ABSTRACT

Title of Document: **A PROBABILISTIC ESTIMATION MODEL FOR THE RECOVERABLE LEAKAGE OF WATER DISTRIBUTION NETWORKS**

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Water Distribution Networks (WDN) play a vitally important role in preserving and providing a desirable life quality to the public. A WDN should provide, during its economic life, the required quality and quantity of water at required pressures.

Leakage rate and its high associated cost of failure have reached a level that now draws the attention of both policy and decision makers. Leakage is usually the major cause of water loss in water distribution systems. EPA reported in 2007, 240,000 water main breaks per year in the US. The USGS in 2007 estimated that water lost from water distribution systems is 1.7 trillion gallons per year at a national cost of $2.6 billion per year.

Leakage occurs in different components of the water distribution system. Causes of leaks include corrosion, soil corrosivity, excessive water pressure, material defects, water hammer, excessive loads and vibration from road traffic and stray electric current.

In this dissertation a probabilistic estimation model for the recoverable leakage of WDNs was presented factoring key causes that lead to high percentages of leakage in different components of the WDN. The model receives the deterministic and stochastic description of the leakage of the WDN received from the research survey. It is evident that IWA's model for estimating Unavoidable Annual Real Losses (UARL) does not account

for soil corrosivity. The UARL equation can be modified by adding a new soil corrosivity factor (Cr) that takes the soil corrosivity into consideration.

Linear Regression was used to develop a relationship between the UARL and the soil corrosivity. Directional cosines analysis examined the importance of the random variables in the new probabilistic estimation model. Two Case Studies were used to validate the modified formulation for the UARL using the data for the leakage component parameters and the system water audit.

Monte Carlo simulation was operated twice till the distribution had minimal change. After adding the Cr the output distributions for the UARL had a 43% decrease in the standard deviation value which shows that the corrosion behavior of WDNs is closely related to the environmental factors.

A PROBABILISTIC ESTIMATION MODEL FOR THE RECOVERABLE LEAKAGE OF WATER DISTRIBUTION NETWORKS

By

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Acronyms

AC	Alternating Current
AMI	Advanced Metering Infrastructure
AMR	Automatic Meter Reading
AWWA	American Water Works Association
BMP	Best Management Practice
BMWD	Baltimore Metropolitan Water District
CARL	Current Annual Volume of Real Losses
COB	City of Baltimore
Cr	Soil Corrisivity Factor
ELL	Economic Level of Leakage
EPA	Environment Protection Agency
FY	Fiscal Year
ILI	Infrastructure Leak Index
IWA	International Water Association
JMT	Johnson, Mirimiran and Thompson
MG	Million Gallons
MGD	Million Gallons per Day
NRW	Non-Revenue Water
^{o}F	Degree Fahrenheit
P	Pressure
pH	Power of Hydrogen
PRV	Pressure Reducing Valve
psi	Pounds per square inch
PWS	Public Water Systems
RCM	Reliability Centered Maintenance
SR	Soil Resistivity
UARL	Unavoidable Annual Real Losses
US	United States

WDN	Water Distribution Network
WSSC	Washington Suburban Sanitary Commission

Chapter 1. Introduction

1.1. Water Distribution Networks

Water Distribution Networks play an important role in preserving and providing an appropriate life quality to society. The design of water distribution networks has been driven by the minimization of cost. The amount of effort and attention given to developing procedures for system performance reliability evaluation has been limited.

Water utilities construct, operate, and maintain water supply systems. The basic objective of these water utilities is to obtain water from a source, treat the water to an acceptable quality, and deliver the appropriate quantity of water to the place of need at the required time. The analysis of a water system is usually for one or more of the major functional components of the utility: source development; raw-water transmission, raw water storage, treatment, finished water storage; and finished water distribution. The water distribution network is our main focus in this dissertation.

Providing communities with reliable and safe water has increasingly become a topic of concern. Water Distribution networks are buried underground and, as a result, they have received misappropriated attention from decision makers. Most aged-infrastructures in our communities, including water distribution networks, have deteriorated to the point that their serviceability has drawn much attention (ASCE infrastructure grade report, 2009).

A water distribution network should provide, during its economic life, the required quality and quantity of water at required pressures. The system must be able to supply water during unusual conditions such as pipe breaks, mechanical failure of pumps and valves, power outages, malfunction of storage facilities, and uncertain demand projections.

1.2. Components of Water Distribution Networks

The purpose of a water distribution network is to supply the system's users with the required water demand such as fire demands at different nodes, peak daily demands, a series of patterns varying throughout a day, or a critical load when one or more pipes are broken, and to supply this water with adequate pressure under various loading conditions.

Distribution system infrastructure generally comprises pipes, pumps, valves, storage tanks, reservoirs, meters, fittings, and other hydraulic accessories that connect treatment plants or well supplies to consumers' taps (Figure 1-1). The characteristics, general maintenance requirements, and desirable features of the basic infrastructure components in a drinking water distribution system are briefly discussed below.

1.2.1. Pipes

The systems of pipes that transport water from the source (such as a treatment plant) to the customer are often categorized from largest to smallest as transmission or trunk mains, distribution mains, service lines, and premise plumbing. Transmission or trunk mains usually convey large amounts of water over long distances, such as from a treatment facility to a storage tank within the distribution system. Distribution mains are typically smaller in diameter than the transmission mains and generally follow the city streets. Service lines carry water from the distribution main to the building or property being served. Service lines can be of any size depending on how much water is required to serve a particular customer and are sized so that the utility's design pressure is maintained at the customer's property for the desired flows. Premise plumbing refers to the piping within a building or home that distributes water to the point of use. In premise plumbing, the pipe diameters are usually comparatively small, leading to a greater surface-to-volume ratio than in other distribution system pipes.

