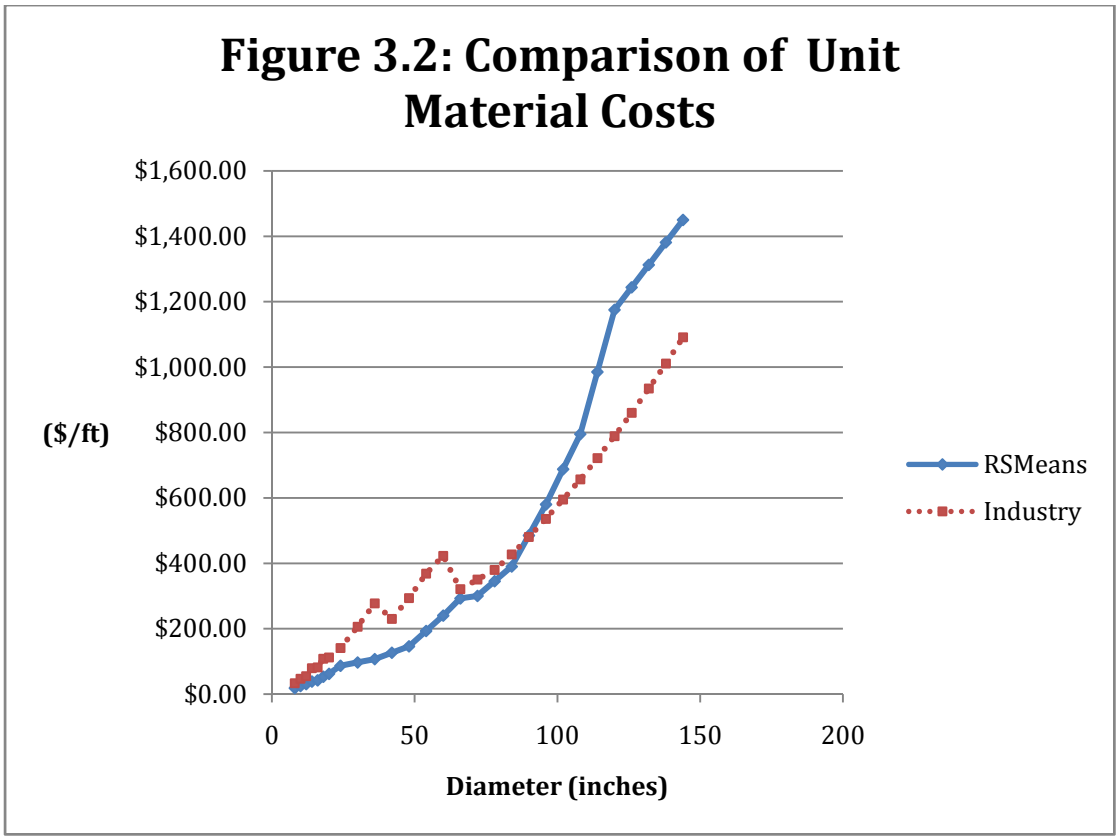

Figure 3.2: Comparison of Unit Material Costs

The results for the unit installation (placement) costs are presented in Table 3.7 and displayed in Figure 3.3.

Table 3.7: Pipeline Installation Unit Cost Comparison			
Diameter (inches)	Clark ($/ft)	Means ($/ft)	Industry ($/ft)
8	$6.00	$22.80	$24.08
10	$7.69	$27.50	$25.47
12	$9.63	$30.00	$26.94
14	$11.82	$32.50	$28.49
16	$14.25	$43.00	$30.13
18	$16.91	$46.00	$31.87
20	$19.80	$56.00	$33.70
24	$16.05	$35.50	$37.69
30	$20.55	$43.25	$44.59
36	$25.41	$51.00	$52.75
42	$30.59	$63.50	$62.40
48	$36.05	$76.00	$73.81
54	$41.79	$83.00	$87.31
60	$68.58	$90.00	$103.29
66	$76.95	$102.50	$122.18
72	$85.51	$95.00	$144.53
78	$94.26	$115.00	$170.97
84	$103.17	$135.00	$202.25
90	$112.25	$155.00	$239.25
96	$121.48	$175.00	$283.02
102	$130.85	$202.50	$334.79
108	$140.36	$230.00	$396.04
114	$150.01	$315.00	$468.49
120	$159.78	$400.00	$554.19
126	$169.68	$406.25	$655.57
132	$179.70	$412.50	$775.50
138	$189.83	$418.75	$917.37
144	$200.07	$425.00	$1,085.19

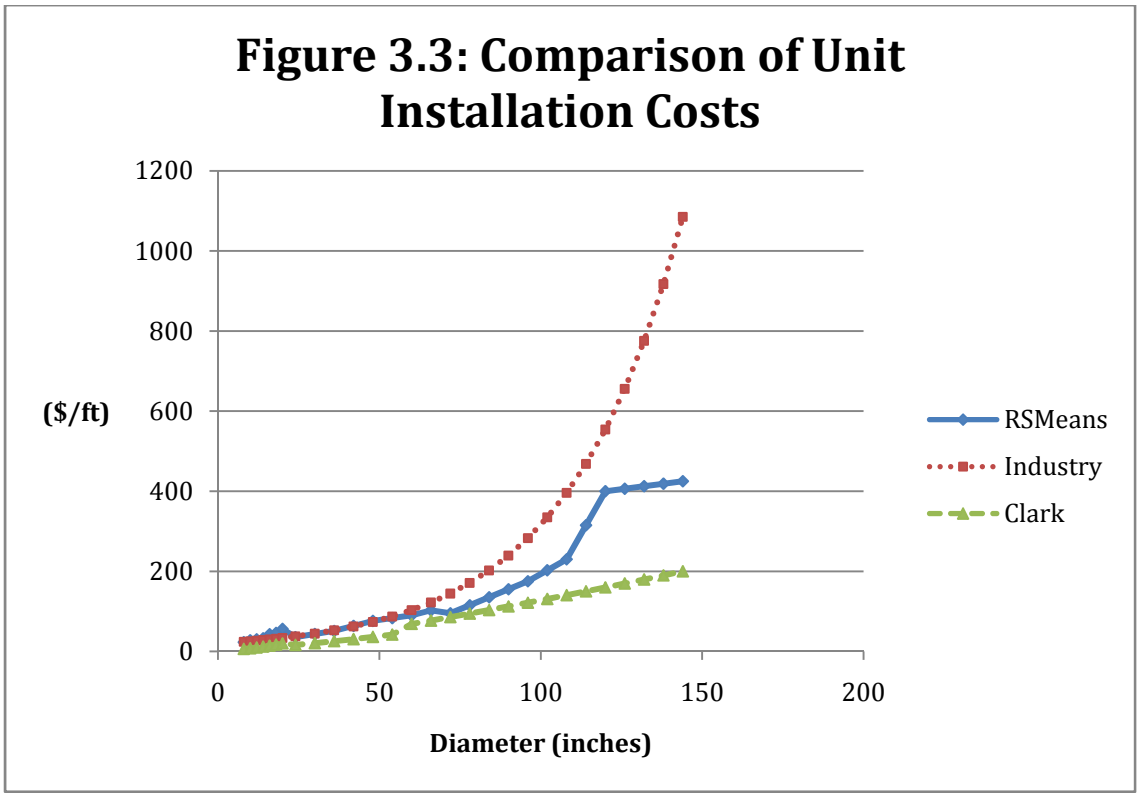

Figure 3.3: Comparison of Unit Installation Costs

The results for total pipeline unit costs for all three estimation methods are presented in Table 3.8 and displayed in Figure 3.4.

Table 3.8: Total Pipeline Unit Cost Comparison			
Diameter (inches)	Clark ($/ft)	Means ($/ft)	Industry ($/ft)
8	$28.89	$45.69	$61.85
10	$36.99	$56.81	$77.18
12	$45.06	$65.43	$86.25
14	$55.40	$76.07	$112.62
16	$61.48	$90.22	$116.93
18	$75.30	$104.38	$145.09
20	$87.35	$123.55	$151.21
24	$108.47	$127.92	$184.12
30	$123.85	$146.55	$257.08
36	$142.39	$167.98	$339.95
42	$167.93	$200.84	$302.97
48	$193.84	$233.79	$378.77
54	$247.60	$288.81	$468.19
60	$326.12	$347.54	$543.44
66	$388.23	$413.79	$461.60
72	$405.62	$415.11	$514.64
78	$460.76	$481.50	$571.84
84	$520.68	$552.51	$656.65
90	$626.36	$669.11	$748.50
96	$732.27	$785.79	$849.40
102	$850.89	$922.54	$962.19
108	$975.17	$1,064.81	$1,092.07
114	$1,176.77	$1,341.76	$1,231.76
120	$1,378.57	$1,618.79	$1,386.77
126	$1,465.60	$1,702.17	$1,567.83
132	$1,546.60	$1,779.40	$1,764.33
138	$1,627.77	$1,856.70	$1,984.67
144	$1,709.13	$1,934.06	$2,235.20

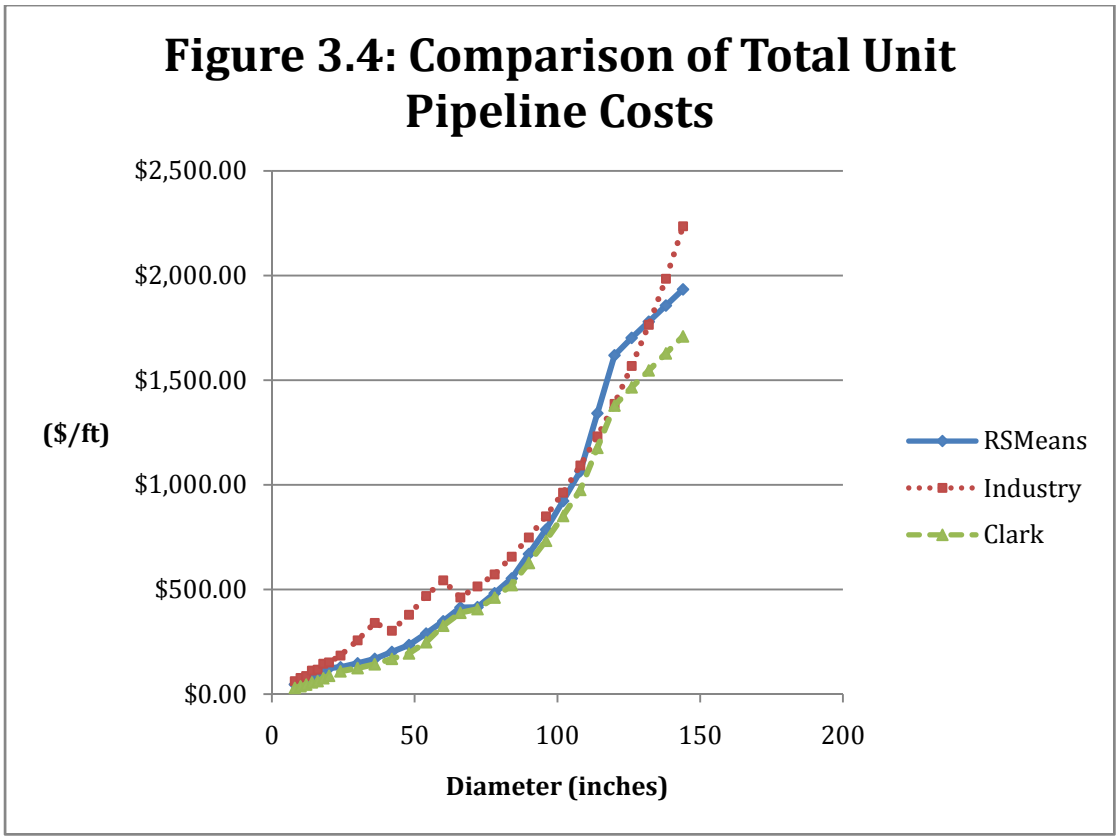

Figure 3.4: Comparison of Total Unit Pipeline Costs

3.1.6 Total Distribution System Pipeline Costs

The total distribution system pipeline costs for each scenario are calculated by summing the total length of each pipe size and multiplying them by the corresponding unit pipeline costs listed in Table 3.8.