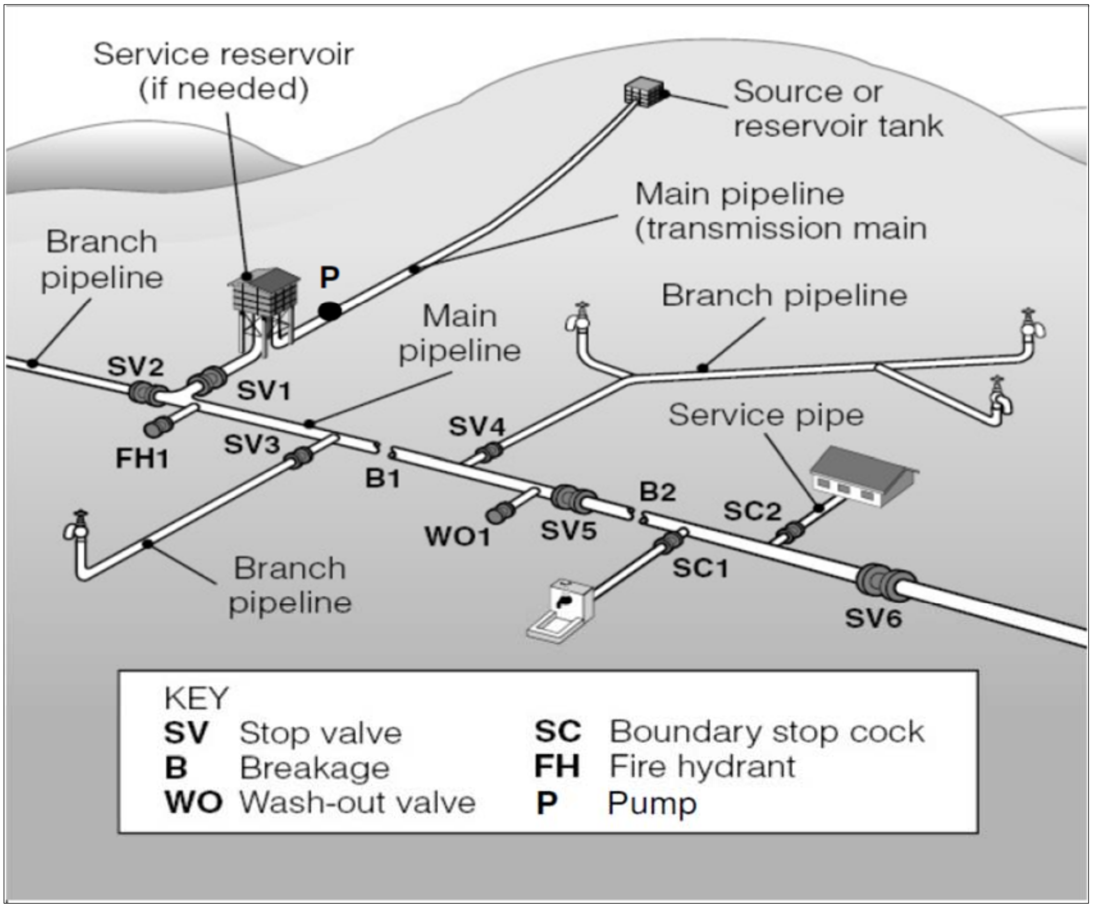


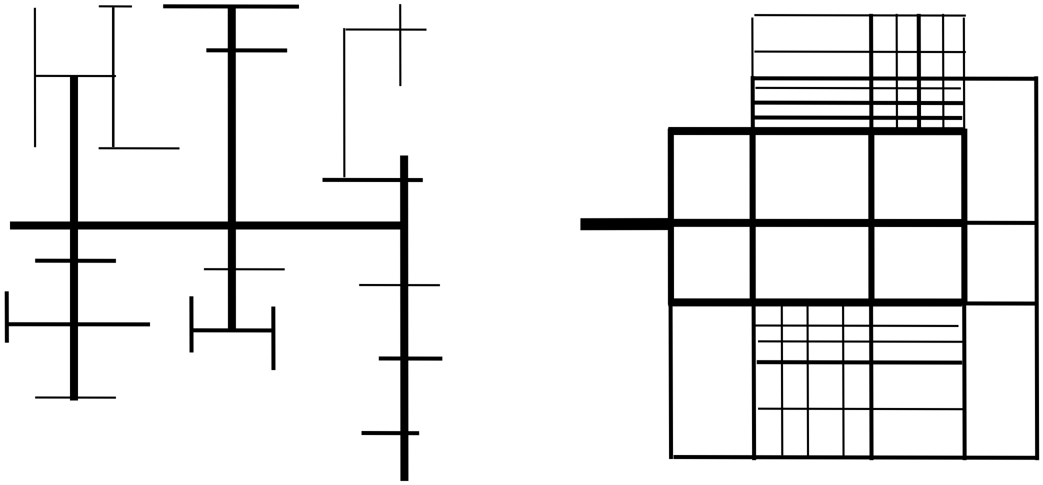
Figure 1-1. Components of a Water Distribution Network

The three requirements for a pipe include its ability to deliver the quantity of water required, to resist all external and internal forces acting upon it, and to be durable and have a long service life (Clark and Tipper 1990). The materials that are commonly used to accomplish these goals today are ductile iron, pre-stressed concrete, polyvinyl chloride (PVC), reinforced plastic, and steel. The material of the pipe is a major element for changing the reliability of a network.

1.2.2. Pipe-Network Configuration

The two basic configurations for most water distribution systems are the branch and grid loop (Figure 1-2.). A branch system is similar to that of a tree branch, in which smaller pipes branch off larger pipes throughout the service area, such that the water can take only one pathway from the source to the consumer. This type of system is most frequently used in rural areas. A grid/looped system, which comprises connected pipe loops throughout the area to be served, is the most widely used configuration in large municipal areas. In this type of system, water can follow several pathways from the source to the consumer.

Looped systems provide a high degree of reliability should a line break occur, because the break can be isolated with little impact on the consumers outside the immediate area (Clark et al., 2004). In addition, by keeping water moving, looping reduces some of the problems associated with water stagnation, such as adverse reactions with the pipe walls, and it increases the fire-fighting capability. However, loops can be dead-ends, especially in suburban areas such as cul-de-sacs, and have associated water quality problems. Most systems are a combination of both looped and branched portions. The design of water networks is very much dependent on the specific topography and street layout in a given community.



(A) (B)

Figure 1-2. Two Basic Configurations for Water Distribution Networks (A) Branched Configuration (B) Looped Configuration

1.2.3. Storage Tanks

Storage tanks are used to provide storage capacity to meet fluctuations in demand, provide a reserve supply for firefighting use and emergency needs, stabilize pressures in the distribution system, increase operating convenience and provide flexibility in pumping, provide water during source or pump failures, and blend different water sources. The recommended location of a storage tank is just beyond the center of demand in the service area (AWWA, 1995). Elevated tanks are used most frequently, but other types of tanks and reservoirs include in-ground tanks and open or closed reservoirs. Common tank materials include concrete and steel.

1.2.4. Pumps

Pumps are used to impart energy to the water in order to boost it to higher elevations or to increase pressure. Pumps are typically made from steel or cast iron. Most pumps used in distribution systems are centrifugal in nature, in that water from an intake pipe enters the pump through the action of a "spinning impeller" where it is discharged outward between vanes and into the discharge piping. The cost of power for pumping constitutes one of the major operating costs for a water supply.

1.2.5. Valves

The two types of valves generally utilized in a water distribution system are isolation valves (or stop or shutoff valves) and control valves. Isolation valves (typically either gate valves or butterfly valves) are used to isolate sections for maintenance and repair and are located so that the areas isolated will cause a minimum of inconvenience to other service areas. Maintenance of the valves is one of the major activities carried out by a utility.

Control valves are used to control the flow or pressure in a distribution system. The typical types of control valves include pressure-reducing, pressure-sustaining, pressure-relief valves, flow-control valves, throttling valves, float valves, and check valves. Most valves are either steel or cast iron, although those found in premise plumbing to allow for easy shut-off in the event of repairs are usually brass. They exist throughout the distribution system and are more widely spaced in the transmission mains compared to the smaller diameter pipes.

Other valves in a water system include blow-off and air- release/vacuum valves, which are used to flush water mains and release entrained air. On transmission mains, blow-off valves are typically located at every low point, and an air release/vacuum valve at every high point on the main. Blow-off valves are sometimes located near dead ends, where water can stagnate or where rust and other debris can accumulate.

1.2.6. Hydrants

Hydrants are primarily part of the fire-fighting aspect of a water system. Proper design, spacing, and maintenance are needed to ensure an adequate flow and pressure to satisfy fire-fighting requirements. Fire hydrants are installed in areas that are easily accessible by fire-fighters and are not obstacles to pedestrians and vehicles.

1.3. Leakage of Water Distribution Networks

Leakage rate and its high associated cost of failure have reached a level that now draws the attention of both policy and decision makers. As a result, dealing with the risk of water leakage has been undergoing a great change in concept from reacting to failure events to taking preventive actions that maintain water networks in good working conditions.

Water loss through leakage has been identified as a problem in virtually all water distribution systems. Leakage is inevitable in all water systems. It is estimated that roughly 32 billion cubic meters of water is lost annually from water distribution systems by way of leakage (Kingdom, Limberger and Marin 2006). Thornton (2002) estimated that the worldwide value of this loss was on the order of $81 billion per year.Losses of water in the distribution network can reach high percentages of the volume introduced. The problems with leakage affect both the efficiency of the network and the water quality.

1.3.1. Pipe Leakage in the United States

Leakage is usually the major cause of water loss in water distribution systems. Environmental Protection Agency (EPA) in 2011 reported 240,000 water main breaks per year in the United States. The number of breaks increases substantially near the end of the system's service life. Large utility breaks in the Midwest increased from 250 per year to 2,200 per year during a 19-year period.

The City of Baltimore, Maryland reported 1,190 water main breaks in 2003, that's more than three per day. The U.S. Geological Survey estimates that the quantity of water lost from water distribution systems is 1.7 trillion gallons per year at a national cost of $2.6 billion per year.

Figure 1-3. Water Leakage in a Joint

1.3.2. Locations of leaks

Leakage occurs in different components of the distribution system transmission pipes, distribution pipes, service connection pipes, joints, valves, and fire hydrants. The amount of leakage at a given time in a distribution network depends on the structure of the network, material of the network, the pipe flows, and the age of the network and the length of the work cycle (Figure 1-4).

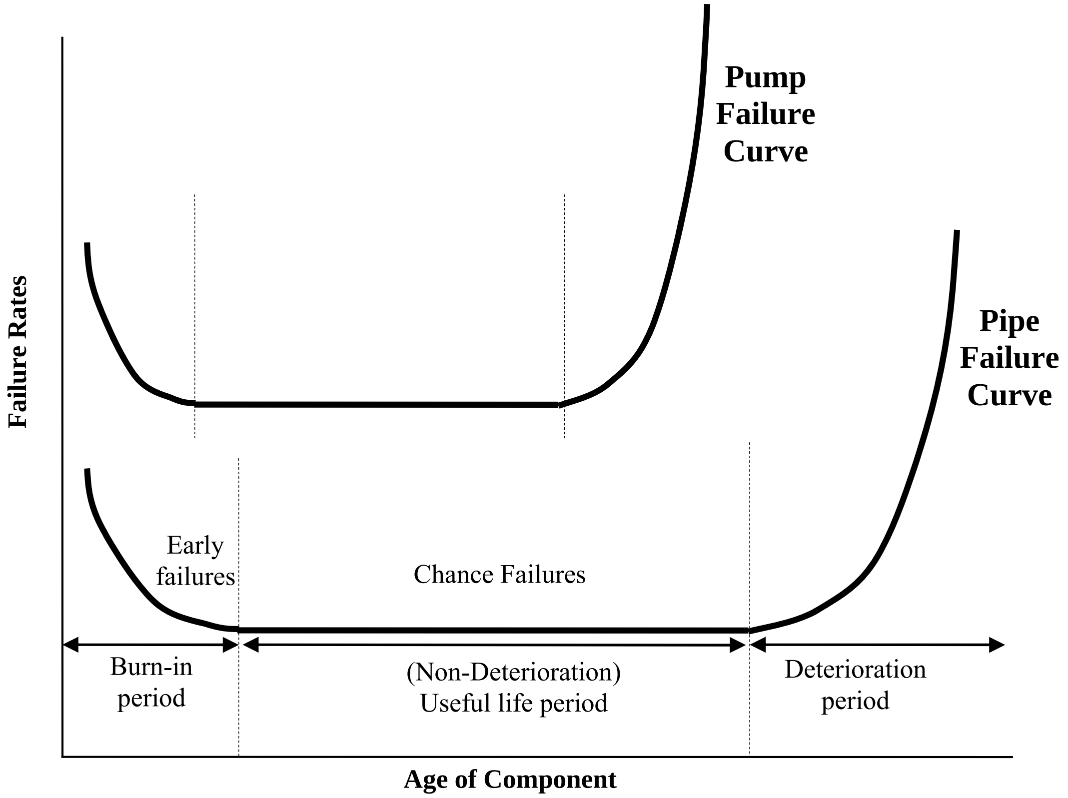


Figure 1-3. Work Cycle

1.3.3. Causes of Leaks

The main causes that lead to high percentages of leakage in different components of the water distribution network are corrosion, soil corrosivity, excessive water pressure, material defects, water hammer, excessive loads and vibration from road traffic, stray electric current and water temperature (Figure 1-5).

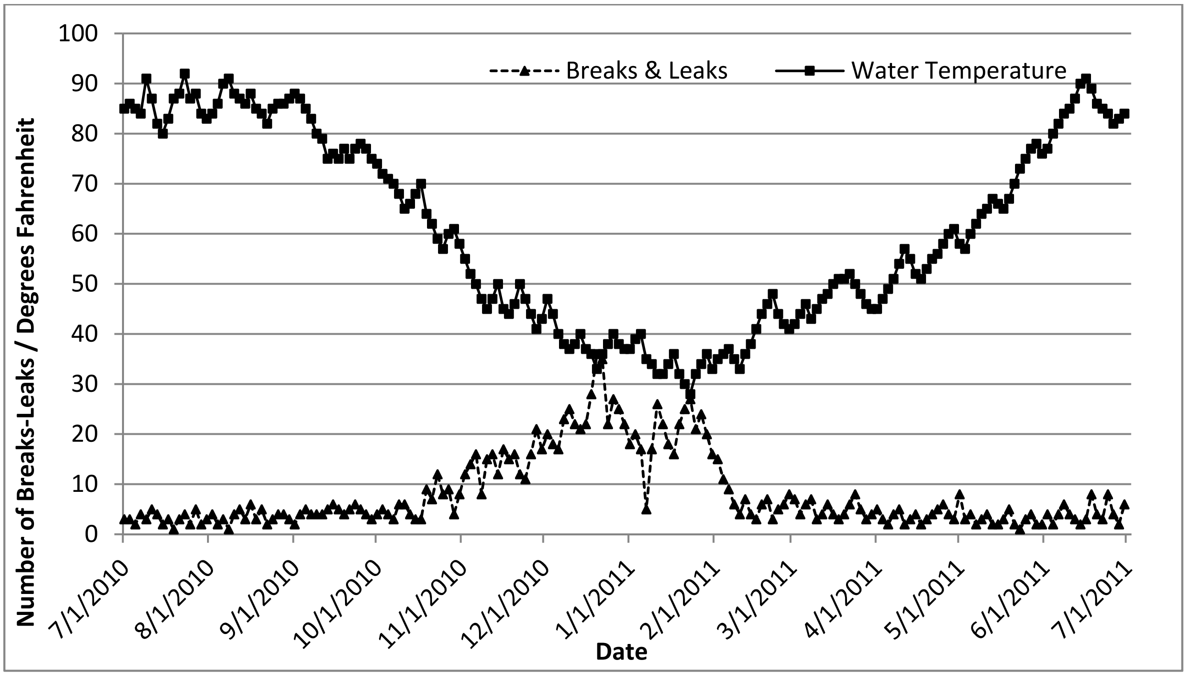


Figure 1-4. Potomac Water Temperature vs. Water Main Breaks and Leaks (Fiscal Year 2011)