3.1.7 Current-value Conversion

In this study, DWTP O & M and distribution system pumping costs occur every year of the 50-year design life and must be converted into present-value terms for comparison. In each scenario, these recurring costs are converted to present-value terms using a 7% annual discount rate by,

$$PV = FV * ((1 - (1 + i)^{-n}) / i)$$

Where, PV = Present-value ($)

FV = Future-value ($)

i = Annual discount rate

n = Design life (years)

3.2 Life Cycle Assessment (LCA)

The scope of LCAs in the water industry are typically limited in three ways; by limiting the level of detail being modeled, by limiting assessment to the operational phase of the lifecycle, and by limiting the number of water system components studied. The scope of the LCA in this study will be limited to the pumping energy requirements of each scenario without regard to DWTP operation or the construction of the water supply systems.

In this study, the LCA is developed using SimaPro 7.2 software and the TRACI2 V3.01 method of impact assessment. The LCA results for the production of 1 kWh of electricity are shown in Table 3.9: SimaPro 7.2 LCA of 1 kWh 'Electricity, high voltage, at grid/US U'. The total daily environmental effects of each scenario will be calculated by multiplying the daily flow (m^3/d) by the unit value of each impact category.

Table 3.9: LCA per Unit of Energy (1 kWh 'Electricity, high voltage, at grid/US U')

Method:	TRACI 2 V3.01	
Impact category	Value	Unit
Global Warming	7.56E-01	(kg CO2 eq)
Acidification	2.69E-01	(H+ moles eq)
Carcinogenics	9.26E-04	(kg benzen eq)
Non carcinogenics	4.67E+00	(kg toluen eq)
Respiratory effects	1.42E-03	(kg PM2.5 eq)
Eutrophication	1.39E-04	(kg N eq)
Ozone depletion	2.01E-08	(kg CFC-11 eq)
Ecotoxicity	1.51E+00	(kg 2,4-D eq)
Smog	1.46E-03	(kg NOx eq)

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Energy Use

The total pumping-energy and unit pumping-energy requirements for each scenario are presented in Table 4.1 and the unit pumping-energy is displayed in Figure 4.1.

Table 4.1: Distribution System Pumping-Energy Requirements		
Case #	Pumping Energy	Unit Pumping Energy
	(kWh/day)	(kWh/m^3)
1	5,611.7	0.152
2	11,513.2	0.157
3	26,513.1	0.180
4	58,617.8	0.199
5	130,849.6	0.222
6	281,821.6	0.239

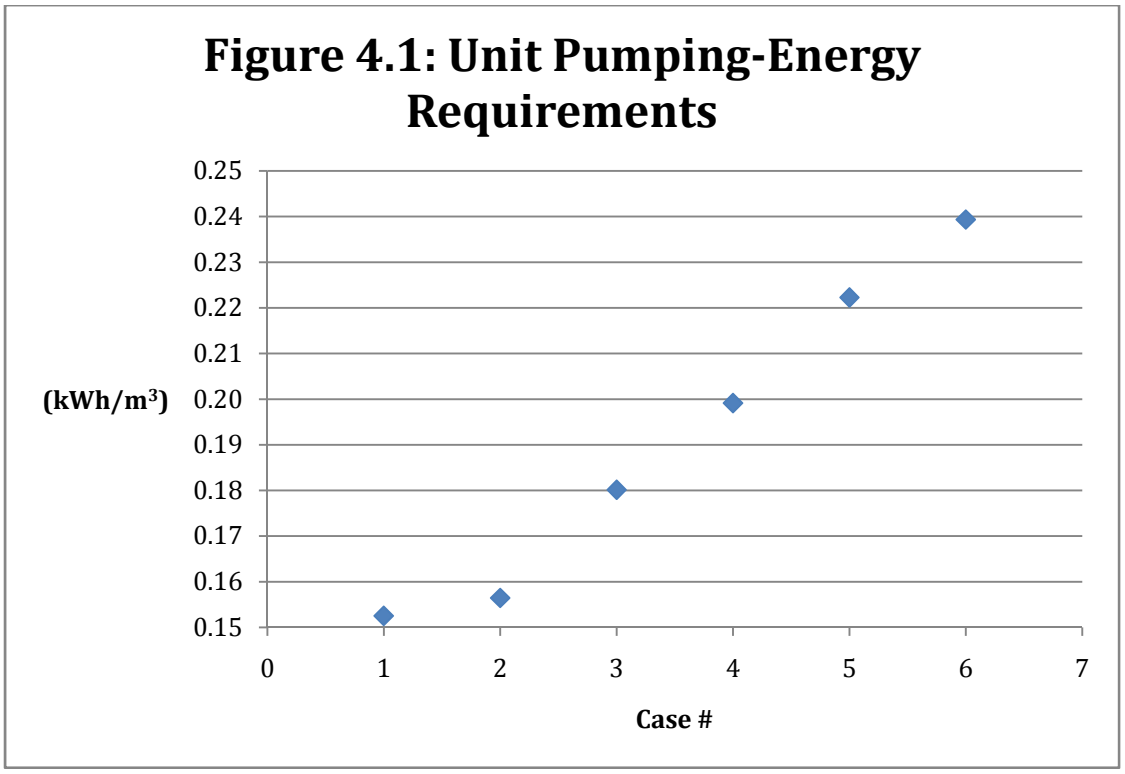

Figure 4.1: Unit Pumping-Energy Requirements

As the primary pumps are sized and controlled according to the same procedure for each scenario, the additional pumping energy required for each scenario are the result of booster pump operation. In scenarios #1 and #2 the booster pumps are not required during normal operation and the slight unit cost difference is due to the decreased primary pumping operation of pump #303 in scenario #1. Case #3 is the first scenario where additional energy input is required from the booster pumps and the amount of energy required for each unit of water entering the system progressively increases for scenarios #3-6. This increase in unit pumping-energy is due to the additional head losses incurred as progressively longer runs of pipe are required to deliver water to the system periphery. The reduced unit pumping-energy requirements of the smaller scenarios can also be partially attributed to the fact that pipe sizing decisions are based on worst-case

flow conditions where the estimated fire flows are a larger percentage of normal flow in the smaller systems. This concept results in a "over-design" factor that increases the required pipe diameters and has a greater affect on the smaller scenarios, as friction losses are reduced with increasing diameter, the energy requirements are affected accordingly.

On the other hand, energy-efficiency of all the pumps in this study is assumed to be 75%, while the actual efficiency would generally increase with pump size. In addition, the Hazen-Williams friction factor was assumed to be 140 for all of the pipes in this study. In reality, changes in pipe diameter affect this value and the resulting head losses. Consideration of these factors would likely result in decreased unit energy requirements in larger systems.

4.2 Capital Costs

The capital cost estimates in this study are based on DWTP and distribution system construction costs. These cost estimates do not include pumping stations, storage tanks, valves, and pipe fittings. The capital cost estimates are presented in the following sections.

4.2.1 DWTP Capital Costs

The cost to provide each scenario with a conventional DWTP are presented in Table 4.2 and illustrated in Figure 4.2.

Table 4.2: DWTP Capital Cost Estimates		
Case #	Flow	DWTP Capital Cost
	(MGD)	($)
1	9.72	$33,983,895
2	19.44	$51,356,653
3	38.88	$77,610,465
4	77.76	$117,285,373
5	155.52	$177,242,318
6	311.04	$267,849,592

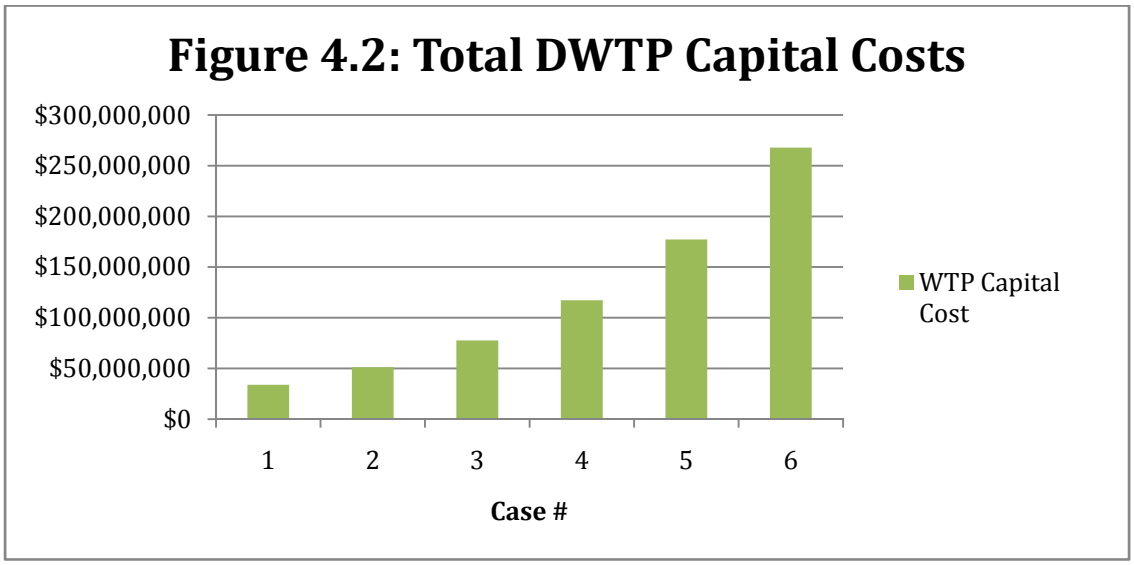

Figure 4.2: Total DWTP Capital Costs

From Figure 4.2 it is evident that while the size of each scenario is doubled from one scenario to the next, the DWTP capital costs are increased by a factor of 51.1%. This concept is more clearly represented when the DWTP capital costs are converted to unit costs in Table 4.3 and Figure 4.3. These unit costs savings are the result of the economies of scale associated with DWTP construction and are the driving force behind the idea that “bigger is better”.

Table 4.3: WTP Capital Costs per m^3 of Daily Flow		
Case #	Flow	WTP Capital Cost
	(m^3/d)	($/$m^3$)
1	36,794.20	$23.28
2	73,588.40	$14.28
3	147,176.81	$8.76
4	294,353.62	$5.37
5	588,707.24	$3.29
6	1,177,414.47	$2.02

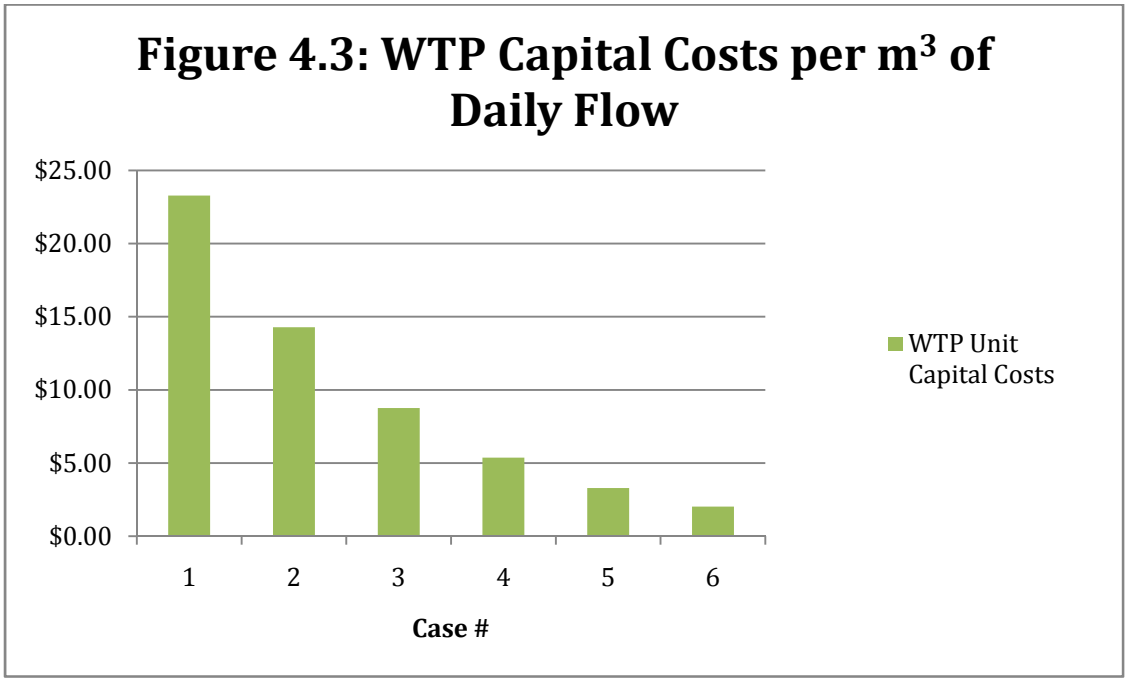

Figure 4.3: DWTP Capital Costs per m^3 of Daily Flow

4.2.2 Distribution System Capital Costs

The total capital costs of distribution system piping are presented in Table 4.4 and Figure 4.4. It is evident from these figures that while the size of each scenario is doubled

from one scenario to the next, the total pipeline capital costs are increased by a factor of roughly 245%.