1.4. Objective of this Research

Our water supply is finite, which means that we do not have an endless supply. We only have the water that we have now. Ninety seven percent of all the water on earth is saltwater that is not suitable for drinking. Only three percent of all the water is freshwater and only one percent is available for drinking water. The other two percent is locked in ice caps and glaciers.

Utilities can no longer tolerate inefficiencies in water distribution systems and the resulting loss of revenue associated with underground water system leakage. Increases in pumping, treatment, and operational costs make these losses prohibitive.

The economic benefits of decreasing pipe leakage can be easily estimated. For an individual leak, the amount lost in a given period of time, multiplied by the retail value of that water will provide a dollar amount. In addition to this dollar amount, we should factor in the costs of developing new water supplies and other hidden costs. Some other potential benefits of decreasing pipe leakage that are difficult to quantify include (Lahlou 2001):

- Increased knowledge about the distribution system, which can be used to respond more quickly to emergencies and to set priorities for replacement or rehabilitation programs
- More efficient use of existing supplies and delayed capacity expansion
- Improved relations with both the public and utility employees
- Improved environmental quality
- Increased firefighting capability
- Reduced property damage, reduced legal liability, and reduced insurance because of the fewer main breaks
- Reduced risk of contamination.

In this dissertation a validated model that estimates the recoverable leakage of water distribution networks was proposed by examining key causes that lead to high percentages of leakage in different components of the water distribution network, Monte Carlo simulation will be used to examine the strength of the new probabilistic estimation model. Determining the key causes that lead to high percentages of leakage in different components of the water distribution network, will help water utilities perform a predictive or preventive action plan rather than reacting to the failure and losses occurring due to the leakage of the water distribution network.

1.5. Structure of the Dissertation

This dissertation is organized as follows (Figure 1-6). Chapter 2 summarizes the literature that focuses on the risk of leakage in water distribution networks. The water balance and causes of leakage are described in Chapter 3. A probabilistic model for solving the problem (Proposed Model) is discussed in Chapter 4, model validation using two case studies is described in Chapter 5 with the Monte Carlo simulation, and in Chapter 6 the conclusion and recommendations are summarized. In Appendix “A”, the preliminary Sample Questionnaire that was sent to water utility companies is reported and in Appendix “B” the Survey Responses are tabulated. The AWWA component analysis to calculate UARL is listed in Appendix “C”. Appendix “D” includes AWWA Audits for the Case Studies. In Appendix “E”, a typical soil resistivity map for the US is included. Appendix “F” lists the Water Loss Recommendations and finally all the references are listed.

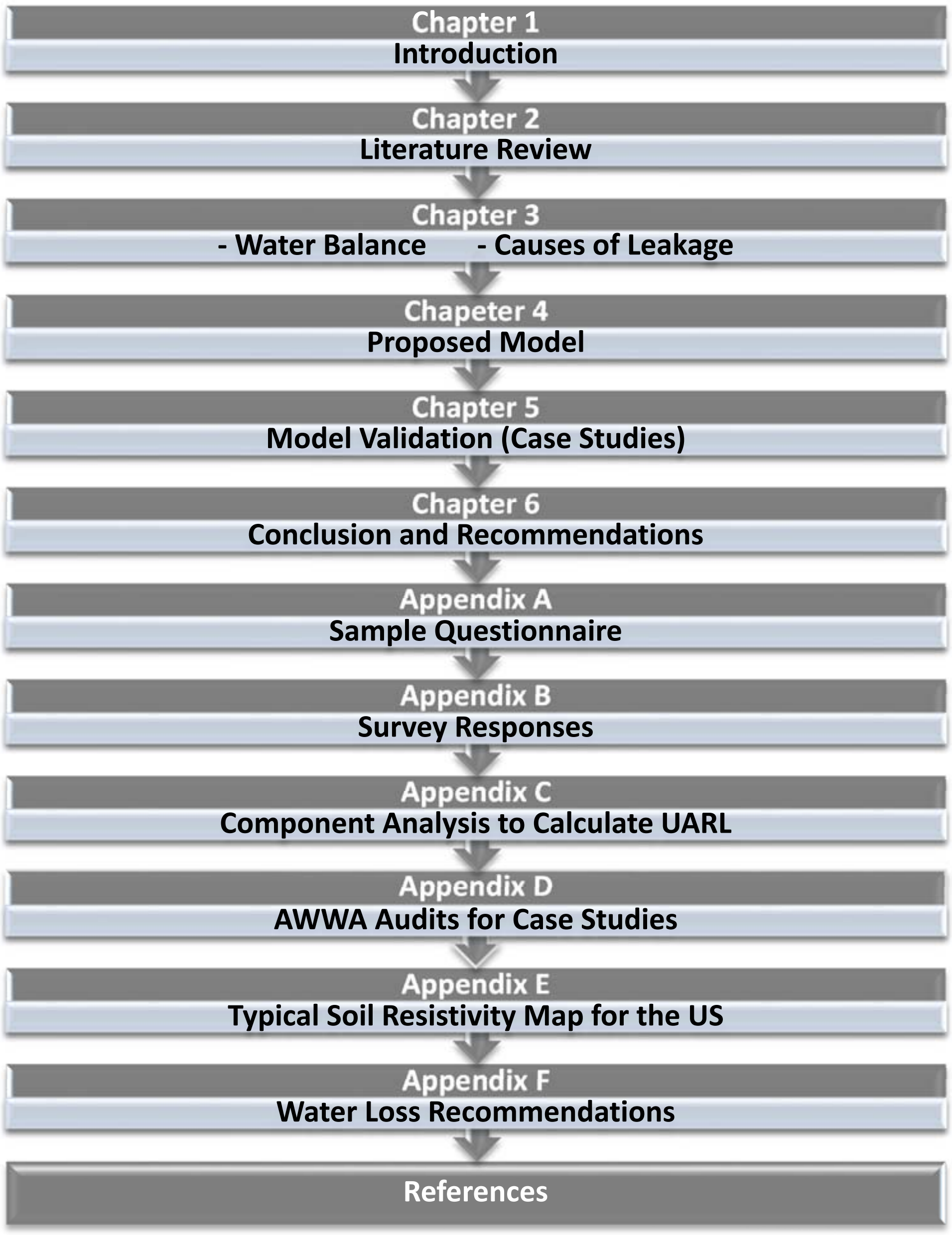


Figure 1-5. Structure of the Dissertation

Chapter 2. Literature review

Key areas related to the risk of water networks are reviewed in order to enhance understanding, define the problem and its attributes and how others approached the problem, and how to best solve the stated problem. In this research, the reviewed topics are pipeline failure risk, leakage of water distribution networks, different factors of risk associated with water main failure, reliability risk studies, and the consequences of failure. Different approaches in evaluation and modeling the risk of failure such as matrix models, probabilistic models were also reviewed.

2.1. Pipeline Failure Risk

Cunha et al. (2010) presented a robust optimization-based approach for designing a water distribution network aimed at obtaining solutions that can cope with the uncertainty and failure risk of the network's working conditions. Robust optimization is a scenario based technique, and in the present case its goal is to provide significant savings in comparison with the worst case scenario solution. The solution was obtained using the link between a simulated annealing algorithm, the optimizer, and a hydraulic simulator to solve the constraints.

Al Barqawi (2006) designed two condition rating models for water mains using artificial neural networks (ANN) and the analytical hierarchy process (AHP). In his research, he considered only the deterioration factors (physical, operational, and environmental). Using the ANN model, he concluded that the most important factors are breakage rate and pipe age. However, when using an integrated ANN/AHP model, pipe age, pipe material, and breakage rate are the most effective factors in evaluating the current condition of water mains. He proposed a condition rating scale from 0 to 10 divided into 6 regions that describe the status of the water main.

Kleiner et al. (2006) developed a methodology to evaluate pipeline failure risk using the fuzzy logic technique. The model comprises three parts: possibility of failure, consequence of failure, and a combination of these two to obtain the failure risk. In the possibility of failure part, a seven-grade fuzzy set is used to describe the asset condition rating and a nine-grade possibility of failure is used to reflect the possibility of failure. The failure condition rating is fuzzified on the nine-grade possibility of failure. In the consequences of failure part, the severity of an asset failure consequence is described in a nine-severity grade. The consequences of failure can be in the form of direct cost, indirect cost, and social cost. The risk of failure is assessed by combining the probability of failure with the consequences of failure in nine fuzzy triangular subsets.

Rogers (2006) developed a model to assess water main failure risk. He used the Power Law form of an Inhomogeneous Poisson Process (NHPP) and Multi-Criteria Decision Analysis (MCDA) based on the Weighted Average Method (WAM) to calculate the probability of failure. Moreover, the developed model considers the consequence of failure using "what-if" infrastructure investment scenarios. The probability of failure and the consequences are directly related by a multiplication operation in order to determine the associated risk.

Kleiner *et al*. (2004) used a fuzzy rule-based, inhomogeneous Markov process to model the deterioration process of buried pipes. The deterioration rate at a specific time is estimated based on the asset's age and condition state using a fuzzy rule-based algorithm. Then, the possibility of failure is estimated for any age of the pipeline based on the deterioration model. The possibility of failure is coupled with the failure consequence through a matrix approach to obtain the failure risk as a function of the pipe's age.

Christodoulou et al. (2003) used Artificial Neural Networks (ANN) to analyze the preliminary water main failure risk in an urban area with historical break data spanning two decades. The type of ANN used in this study is the back propagation algorithm. The outputs of back propagation ANN are the age to failure of each pipe segment, the observation outcome (a break or a non-break), and the relevant weights of the risk factors. Their study indicates that the number of previous breaks, the material, diameter, and length of pipe segments are the most important risk factors for water main failures.

Ezell et al. (2000) introduced the Probabilistic Infrastructure Risk Analysis model (IRAM). This system is developed for small community water supply and treatment systems. It comprises of four phases. In phase 1, the infrastructure failure threats are identified by means of system decomposing. The target of phase 2 is to provide information that describes the state of consequences for a scenario executed against the system under study. An event tree is used together with expert opinion to determine the failure probability of each path in the tree and the inherent consequence. In phase 3, the consequence and the probability of failure are combined together to identify the high risk factors, which are used to manage the infrastructure in phase 4 by setting the acceptance risk level.

2.2. Leakage of Water Distribution Networks

Kornmayer (2011) developed pressure difference-based sensing cells that can be used in an un-tethered leak detection device. This device was deployed in water distribution networks to locate leaks so that water loss can be minimized. The design of these sensing cells and of the leak detection device entails evaluating the size and shape of the leak's low pressure region.

Chen et al. (2010) measured system reliability models for a directed Tree Network System by considering their components lifetime distribution possessing general forms. The component of the system works by the order of the each vertex's path set is introduced with two kinds of failure categories; fail-safe states and fail- dangerous states.

MIWR (2009) recognized the need to harmonize the various methodologies utilized by the various actors in the implementation of water and sanitation interventions. It was agreed that this could be best achieved through the development and distribution of Technical Guidelines, outlining best practices for the development of the 14 types of water supply and sanitation facilities in Sudan.

Buchberger et al. (2004) presented a new method for detecting the magnitude of leaks in small residential service zones of a drinking water distribution system. A performance limit for the proposed method was derived to account for network size and timing averaging.

Rajani et al. (2004) introduced several non-destructive technologies (NDT) that have been developed to inspect water pipes. Some of these technologies exploit the specific pipe material properties and consequently they are not applicable to all pipe materials. Some of these technologies were introduced because some water supply operators prefer not to interrupt the water supply for inspection to avoid customer complaints.

McKenzie et al. (2002) developed four models to assist water suppliers in understanding and managing their non-revenue water. The models are based on Burst and Background Estimate Component Analysis methodology. The models can be used to assess the likely savings of various pressure reduction options in a selected zone metered area.

Hunaidi et al. (2000) presented an overview of techniques and equipment used to detect water leaks in water distribution systems. Hunaidi proposed a technique to improve the effectiveness of leak noise correlators and field procedures in detecting leaks in water distribution networks.

2.3. Failure Factors

Christodoulou (2010) presented a framework for proactive risk-based integrity monitoring of urban water distribution networks. A combination of artificial neural network and statistical modeling techniques were utilized in the investigation of identified risk factors and for the estimation of the forecasted time to failure metric.

Fares et al. (2010) designed a framework to evaluate the risk of water main failure using hierarchical fuzzy expert system considering 16 risk-of-failure factors. The pipe age, pipe material, and breakage rate had the highest effect on risk of water main failure.

Christodoulou et al. (2009) investigated possible risk-of-failure parameters and developed a multi-criteria decision support system (DSS) for the modeling of the behavior of water distribution networks through artificial intelligence techniques (neurofuzzy systems). The analysis incorporates both the scientific aspects of the risk-of-failure for network components and financial and socio-political parameters that are associated with the networks.

The USDA (2004) created a public awareness of soil-related risks and hazards that may not be readily apparent. Discussions of 26 soil-related concerns were developed by authors within the National Cooperative Soil Survey. The description of risks and hazards is to expand the awareness of various soil risks and hazards to human life and property and encourage city and county officials, planners, developers, and others to consider the soil in their land use decisions.