Table 4.4: Effect of Estimation Method on Total Pipeline Costs			
Case #	Clark	RSMeans	Industry
	(Millions of $)	(Millions of $)	(Millions of $)
1	$3.30	$4.83	$6.27
2	$10.10	$13.56	$20.16
3	$25.10	$32.40	$49.85
4	$52.97	$67.59	$100.19
5	$137.14	$165.54	$231.35
6	$326.83	$390.83	$515.80

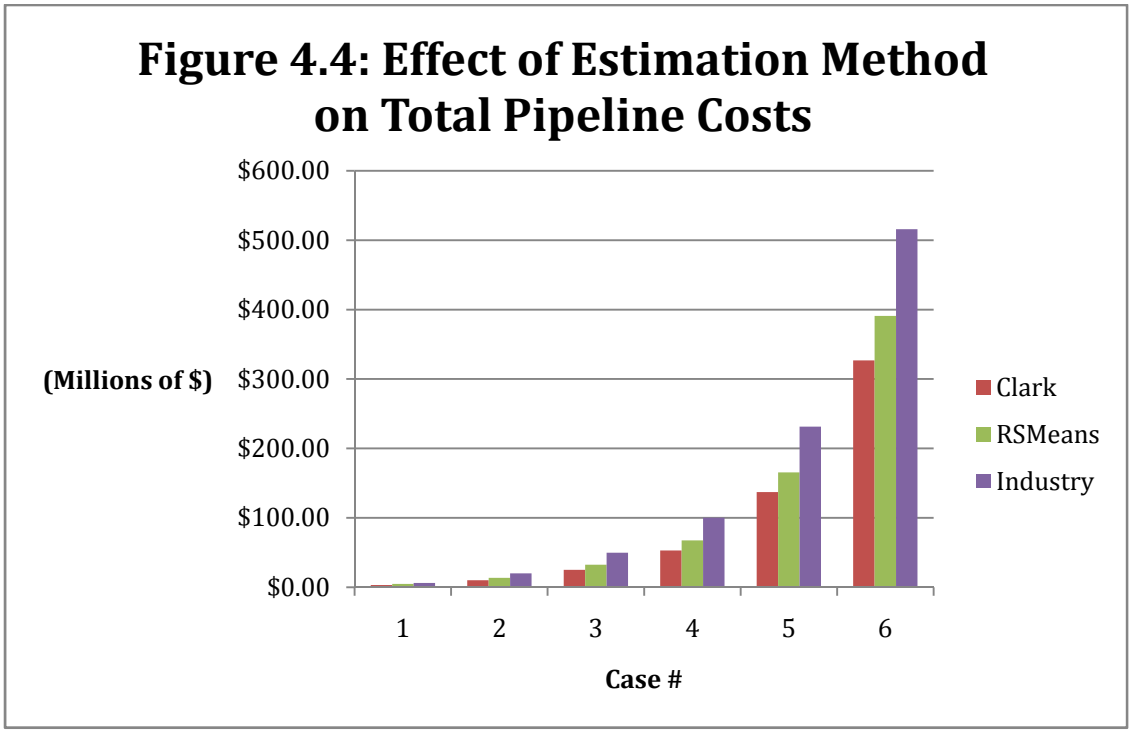

Figure 4.4: Effect of Estimation Method on Total Pipeline Costs

Contrary to the unit cost savings of larger DWTPs, this study found that the unit cost for distribution piping increases as system size increases. This trend is due to the fact that as the diameter of each pipeline increases the unit cost increases exponentially. As larger distribution systems require an increased occurrence of large diameter pipes to accommodate flows, the unit cost for distribution piping increases. This trend can be seen in Table 4.5 and Figure 4.5. It should be noted that this trend is true regardless of the pipeline estimation method used and is particularly significant due to the fact that pipeline optimization had a progressively greater effect on the number of pipelines reduced for larger systems (see Table 2.10). In addition, pipe sizing decisions were based the worst-case flow condition of peak flow + fire flow, where the additional fire flow for smaller systems is a larger percentage of total flow (see the ratio of NF to FF in Table 4.7 for details). The result of this method of pipe sizing is that the smaller systems are "over designed" to a larger degree than larger ones, yet the unit cost of piping is still increased from one scenario to the next.

Table 4.5: Effect of Estimation Method on Total Pipeline Costs per m^3 of 50 Year Flow

Case #	Clark	RSMeans	Industry
	($/$m^3$)	($/$m^3$)	($/$m^3$)
1	$0.0049	$0.0072	$0.0093
2	$0.0075	$0.0101	$0.0150
3	$0.0093	$0.0121	$0.0186
4	$0.0099	$0.0126	$0.0186
5	$0.0128	$0.0154	$0.0215
6	$0.0152	$0.0182	$0.0240

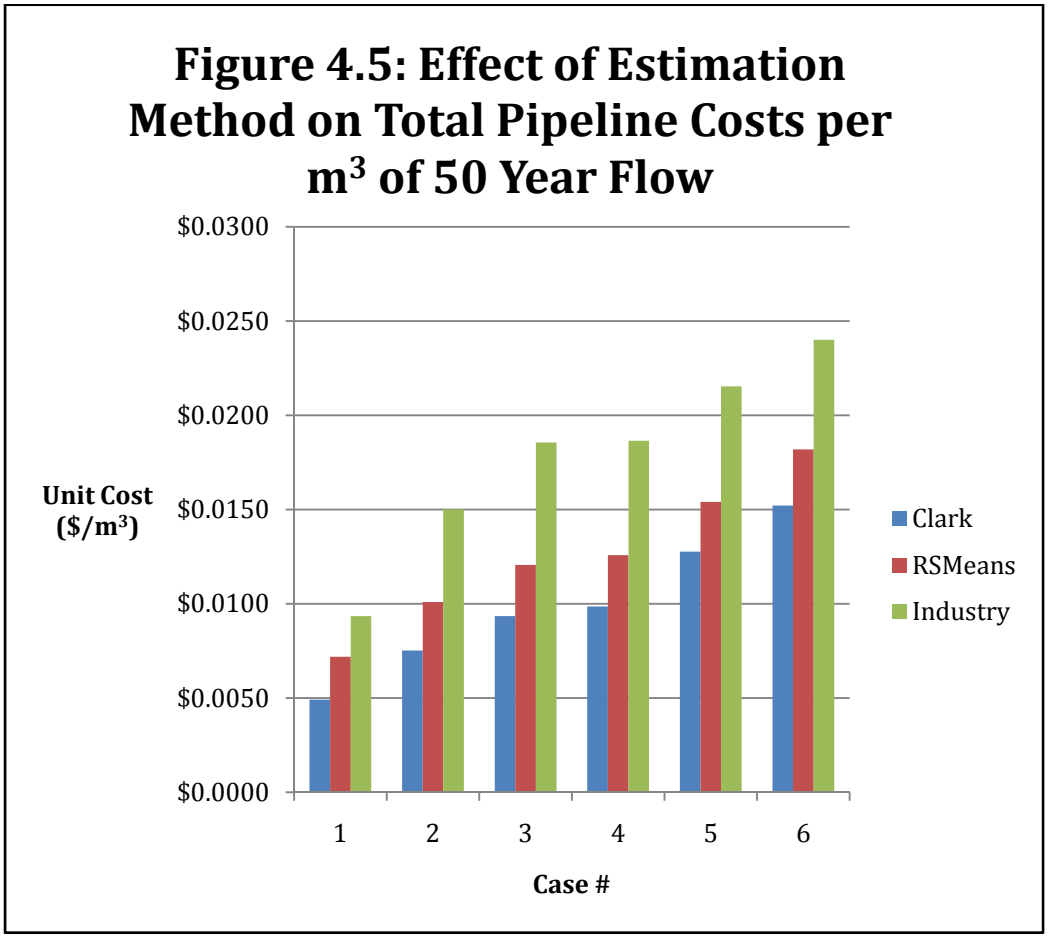

Figure 4.5: Effect of Estimation Method on Total Pipeline Costs per m^3 of 50 Year Flow

Table 4.6: Pipe Density			
Case #	Area Served	Total Pipeline Length	Pipe length per km^2
	(km^2)	(km)	(km/km^2)
1	36	22	0.61
2	72	48	0.67
3	144	90	0.63
4	292	178	0.61
5	576	330	0.57
6	1,152	676	0.59

Table 4.7: Fire Flow Requirements						
Case #	Population in thousands	Normal flow	Fire flow	Fire flow	Fire flow	Ratio of normal flow/ fire flow
	(capita / 1000)	(L/min)	(L/min)	(L/s)	(MGD)	
1	48.6	25,551.5	25,038.1	417.3	9.5	1.0
2	97.2	51,103.1	34,310.1	571.8	13.1	1.5
3	194.4	102,206.1	46,323.5	772.1	17.6	2.2
4	388.8	204,412.2	61,114.9	1,018.6	23.2	3.3
5	777.6	408,824.5	77,636.7	1,293.9	29.5	5.3
6	1,555.2	817,648.9	92,209.1	1,536.8	35.1	8.9

From the data presented above, it is evident that the cost of distribution piping infrastructure can significantly increase with the size of a water supply system. As previously mentioned, this trend is likely due to the need for costly large diameter pipes. However, there are a number of factors that could potentially amplify or reduce this trend. For example, the unit cost estimates for distribution piping in this study are the same regardless of the amount of piping being installed. In practical application, increases in the amount of work being done would result in a decreased unit cost similar to the trend seen in DWTP construction. In addition, the cost of primary and booster pump construction is not considered in this study, and the same "economies of scale" principal would likely apply to pump station construction. Conversely, the trend seen in Figure 4.5 could potentially be amplified when the cost of pipe fittings and valves are considered as the cost of these items increases exponentially as diameter increases.

4.3 O & M Costs

While the initial capital costs of a water supply system can be significant, the O & M costs must also be considered to select the most cost effective design. The O & M costs considered in this research effort include DWTP O & M and distribution pumping energy requirements, while distribution system maintenance/repair is not considered.

4.3.1 DWTP O & M Costs

The estimated DWTP O & M costs are presented in Table 4.8 and Figure 4.6.