McNeill et al. (2000) presented experimental results that many of the factors of corrosion did not control iron corrosion under stagnant water conditions. These results led to a parallel study on lead corrosion that demonstrated that a pipe study can provide mechanistic insight into water quality effects on lead corrosion.

EPA (1984) created a guide with additional information to study the corrosion of water distribution systems in more detail. The manual gives the operators thane understanding of the corrosion causes and how to control corrosion.

2.4. Reliability Studies

EPA (2012) conducted a research to identify and characterize the state of the technology for a structural condition assessment of drinking water transmission and distribution systems. The broad definition of the structural condition assessment of water mains encompasses the physical modeling of the pipe in the soil, understanding of pipe failure modes, empirical/statistical modeling of historical failures, and inspection of a pipe to discern distress indicators, interpretation of distress indicators into pipe condition rating, and modeling deterioration to forecast future failures and pipe residual life.

Debon et al. (2010) compared models for evaluating the risk of failure in water supply networks. Using real data from a water supply company it was identified as to which network characteristics affect the risk of failure and which models better fit the data to predict service breakdown. The comparison using the receiver operating characteristics graph leads to the conclusion of the best model. A proposed procedure was applied to a pipe failure database.

Weiha (2009) proposed a novel method to identify the index of node importance and the critical path from the view of system theory, information theory and control theory to improve the project management level and reduce and avoid risks in project management. Bayesian Theory and Robust Control Strategy were used to study risks on the basis of analyzing the multi-scaling characteristics of risks.

Beuken (2008) proposed a study for improving maintenance schedules based on asset reliability risk analysis. Reliability Centered Maintenance (RCM) is a process to define the requirements for the maintenance of an asset in a specified environment, with the aim to assure its assigned function.

Ghoniema et al. (2007) proposed a methodology to determine the reliability of systems with different configurations and complexity. The methodology considered both mechanical and hydraulic reliability using Monte Carlo simulation for evaluation. The statistical analysis was coupled with a genetic optimization algorithm to provide an efficient tool for designing a water distribution network based on both node demands and systems reliability.

Marinis et al. (2006) presented a methodology for a Risk-Cost based decision support system for the rehabilitation of the water distribution network. The objectives considered were the minimization of the total rehabilitation cost (the sum of the structural costs and the lost revenue costs) and the minimization of risk.

Schefs (2003) developed a risk based replacement strategy to predict the deterioration of Reinforced Concrete Pipes and the consequences of failure. This strategy provided a theoretical approach based on the results of a reliability study, the use of random drawing and the decision tree method to determine an optimal replacement strategy.

Kleiner et al. (2001) quantified the structural deterioration of water mains by analyzing historical data performance data. The physical mechanisms that lead to pipe failure require data that are not available and, therefore, statistical models with various levels of input can be applied. A decision support system was developed based on a specific statistical model.

McNeill et al. (2001) reiterates the conclusions of prior studies regarding the Langelier Index despite its continued widespread use, the Langelier Index does not provide an effective means of controlling iron corrosion. The potential implications of upcoming regulations for iron corrosion were reviewed.

McNeill et al. (2000) presented experimental results that many of the factors of corrosion did not control iron corrosion under stagnant water conditions. These results led to a parallel study on lead corrosion that demonstrated that a pipe study can provide mechanistic insight into the water quality effects on lead corrosion.

2.5. Evaluation of Risks

Karamouz et al. (2010) developed an optimization approach in order to determine the best scheduling system for rehabilitation and maintenance. The criteria of the selection of an appropriate rehabilitation and maintenance schedule is maximizing the lifetime of the structure as well as minimizing the related costs of the system.

Hickey (2008) provided recognized practices for conducting water supply tests at prescribed intervals to measure the water system delivery capability and ensure that the system is meeting the water supply demand. An important part of this objective is to use the results of water supply tests to monitor the performance of the water delivery system in relation to the existing water supply and the constant changes in demand on the water system.

Watson (2004) developed mixed-integer linear programming models for the sensor placement problem over a range of design objectives using two real world water systems.

EPA (2002) has conducted a study to analyze a quantifiable gap between the projected clean water and drinking water investment needs over a twenty-year period from 2000 to 2019 and the current levels of spending. The analysis found that a significant funding gap could develop if the nation's clean water and drinking water systems maintain the current spending and operations practice.

EPA (2002) addressed the health risks related to specific water distribution system topics. The characteristics of deteriorating water distribution systems include the increased frequency of leaks, main breaks, taste, odor and red water complaints, reduced hydraulic capacity due to internal pipe corrosion, and increased disinfectant demands due to the presence of corrosion products, biofilms, and regrowth.

Chapter 3. Water Balance

3.1. Introduction

The International Water Association (IWA) and the American Water Works Association (AWWA) began to finalize the standard methods to assist water utilities in tracking their distribution system losses. These methods are the foundation of water auditing and conservation strategies that are now being used successfully worldwide. In order to understand how to apply the AWWA/IWA methodology, several concepts and terms must be defined and explained. The AWWA/IWA Water Balance Table (Figure 3-1) is the foundation of the methodology and defines the terms that are used in water auditing.

Neither the term "unaccounted-for-water" nor the use of percentages as measures of water loss is sufficient to completely describe the nature and extent of distribution system water loss. Unaccounted-for-water is a term that has been historically used in the United States to quantify water loss from distribution systems. Unaccounted-for-water, expressed as a percentage, is calculated as the amount of water produced by the Public Water System (PWS) minus the metered customer use divided by the amount of water produced multiplied by 100 as shown in Equation 3-1.

$$Unaccounted\ for\ Water = \frac{(Water\ Produced\ by\ PWS - Metered\ Water\ Used) \times 100}{Water\ Produced\ by\ PWS} \tag{3-1}$$

Although this percentage provides a rough idea of how much water is unaccounted for, it does not help answer questions concerning whether the water is really being lost. If it is being lost, where is it being lost? Determining how much water is being lost and where losses are occurring in a distribution system can be a rather difficult task. Without consistent and accurate measurement, water losses cannot be reliably and consistently managed.

3.2. Components of Water Balance

Standardized terminology and definitions are crucial to consistent measurement. These standards are needed to accurately track performance and improvements. In the AWWA/IWA methodology, all of the water that enters and leaves the distribution system can be classified as belonging to one of the categories in the water balance table shown in Figure 3-1. The table is balanced because it accounts for all of the water in the distribution system and the sum of any of the columns should also total the System Input Volume.

<table>
<tr><td rowspan="8">System Input Volume</td><td rowspan="4">Authorized Consumption</td><td rowspan="2">Billed Authorized Consumption</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Un-metered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="6">Non Revenue Water (NRW)</td></tr>
<tr><td>Unbilled Un-metered Consumption</td></tr>
<tr><td rowspan="4">Water Losses</td><td rowspan="2">Apparent Losses (Commercial Losses)</td><td>Unauthorized Consumption</td></tr>
<tr><td>Customer Meter Inaccuracies and Data Handling Errors</td></tr>
<tr><td rowspan="2">Real Losses (Physical Losses)</td><td>Storage Leaks and Overflows from Water Storage Tanks</td></tr>
<tr><td>Service Connections Leaks up to the Meter</td></tr>
</table>

Figure 3-1. AWWA/IWA Water Balance Table

System Input Volume is the amount of water that is produced and added to a distribution system by a PWS. It also includes water that may have been purchased from another water supplier to supplement the needs of the PWS.