Table 4.8: Annual DWTP O & M Costs

Case #	Flow	DWTP O&M Cost
	(MGD)	($)
1	9.72	$856,716
2	19.44	$1,050,801
3	38.88	$1,288,855
4	77.76	$1,580,838
5	155.52	$1,938,969
6	311.04	$2,378,232

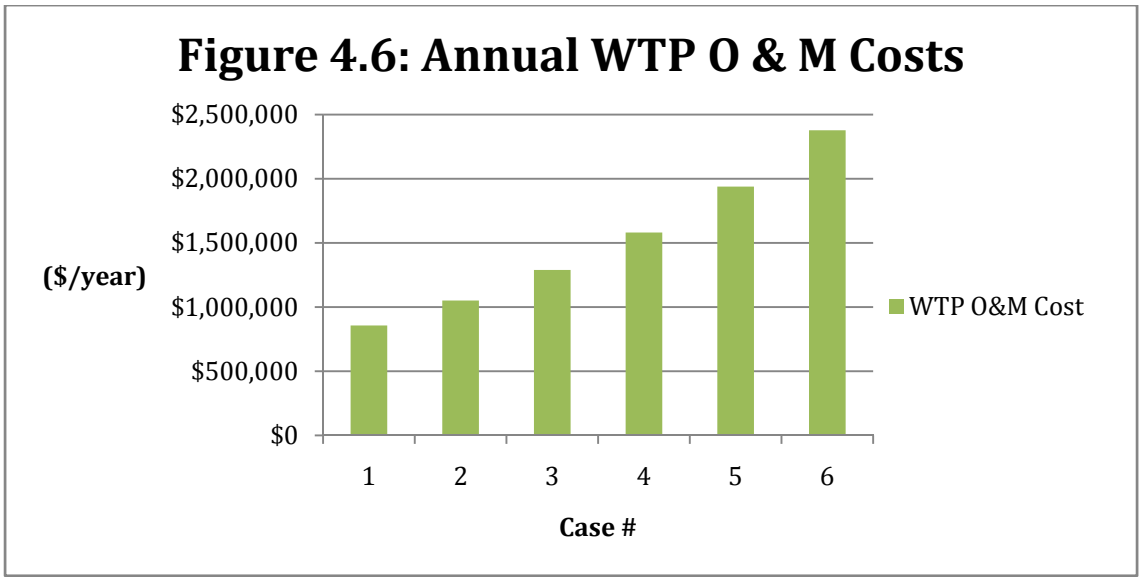

Figure 4.6: Annual DWTP O & M Costs

As with DWTP capital costs, the economies of scale provide that while the relative size of each scenario is doubled, the DWTP O & M costs are increased by only 22.7% from one scenario to the next. This concept is evident when the DWTP O & M costs are converted to unit costs in Table 4.9 and Figure 4.7.

Table 4.9: WTP O & M Costs per m^3 of Daily Flow

Case #	Flow	WTP Capital Cost
	(m^3/d)	($/$m^3$)
1	36,794.20	$23.28
2	73,588.40	$14.28
3	147,176.81	$8.76
4	294,353.62	$5.37
5	588,707.24	$3.29
6	1,177,414.47	$2.02

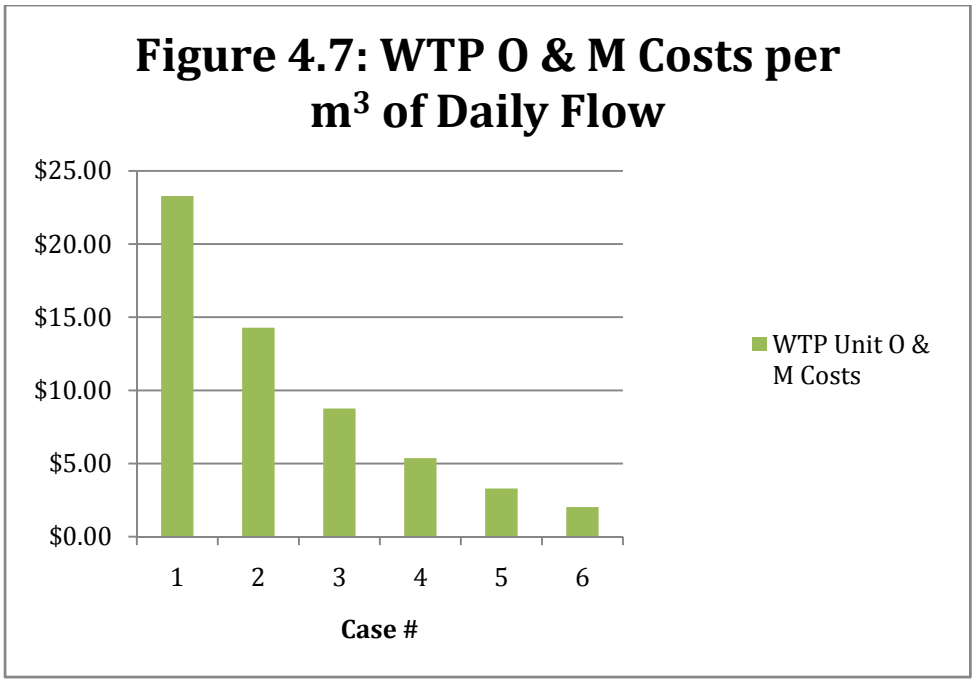

Figure 4.7: WTP O & M Costs per m^3 of Daily Flow

4.3.2 Distribution Pumping-Energy Costs

Annual distribution pumping costs for each scenario are based on a unit energy cost of $0.20/kWh and the results are presented in Table 4.10 and Figure 4.8.

Table 4.10: Distribution System Pumping Costs

Case #	Pumping Costs	Unit Pumping Costs
	($/year)	($/$m^3$)
1	$409,654.10	$0.03050
2	$840,463.60	$0.03129
3	$1,935,456.30	$0.03603
4	$4,279,099.40	$0.03983
5	$9,552,020.80	$0.04445
6	$20,572,976.80	$0.04787

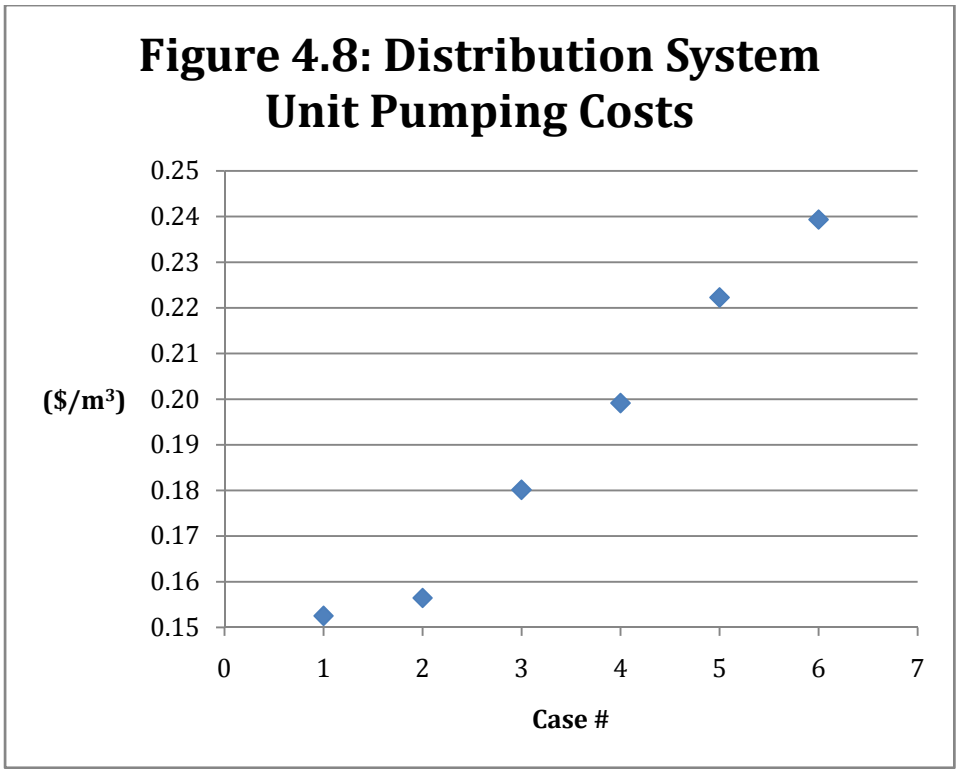

Figure 4.8: Distribution System Unit Pumping Costs

It is evident from Figure 4.8 that the unit cost of pumping energy is significantly increased from one scenario to the next. However, in practical applications this trend could reduce or amplified by a number of factors. For example, the energy costs used in these estimates is assumed to remain constant, but the actual cost of energy can fluctuate according to the time of day to discourage consumption during peak energy demand. As electricity is not easily stored, energy providers often employ this type of tiered price structure to curb peak energy demands and encourage consumption during times of decreased energy demand. In addition, peak energy demand and peak water demand often coincide, and a large portion of booster pump operation would likely incur increased energy prices. According to Boulos and Bros (2010) peak demand charges can account for up to 20% of municipal energy costs. As booster pump utilization is

increased in the larger scenarios of this study, this factor could significantly increase the cost of distribution system pumping presented in the figures above.

4.4 Life Cycle Costs

Life cycle cost analysis compares the total cost of a product or system over the entire life cycle by converting all values to present value terms. The LCC comparison presented in Table 4.11 is formed using the following assumptions: the annual discount rate is 7%, the unit energy cost is $0.20/kWh, and the Clark method is used for pipeline cost estimation. The values presented will be used as the base scenario for the sensitivity analysis later in this chapter. By converting the annual O & M and pumping costs into present value terms the LCCs of each scenario can be readily compared. The LCC results are presented in Table 4.11, Figure 4.9: LCCs, and Figure 4.10.

Table 4.11: Summary of LCCs (Millions of Dollars)

Case #	1	2	3	4	5	6
DWTP O & M	$11.82	$14.50	$17.79	$21.82	$26.76	$32.82
Pumping	$5.65	$11.60	$26.71	$59.05	$131.83	$283.92
Pipelines	$3.30	$10.10	$25.10	$52.97	$137.14	$326.83
DWTP Capital	$33.98	$51.36	$77.61	$117.29	$177.24	$267.85
Total Cost	$54.76	$87.56	$147.20	$251.12	$472.97	$911.42