Authorized Consumption is the water used by known customers of the PWS. Authorized consumption is the sum of the billed authorized consumption and unbilled authorized consumption and is a known quantity.

Billed Metered Consumption is an authorized consumption that is directly measured. It is the quantity of water that is metered and generates revenues through the periodic billing of the consumer.

Billed Unmetered Consumption is an authorized consumption that is based on an estimate or flat fee. This billing method is used for customers that do not have meters. Estimated use is often based on historical or average use data. The fee may vary for different types of customers such as residential or industrial.

Unbilled Authorized Consumption consists of known uses, condoned by the utility, for which no revenue is received. Unbilled authorized consumption can be either metered or unmetered. Unbilled authorized consumption is shown in yellow in Figure 3-1. Some examples might include filling city street cleaner trucks or a city swimming pool, flushing water lines or sewers, or water used by the fire department. All are legitimate water uses, with the full cognizance of the utility.

Unbilled Metered Consumption is that quantity of water that does not generate revenues but which is accounted and not lost from the system. Water that is used in the treatment process or water provided without charges are examples of these quantities.

Revenue Water is water that is consumed and for which the utility receives payment. Revenue water consumption volume is measured or estimated. Revenue water includes metered and un-metered billed authorized consumption. Revenue water is shown in green in Figure 3-1.

Non-Revenue Water (NRW) is water that is not billed and no payment is received. It can either be authorized, unauthorized, or result from a water loss. Authorized NRW consists of unbilled metered consumption and unbilled unmetered consumption.

Unbilled Unmetered Consumption is the quantity of water that is authorized for use by the PWS but is not directly measured and creates no revenues. Water main flushing, street cleaning, cleaning and lining projects and firefighting are often examples of this category.

Unbilled Metered Consumption is directly measured water use with no charge. This category can include water use at city government offices, street cleaning, or city park irrigation. Some water utilities either meter or estimate use by the city or public services such as fire departments even though no fee is charged. These systems will have an advantage when preparing a water audit since this information will be required to complete the water balance.

Unauthorized Consumption is that quantity of water which is removed from the system without authorization and presumably without the PWS's knowledge. Unauthorized consumption includes theft by illegal meter by-passes, vandalism, or unmetered hydrant use for construction or recreation. This water quantity is very difficult to estimate but must be accounted and is amenable to reduction through administrative action. Unauthorized consumption can also be a potential source of contamination due to lag of backflow prevention device in use.

The lower part of the Water Balance Table consists of Water Losses. Water losses are categorized as either real or apparent. Real Losses**,** also referred to as physical losses, are actual losses of water from the system. When performing financial calculations related to real losses, the water is priced at the cost of production rate since it is not available for a consumer to use and costs only what it takes to produce. Correcting real losses will result in a lower operating cost through reduced production requirements and reduced water process chemical and electrical use.

Real Losses are the physical leaks shown in gray in Figure 3-1 and consist of leakage from transmission and distribution mains, leakage and overflows from the utilities storage tanks, and leakage from service connections up to and including the meter. Preventing or repairing real losses usually requires an investment in PWS infrastructure. Infrastructure investment can reduce losses such as:

- Distribution and transmission main leaks, which represent the quantity of water that is lost from the system, generates no revenue, can severely damage system reliability if not corrected, and may result in water quality problems.

- Storage leaks and overflows from water storage tanks, which consist of the quantity of water that is lost from the storage facilities within the system. Depending on the climate and storage configuration, these losses can also be due to surface evaporation.

- Service connection leaks**,** which consist of the quantity of water that is lost from leaks from the main to the customer's point of use. Even though a leak after a customer's meter can generate revenues for the PWS and is often the responsibility of the customer, it is wasteful and can strain customer and PWS relations. Service connection leaks represent real losses from the system and are frequently easy to detect. In the AWWA/IWA water audit methodology, only service connection leaks up to the meter are included.

Apparent losses, also referred to as commercial losses, occur when water that should be included as revenue generating water appears as a loss due to theft or a calculation error. Apparent losses consist of unauthorized consumption, metering calibration errors, and data handling errors. Apparent losses are shown in orange in Figure 3-1.

Meter calibration error and data error losses can be thought of as accounting losses. This quantity of water is not lost from the system and generates no revenues but if not included in loss calculations can produce misleading water loss estimates. These errors arise from service meter calibration errors, meter reading errors, data handling and billing errors, and billing period variances. These quantities may be reduced through administrative action.

3.3. Performance Indicators

The AWWA/IWA audit methodology heavily relies on three performance indicators to help characterize real losses from distribution systems. These performance indicators are the Current Annual Volume of Real Losses (CARL), the Unavoidable Annual Real Losses (UARL) and the Infrastructure Leak Index (ILI).

The *Current Annual Volume of Real Losses (CARL)* is the volume of water that is lost from the system due to leaks in the transmission and distribution systems, losses at the utility's storage tanks and leaks in the service lines from the main to the point of customer usage. The CARL is given in gallons/day averaged over a one-year period. This total volume is largely straightforward and easily computed by most utilities. It should be recognized that this volume contains water losses that can be identified, located, and repaired as well as those unavoidable leaks that every system contains.

$$CARL\left(\frac{gallons}{day}\right) = \begin{array}{l}\text{Transmission Losses} + \text{ Distribution Losses}\\ +\text{Storage Losses} + \text{Service Line Losses}\end{array} \qquad (3\text{-}2)$$

The Unavoidable Annual Real Losses (UARL) is a subset of a system's CARL leaks that are unavoidable, which may be too small to be discovered, and may prove to be too expensive or inaccessible to be repaired. The UARL is also given in gallons/day averaged over a one-year period. By defining and then calculating the volume of the UARL in the system, an indication of the potentially Recoverable Real Losses can be calculated as the difference between the CARL and the UARL.

Unfortunately, UARL are very difficult to estimate. However, AWWA/IWA research across a large number of systems, together with actual operating data from many countries, has resulted in the development of a relationship between various system parameters and the UARL with statistically good accuracy as shown in Equation (3-3). The volume of a system's UARL turns out to be a function of the length of the distribution system (Lm) in miles, the number of service connections (Nc), the length of the service lines (Lp) in miles and the average system operating pressure (P) in psi.

$$UARL\left(\frac{gallons}{day}\right) = (5.41 \times Lm + 0.15 \times Nc + 7.5Lp) \times P \tag{3-3}$$

The *Infrastructure Leak Index (ILI)* is an index recommended by the IWA for establishing utility water loss management targets. The ILI was developed to overcome the shortcomings of other water loss target systems in use and to generate a verifiable target that could be used for the management of a water loss program that is readily comparable to industry benchmarking. The ILI is defined as the ratio between the Current Volume of Real Losses and the volume of Unavoidable Losses (Equation 3-4).

$$ILI = \frac{CARL}{UARL} \tag{3-4}$$

The ILI is substantially different and more meaningful than the frequently used simple ratio between unaccounted-for water and total plant production for comparing system efficiencies. This unaccounted-for water ratio provides only limited information about the real water loss characteristics of the system. The ratio will not change as operating conditions are altered. In fact, it can even appear to improve when actual water losses are increasing. For example, a new subdivision goes on line and the total production increases to meet the additional demand with little if any additional unaccounted for losses. However, the ratio of unaccounted for water divided by plant production will actually decrease as the plant production increases, even though the total quantity of water loss from the system has not decreased.