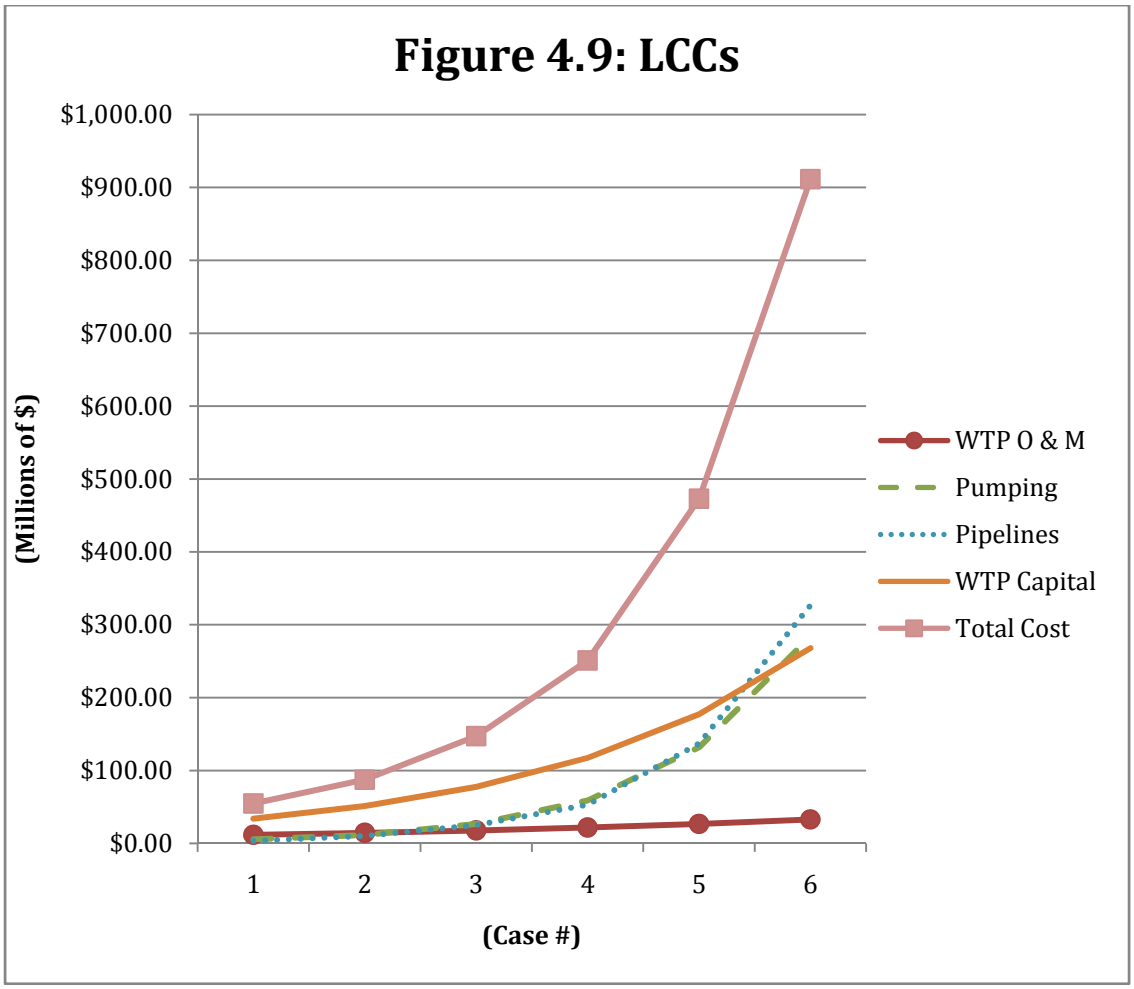

Figure 4.9: LCCs

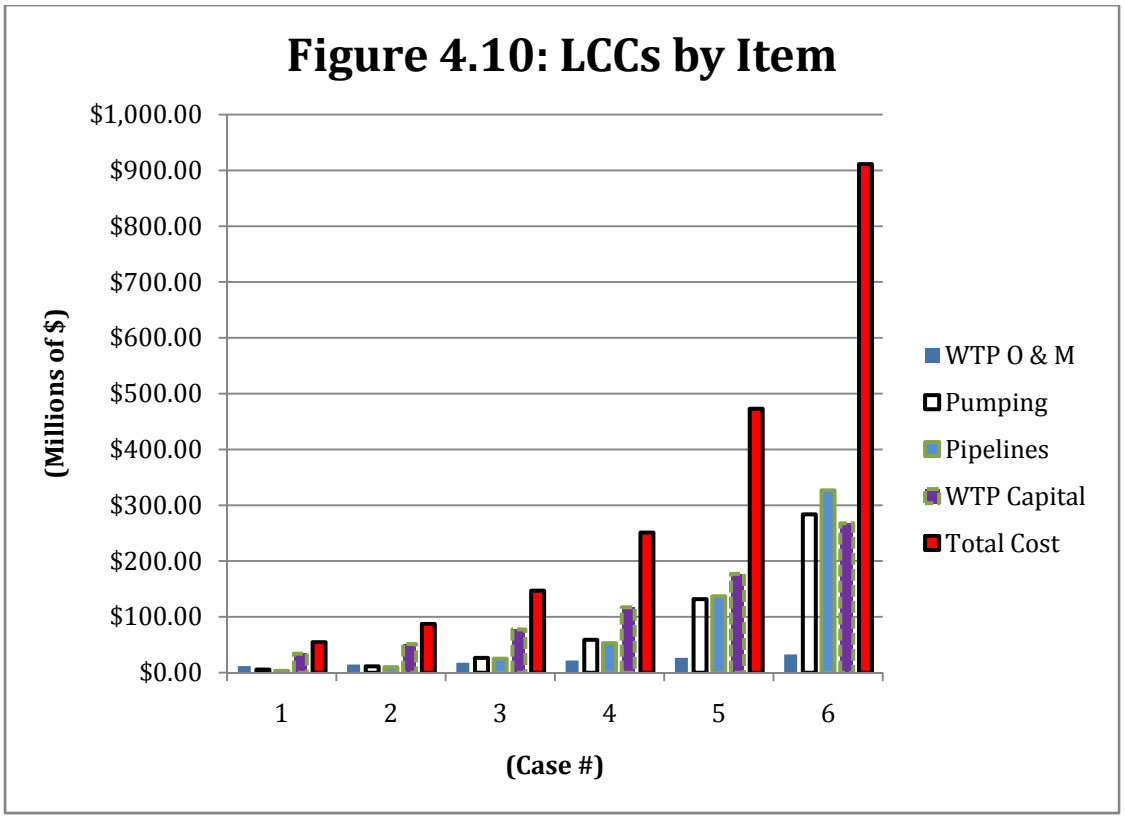

Figure 4.10: LCCs by Item

As the scenarios in this study vary in size, unit LCCs are particularly useful for comparison of fiscal efficiency. The unit LCCs per m^3 of 50 year flow are presented in Table 4.12, Figure 4.11, and Figure 4.12.

Table 4.12: Summary of LCCs per m^3 of 50 Year Flow						
Case #	1	2	3	4	5	6
Item	($/$m^3$)	($/$m^3$)	($/$m^3$)	($/$m^3$)	($/$m^3$)	($/$m^3$)
WTP O & M	$0.0176	$0.0108	$0.0066	$0.0041	$0.0025	$0.0015
Pumping	$0.0084	$0.0086	$0.0099	$0.0110	$0.0123	$0.0132
Pipeline	$0.0049	$0.0075	$0.0093	$0.0099	$0.0128	$0.0152
WTP Capital	$0.0506	$0.0382	$0.0289	$0.0218	$0.0165	$0.0125
Total Cost	$0.0816	$0.0652	$0.0548	$0.0467	$0.0440	$0.0424

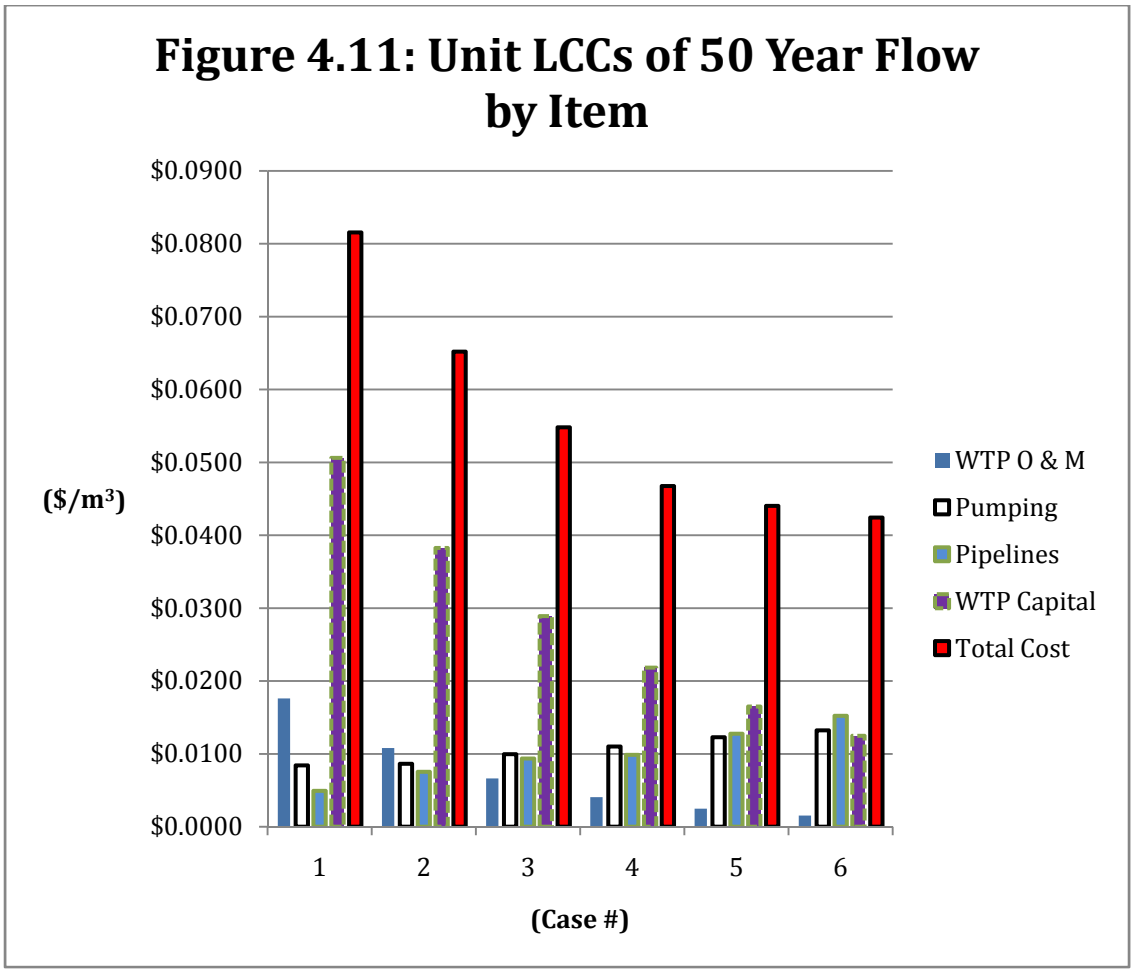

Figure 4.11: Unit LCCs of 50 Year Flow by Item

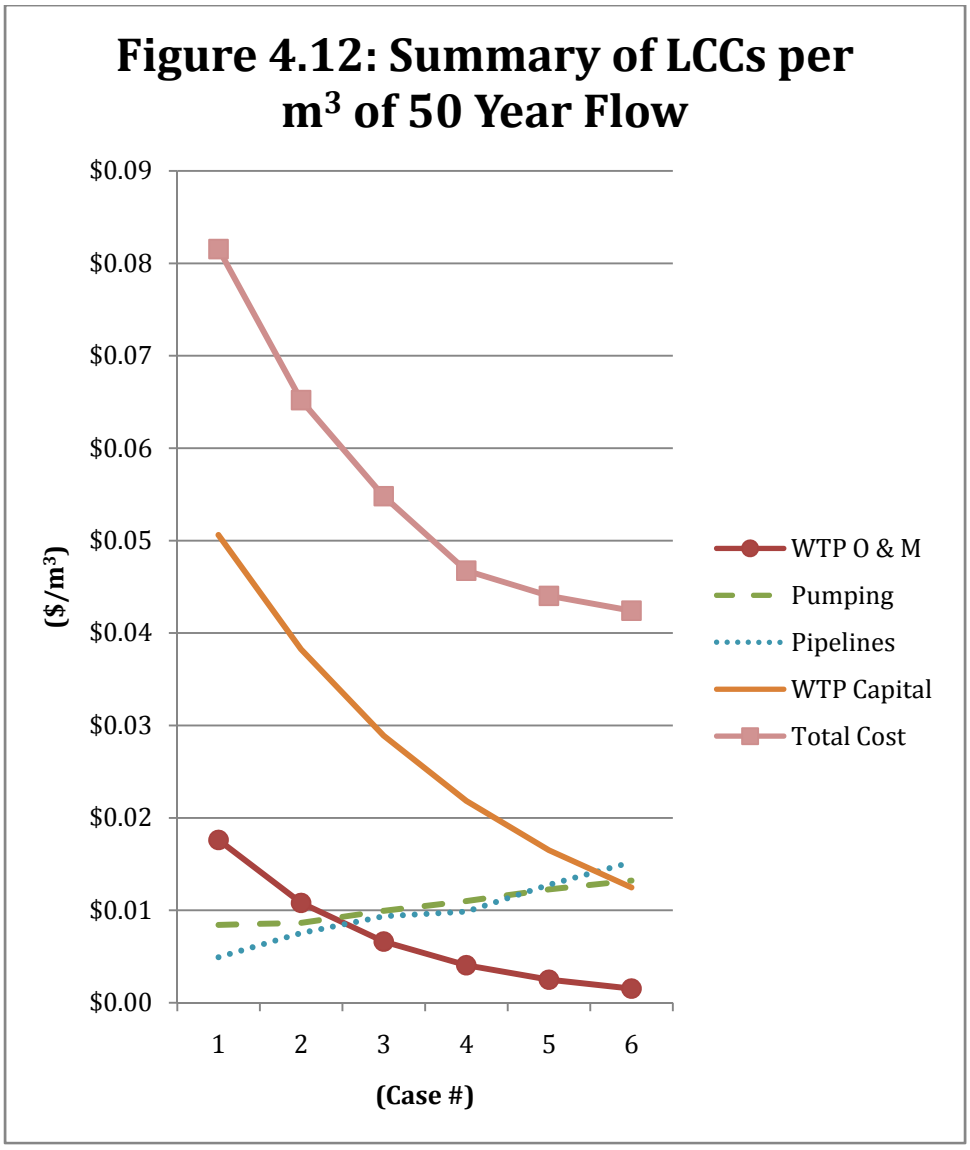

Figure 4.12: Summary of LCCs per m^3 of 50 Year Flow

The LCC data presented in Figure 4.12 provides a clear understanding of the unit cost trends as system size increases. As expected, there is a conflicting cost dynamic where cost savings from the DWTP are partially negated by increasing costs associated with the distribution system. However, the rate of distribution cost increases is dwarfed by substantial DWTP savings and the result is an overall decrease in the total unit cost for water supply. To better understand the relative contributions of each item to the unit

LCCs, the LCCs for each item are expressed as a percentage of the total unit cost in Table 4.13 and Figure 4.13.

Table 4.13: LCC % Contributions by Item				
Case #	DWTP O & M	Pumping	Pipelines	DWTP Capital
1	21.59%	10.32%	6.03%	62.06%
2	16.56%	13.25%	11.54%	58.65%
3	12.08%	18.15%	17.05%	52.72%
4	8.69%	23.52%	21.09%	46.70%
5	5.66%	27.87%	29.00%	37.47%
6	3.60%	31.15%	35.86%	29.39%

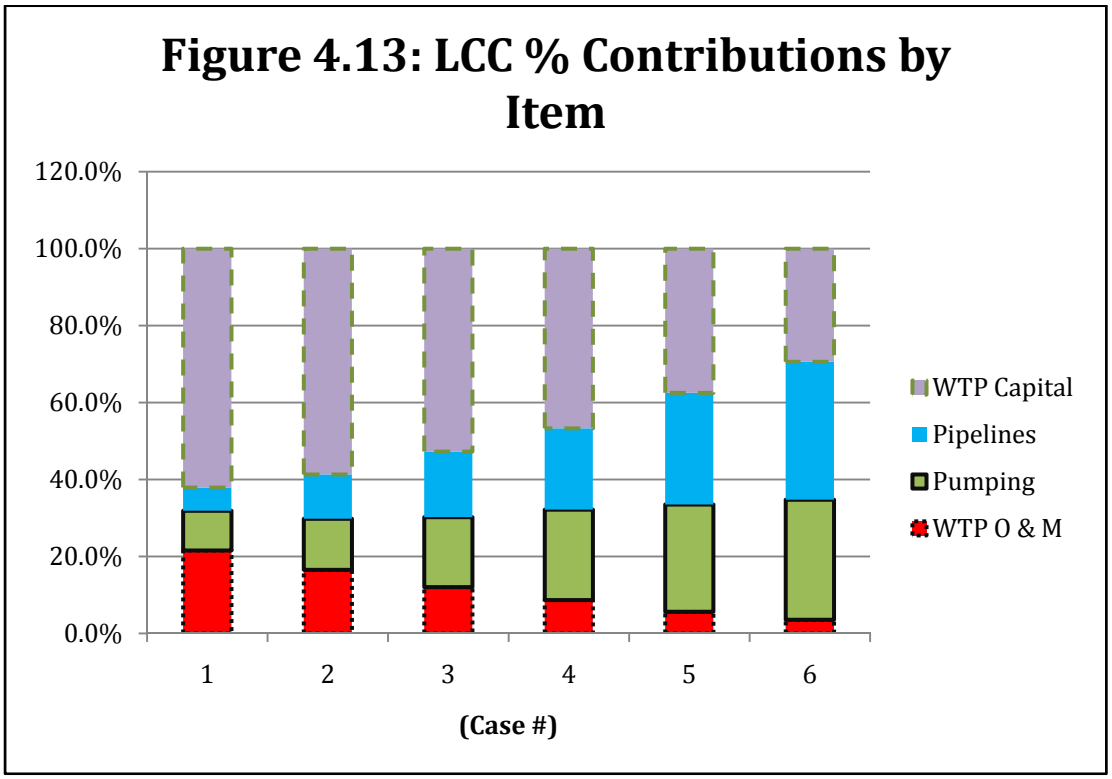

Figure 4.13: LCC % Contributions by Item

Figure 4.13: LCC % Contributions by Item illustrates the concept that for smaller systems the costs associated with water treatment dominate, however; as system size increases, the relative contribution of distribution becomes increasingly significant.

4.5 Life Cycle Assessment

While fiscal costs are an important consideration in the water supply industry, a growing concern for the environment demands the consideration of the environmental effects of a design. In this study, energy consumption serves as an environmental performance indicator and the environmental impacts are quantified for each kWh of electricity as illustrated in Table 4.14.

Table 4.14: LCIs per Unit of Energy (1 kWh 'Electricity, high voltage, at grid/US U')

Software:	SimaPro 7.2	
Method:	TRACI 2 V3.01	
Impact category	Value	Unit
Global Warming	7.56E-01	(kg CO2 eq)
Acidification	2.69E-01	(H+ moles eq)
Carcinogenics	9.26E-04	(kg benzen eq)
Non carcinogenics	4.67E+00	(kg toluen eq)
Respiratory effects	1.42E-03	(kg PM2.5 eq)
Eutrophication	1.40E-04	(kg N eq)
Ozone depletion	2.01E-08	(kg CFC-11 eq)
Ecotoxicity	1.52E+00	(kg 2,4-D eq)
Smog	1.46E-03	(kg NOx eq)

As the energy consumed by distribution pumps is the only source of environmental impacts in this study, the impacts for all categories are directly

proportional to the energy consumption of each scenario. This concept is illustrated in Figure 4.14.

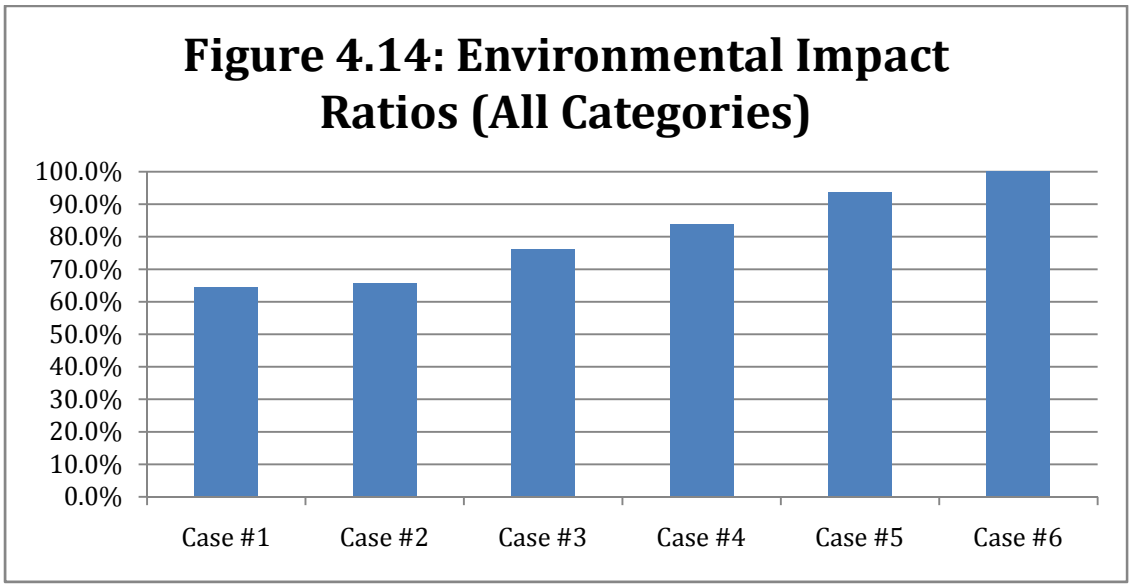

Figure 4.14: Environmental Impact Ratios (All Categories)

In regard to global warming potential, case #6 produced 0.179 kg of carbon dioxide equivalents per m^3 of water produced. A similar LCA performed in South Africa (Friedrich, Pillay et al. 2007) reported that drinking water distribution accounted for 0.195 kg of carbon dioxide equivalents per m^3 of water produced. Although, the LCA methods used are not identical, the results of this study seem reasonable in comparison.

4.6 Sensitivity Analysis

A sensitivity analysis will be performed to determine what factors have the strongest influence over the LCC results. The base scenario presented for analysis includes the following assumptions: the annual discount rate is 7%, the unit energy cost is $0.20/kWh, and the Clark method is used for pipeline cost estimation. The sensitivity

analysis will explore the effect of the following assumption variations: the annual discount rate is 3%, the annual discount rate is 11%, the unit energy cost is $0.10/kWh, the unit energy cost is $0.40/kWh, the unit energy cost is $0.80/kWh, the industry method is used for pipeline cost estimation, the RSMeans method is used for pipeline cost estimation, and the RSMeans method is used for pipeline cost estimation where the estimated pipeline cost is increased by a factor of 50%. In addition, a scenario will be analyzed to simulate water quality degradation in the largest three scenarios where disinfection byproduct formation results in the need for advanced treatment. Where advanced treatment is required, a microfiltration membrane DWTP is modeled (McGivney and Kawamura 2008) where the DWTP capital costs are estimated by,

$$Y = 7.1052 * x^{0.8302}$$

Where, Y = Total Capital Costs (millions of U.S. dollars)

x = Design Flow (MGD)

The O & M costs for the advanced DWTP are calculated by,

$$Y = 0.4441 * x^{0.4323}$$

Where, Y = Annual O & M Costs (millions of U.S. dollars/year)

x = Design Flow (MGD)

The cost estimate for the advanced DWTP scenario is presented in Table 4.15. Note: Only cases #4-6 require advanced treatment. The results of the advanced DWTP cost estimates are presented in Table 4.15, Figure 4.15, and Figure 4.16.

Table 4.15: Advanced DWTP Cost Estimates			
Case #	Flow (MGD)	DWTP Capital Cost ($)	DWTP Unit O&M Cost ($)
1	9.72	$33,983,895	$856,716
2	19.44	$51,356,653	$1,050,801
3	38.88	$77,610,465	$1,288,855
4	77.76	$263,804,468	$2,916,451
5	155.52	$469,026,358	$3,935,410
6	311.04	$833,896,886	$5,310,376

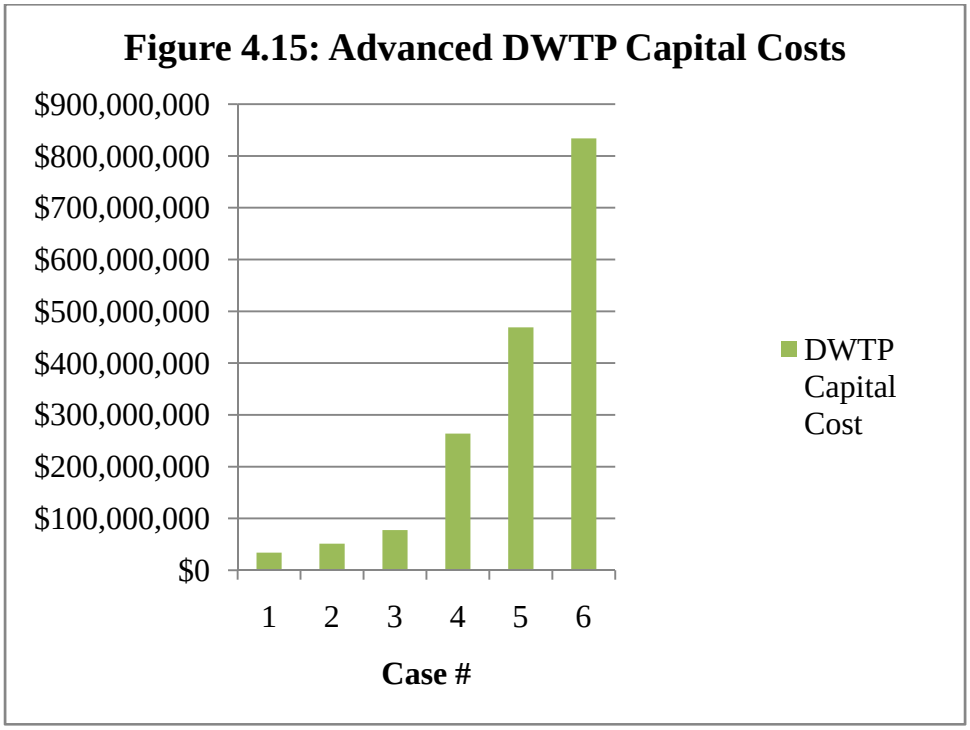

Figure 4.15: Advanced DWTP Capital Costs

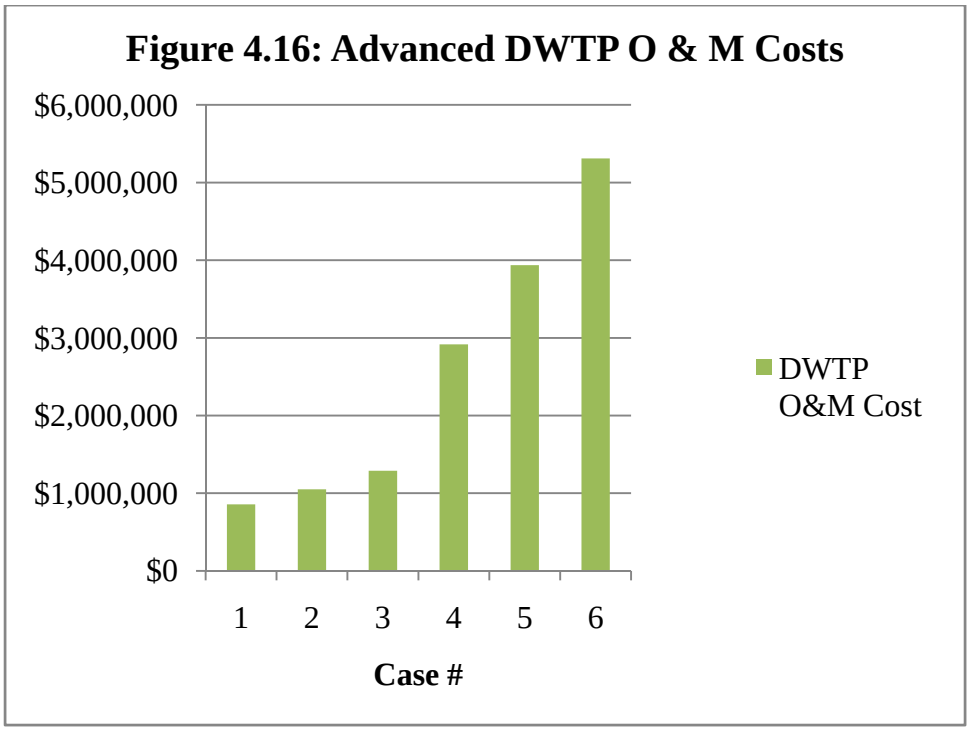

Figure 4.16: Advanced DWTP O & M Costs

The Advanced DWTP costs are displayed in terms of unit cost per m^3 of 50 year flow in Table 4.16, Figure 4.17, and Figure 4.18.

Table 4.16: Advanced DWTP Unit Cost Estimates

Case #	Flow (m^3/d)	DWTP Unit Capital Cost ($/$m^3$)	DWTP Unit O&M Cost ($/$m^3$)
1	36,794	$0.0506	$0.0013
2	73,588	$0.0382	$0.0008
3	147,177	$0.0289	$0.0005
4	294,354	$0.0491	$0.0005
5	588,707	$0.0437	$0.0004
6	1,177,414	$0.0388	$0.0002

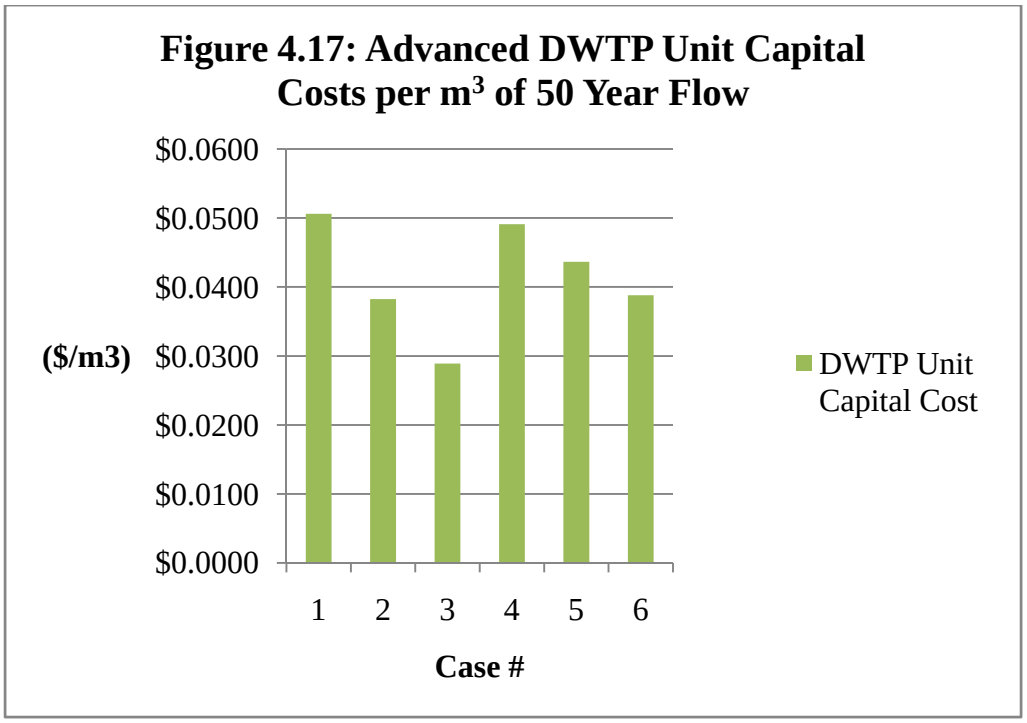

Figure 4.17: Advanced DWTP Unit Capital Costs per m^3 of 50 Year Flow

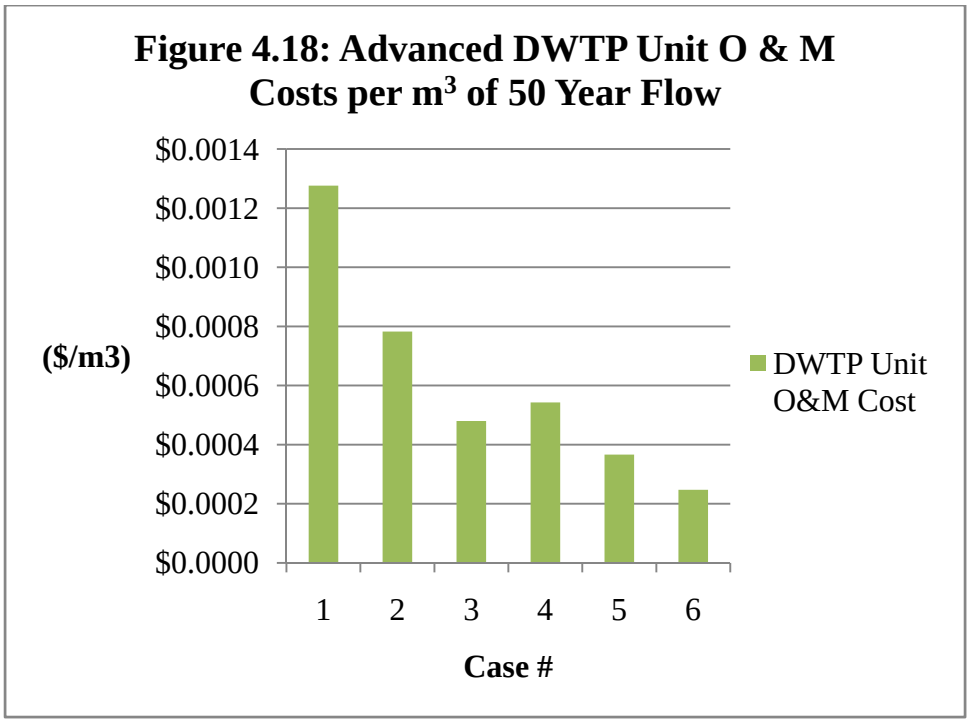

Figure 4.18: Advanced DWTP Unit O & M Costs per m^3 of 50 Year Flow

The LCC results for the advanced DWTP scenario are listed in Table 4.17.

Table 4.17: Final Results (Estimation Method Clark, Microfiltration Required for Case #s 4-6)

Case #	DWTP O & M (50 year current value) (Millions of $)	Pumping Cost (50 year current value) (Millions of $)	Total O & M (50 year current value) (Millions of $)	Total Pipe Cost (Millions of $)	DWTP Capital Cost (Millions of $)	Total Capital cost (Millions of $)	Total Capital + O & M (current value) (Millions of $)
1	$11.8	$5.7	$17.5	$3.3	$34.0	$37.3	$54.8
2	$14.5	$11.6	$26.1	$10.1	$51.4	$61.5	$87.6
3	$17.8	$26.7	$44.5	$25.1	$77.6	$102.7	$147.2
4	$40.2	$59.1	$99.3	$53.0	$263.8	$316.8	$416.1
5	$54.3	$131.8	$186.1	$137.1	$469.0	$606.2	$792.3
6	$73.3	$283.9	$357.2	$326.8	$833.9	$1,160.7	$1,517.9

The total cost results of the sensativity analysis are presented in Table 4.18, Table 4.19, and Figure 4.19.

Table 4.18: Total Cost Sensitivity Analysis						
Case #	1	2	3	4	5	6
Variable	(Millions of $)	(Millions of $)	(Millions of $)	(Millions of $)	(Millions of $)	(Millions of $)
Base	$54.76	$87.56	$147.20	$251.12	$472.97	$911.42
$0.10/kWh	$51.93	$81.76	$133.85	$221.60	$407.05	$769.46
$0.40/kWh	$60.41	$99.16	$173.91	$310.18	$604.79	$1,195.35
$0.80/kWh	$71.72	$122.36	$227.34	$428.29	$868.44	$1,763.19
i=3%	$69.87	$110.12	$185.67	$321.03	$610.04	$1,185.21
i=11%	$48.73	$78.56	$131.86	$223.24	$418.28	$802.20
RSMeans	$56.29	$91.02	$154.51	$265.75	$501.37	$975.42
Industry	$57.73	$97.62	$171.96	$298.34	$567.18	$1,100.39
RSMeans - 50% increase	$58.71	$97.80	$170.71	$299.55	$584.14	$1,170.83
#4-6 Adv. DWTP	$54.76	$87.56	$147.20	$416.07	$792.30	$1,517.94

Table 4.19: Unit Cost Sensitivity Analysis						
Case #	1	2	3	4	5	6
Variable	($/m^3)	($/m^3)	($/m^3)	($/m^3)	($/m3)	($/m3)
Base	$0.0816	$0.0652	$0.0548	$0.0467	$0.0440	$0.0424
$0.10/kWh	$0.0773	$0.0609	$0.0498	$0.0413	$0.0379	$0.0358
$0.40/kWh	$0.0900	$0.0738	$0.0647	$0.0577	$0.0563	$0.0556
$0.80/kWh	$0.1068	$0.0911	$0.0846	$0.0797	$0.0808	$0.0821
i=3%	$0.1040	$0.0820	$0.0691	$0.0598	$0.0568	$0.0552
i=11%	$0.0726	$0.0585	$0.0491	$0.0416	$0.0389	$0.0373
RSMeans	$0.0838	$0.0678	$0.0575	$0.0495	$0.0467	$0.0454
Industry	$0.0860	$0.0727	$0.0640	$0.0555	$0.0528	$0.0512
RSMeans - 50% increase	$0.0874	$0.0728	$0.0636	$0.0558	$0.0544	$0.0545
#4-6 Adv. DWTP	$0.0816	$0.0652	$0.0548	$0.0775	$0.0737	$0.0706

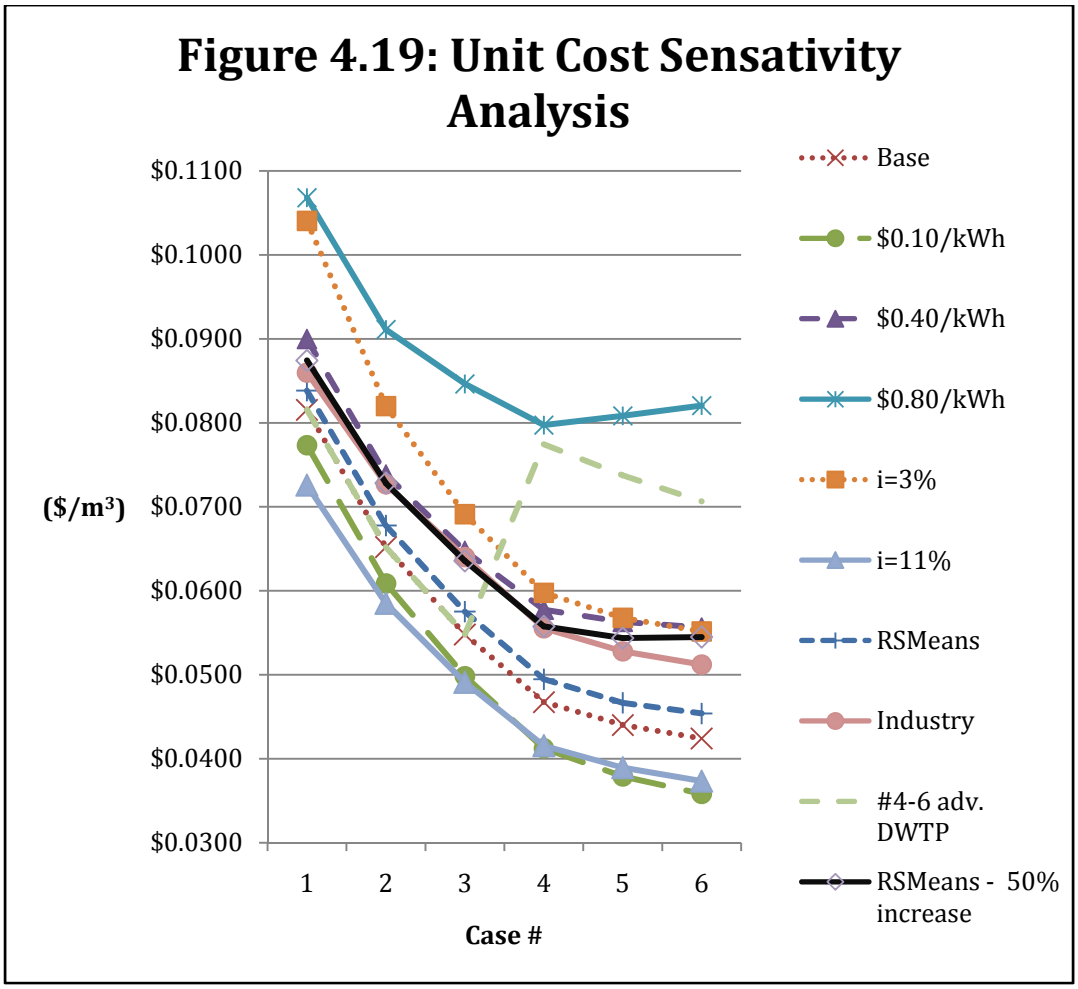

Figure 4.19: Unit Cost Sensitivity Analysis

The effect of each parameter variation can be seen in Figure 4.19. The unit cost for each scenario is reduced when energy prices are lowered or the annual discount rate is increased, while the unit cost is increased when energy prices are raised, alternate pipeline estimation methods are used, the annual discount rate is reduced, or advanced treatment is required. Although, the unit cost trends are changed for each parameter variation, only the extreme cases of increased energy cost, increased distribution piping costs, and advanced treatment required reverse the downward unit cost trend to suggest

that there may be a breakpoint where the unit cost will begin to increase with size. Accordingly, these extreme situations are discussed in the following paragraphs.

It is unlikely that energy prices will reach $0.80/kWh in the near future, however; it is possible that situations where the population density is sparse or topographic changes are significant could produce results that mimic this trend. Future research efforts should involve the effect of these factors to investigate their effects.

It was the original intention to build on the work of Weber and Norton (2008) where DBP formation was included as a limiting factor. Unfortunately, the hydraulic retention times in each scenario studied varied by less than 24 hours and the effect of DBP formation was negligible. However, in practical application large water supply systems have the potential to form excessive DBPs and the scenario involving advanced water treatment is intended to represent this situation. The effect of advanced water treatment was found to have the most significant effect on the LCC analysis of this work. Accordingly, future research should address this concept through a more appropriate means of relating system size to DBP formation.

When a LCC analysis is performed with the RSMeans method of pipeline estimation and the assumption is made that the cost of each pipelines is increased by 50%; the unit cost curve turns upward slightly from cases #5 to 6. Although this assumption is a bit extreme, it is obvious that actual distribution piping would be increased significantly when valves, fittings, and service lines are included in the estimate. Therefore, this scenario may be a valid one and future research should investigate it more thoroughly.

CHAPTER 5

CONCLUSIONS

5.1 Energy and Environmental Impacts

This study showed that as water supply system size increased, pumping energy requirements were significantly increased. The most probable explanation for this trend is that the longer runs of distribution piping required to deliver water to the system periphery result in increased friction losses that necessitate additional energy input. This effect would likely be amplified if service lines were also considered. In addition, the effect of local topography is not considered in this effort and situations with topographic variation could significantly increase the pumping-energy required.

As the environmental impacts of this study are directly related to energy consumption, smaller water supply systems were found to be more environmentally friendly than larger ones. However, in a more comprehensive LCA, the environmental effects of DWTP operation could prove significant.

5.2 Life Cycle Costs

This study found that the cost savings incurred from the economies of scale of DWTP construction were significant and dominate the unit cost of smaller systems. For large systems the capital cost of distribution system piping and operational cost of pumping-energy become increasingly important, however; the cost savings incurred from DWTP construction provide that the overall unit cost was progressively decreased for all six scenarios studied.

REFERENCES

Adams, C. D. and J. Sutherland (2000). Applicability of ozone in small drinking water systems. Proceedings of the ASCE National Conference on Environmental and Pipeline Engineering 2000, July 23, 2000 - July 26, 2000, Kansas City, MO, United states, American Society of Civil Engineers.

Arnal, J. M., B. García-Fayos, et al. (2009). "Design and installation of a decentralized drinking water system based on ultrafiltration in Mozambique." Desalination **250**(2): 613-617.

Arpke, A. and N. Hutzler (2006). "Domestic Water Use in the United States: A Life-Cycle Approach." Journal of Industrial Ecology **10**(1-2): 169-184.

Bichai, F. and B. Barbeau (2006). "Assessing the disinfecting power of chlorite in drinking water." Water Quality Research Journal of Canada **41**(Compendex): 375-382.

Bolto, B., D. Dixon, et al. (2002). "Removal of natural organic matter by ion exchange." Water Research **36**(20): 5057-5065.

Boulos, P. F. and C. M. Bros (2010). "assessing the carbon footprint of water supply and distribution systems." AWWA **102**(11): 47-54.

Casson, L. W. and G. L. Hunter (2008). "Disinfection Alternatives and Sustainability: Optimization of Energy Usage, Disinfection Effectiveness and Sustainability." Proceedings of the Water Environment Federation **2008**: 7739-7749.

Chae, S. and C. Wang (2002). Formation characterization and removal of disinfection by-products in water treatment processes, IWA Publishing.

Chiang, P. C., E. E. Chang, et al. (2002). "NOM characteristics and treatabilities of ozonation processes." Chemosphere **46**(6): 929-936.

Chung, G., K. Lansey, et al. (2008). "A general water supply planning model: Evaluation of decentralized treatment." Environmental Modelling & Software **23**(7): 893-905.

Clark, R. M. and M. Sivaganesan (2002). "Predicting Chlorine Residuals in Drinking Water: Second Order Model." Journal of Water Resources Planning and Management **128**(2): 152-161.

Clark, R. M., M. Sivaganesan, et al. (2002). "Cost models for water supply distribution systems." Journal of Water Resources Planning and Management-Asce **128**(5): 312-321.

Das, T. (2002). "Evaluating the life cycle environmental performance of chlorine disinfection and ultraviolet technologies." Clean Technologies and Environmental Policy **4**(1): 32-43.

Diamant, B. Z. (1996). "Water treatment: The ultra-violet option." International Water and Irrigation **16**(Compendex).

DOE, U. S. (2006). Energy Demands on Water Resources: Repot to Congress on the Interdependency of Energy and Water. U. S. DOE.

EPA, U. S. (1998). National Primary Drinking Water Regulations: Disinfectants and Disinfection Byproducts; Final Rule.

EPA, U. S. (2001). Controlling Disinfection By-Products and Microbial Contaminants in Drinking Water. U. S. EPA.

EPA, U. S. (2005). Technologies and Costs Document for the Final Long Term 2 Enhanced Surface Water Treatment Rule and Final Stage 2 Disinfectants and Disinfection Byproducts Rule.

Friedrich, E. (2002). Life-cycle assessment as an environmental management tool in the production of potable water, IWA Publishing.

Friedrich, E., S. Pillay, et al. (2007). "The use of LCA in the water industry and the case for an environmental performance indicator." Water SA **33**(4): 443-452.

Herz, R. K. and A. Lipkow (2002). "Life cycle assessment of water mains and sewers." Water Science & Technology: Water Supply **2**(4): 51.

Hijnen, W. A. M. and G. J. Medema (2005). Inactivation of viruses, bacteria, spores and protozoa by ultraviolet irradiation in drinking water practice: A review, IWA Publishing.

Hong, S. and M. Elimelech (1997). "Chemical and physical aspects of natural organic matter (NOM) fouling of nanofiltration membranes." Journal of Membrane Science **132**(2): 159-181.

Kerwick, M., S. Reddy, et al. (2005). "A methodology for the evaluation of disinfection technologies." Journal of Water and Health **3**: 393-403.

Less, J. R. (2011). Comparison of Natural Organic Matter (NOM) Removal Processes on Disinfection Byproduct (DBP) Formation During Drinking Water Treatment, Akron University.

Lundie, S., G. M. Peters, et al. (2004). "Life Cycle Assessment for Sustainable Metropolitan Water Systems Planning." Environmental Science & Technology **38**(13): 3465-3473.

Managers, W. S. C. o. t. G. L.-.-U. M. R. B. o. S. a. P. P. H. a. E. (2007). Recommended Standards for Water Works, Albany, NY.

McGivney, W. and S. Kawamura (2008). Cost Estimating Manual for Water Treatment Facilities, John R. Wiley & Sons.

Norton, J. J. W. and J. W. J. Weber (2006). "Breakeven costs for distributed advanced technology water-treatment systems." Water Research **40**(19): 3541-3550.

Peter-Varbanets, M., C. Zurbrugg, et al. (2009). "Decentralized systems for potable water and the potential of membrane technology." Water Research **43**(Compendex): 245-265.

Raluy, R. G., L. Serra, et al. (2005). "Life Cycle Assessment of Water Production Technologies - Part 2: Reverse Osmosis Desalination versus the Ebro River Water Transfer (9 pp)." The International Journal of Life Cycle Assessment **10**(5): 346-354.

Rao, P. (2011). Impacts of urban form on centralized and distributed water distribution networks Faculty of Engineering. Athens, GA, The University of Georgia. **M.S. Environmental Engineering**.

Rossman, L. A. (2000). EPANET Users Manual. U. S. E.-W. S. a. W. R. Division.

Sadiq, R. and M. J. Rodriguez (2004). "Disinfection by-products (DBPs) in drinking water and predictive models for their occurrence: A review." Science of the Total Environment **321**(Compendex): 21-46.

Shammas, N. and L. Wang (2011). Water Supply and Wastewater Removal, John Wiley & Sons, Inc.

Shaw, M. (2002). "Life cycle costs reduced with medium pressure UV." Water Services **105**(Compendex): 18-19.

Spencer, E. (2007). RSMeans Heavy Construction Cost Data, Reed Construction Data.

Stokes, J. and A. Horvath (2006). "Life Cycle Energy Assessment of Alternative Water Supply Systems (9 pp)." The International Journal of Life Cycle Assessment **11**(5): 335-343.

Trussell, R. R. (1999). "An overview of disinfectant residuals in drinking water distribution systems." Aqua **48**(Compendex): 2-10.

Van der Bruggen, B. and C. Vandecasteele (2003). "Removal of pollutants from surface water and groundwater by nanofiltration: overview of possible applications in the drinking water industry." Environmental Pollution **122**(3): 435-445.

Vince, F., E. Aoustin, et al. (2008). "LCA tool for the environmental evaluation of potable water production." Desalination **220**(1-3): 37-56.

Weber Jr, W. (2004). Optimal uses of advanced technologies for water and wastewater treatment in urban environments. London, ROYAUME-UNI, IWA Publishing.

Weber Jr, W. and J. Norton (2008). Financial and Technological Analysis of Water Treatment Technology Implementation Using Distributed Optimal Technology Network (DOT Net) Concepts.

Weber Jr, W. J. (2002). Distributed optimal technology networks: A concept and strategy for potable water sustainability, IWA Publishing.