The system may appear to be more effective than it was the day before the new portion of the distribution system went on line, but in reality, just as much product is being lost as before the addition. Such insensitivity makes this old water loss ratio an ineffective metric for economic or operations planning and is virtually meaningless as a comparison between systems. The ILI calculation includes pipe length and other parameters that adjust for changes to the distribution system and make it more useful as a comparison between different audit periods or even PWSs.

An ILI index of 1.0 indicates that current annual real losses are equal to unavoidable losses and the PWS is operating efficiently when considering real water loss. Actual ILI values typically fall in the range of 1.5 to 2.5 for most PWSs. When a PWS uses the ILI as an evaluation parameter for a water loss reduction project, it must consider the costs it will need to incur and pass on to its customers to reduce its ILI index. Benchmarking is an indicator of a utility's water loss situation with respect to previous audits of other utilities; it does not define the acceptability or appropriateness of the loss rate for the particular PWS.

Acceptable rates of water loss should be established by the PWS or may be established by regulatory authorities.

3.4. Economic Considerations of Real Losses

The objective of a water loss control program is to apply all of the available techniques to recover as much of the losses as possible. A well-run water loss management program has limits to what it can achieve. Ideally, no water would be lost, but this is not achievable in the field. A balance must be maintained between water loss reduction and the costs associated with water loss reducing measures. A PWS can directly affect real water losses by controlling:

- Pressure management
- Speed and quality of repairs
- Active leakage control
- Pipeline and assets management through selection, installation, maintenance, renewal, or replacement

Figure 3-2 is a graphical representation of the component parts of lost water and the actions that an active water loss management program can use to address these losses. Active Leakage Control (ACL) is the process of proactively searching for leaks that are not yet apparent and repairing them.

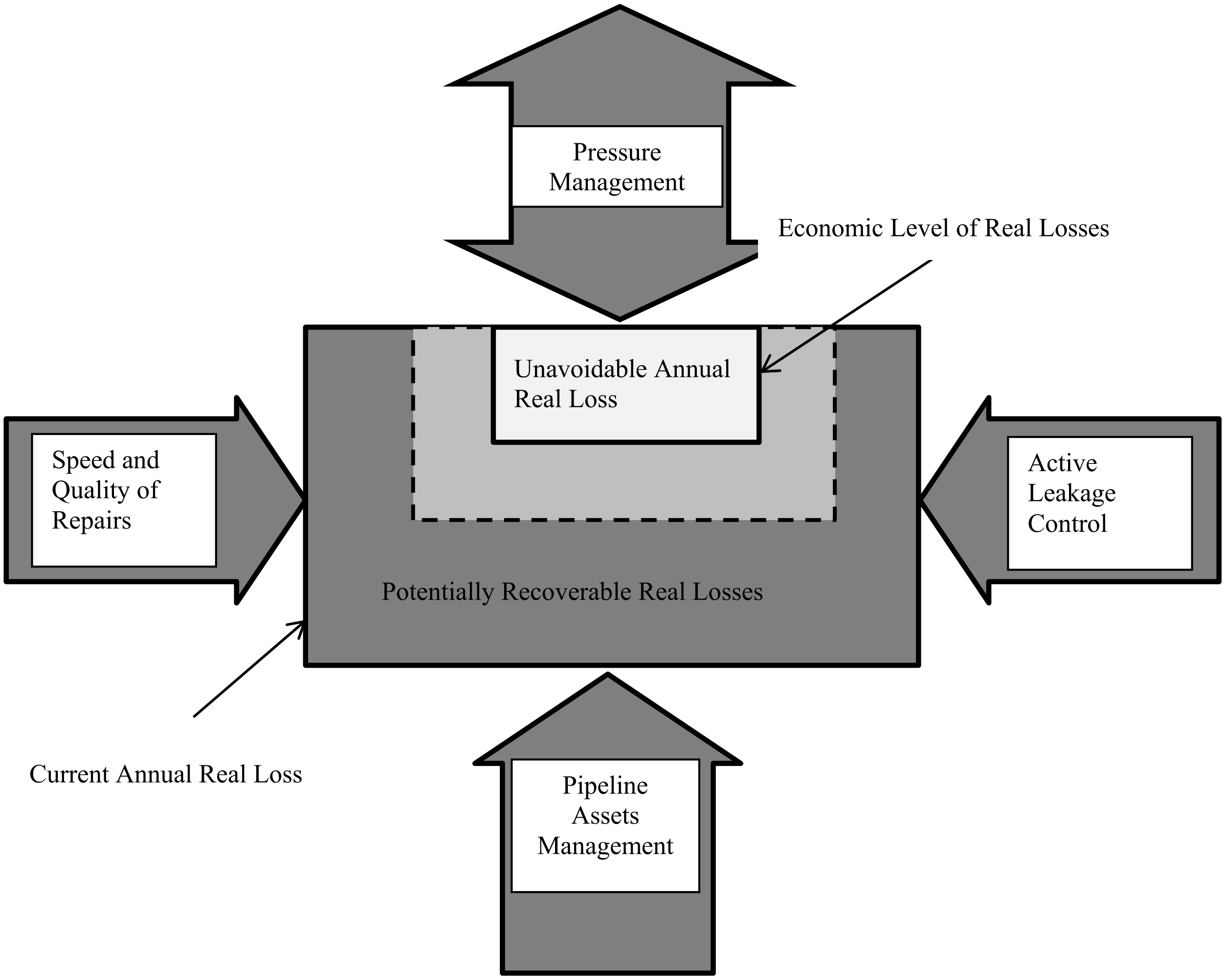


Figure 3-2. Forces Controlling Leakage and Costs

Asset management involves documenting and evaluating the components of a water utility to determine when the optimum time is to replace or repair a component or pipeline. Evaluation of whether to replace or repair a component not only depends on the economics of replacement or repair but also on the impacts on the community that is being served, such as potential health effects, inconvenience, or public opinion and perception of the utility.

Pressure management affects the water loss rates. In addition, the lack of pressure management has been shown to increase pipe failure rates. These are relatively intuitive ideas since more pressure means greater flow whether it is through the pipe or through a crack or hole in the side of the pipe. Higher pressures mean higher stresses on the pipe. Higher pressure also means higher pressure spikes during pressure surges. These higher values translate into increased failure rates. The management goal is to meet customer pressure expectations, fire flow requirements, and adequate pressures to operate the system at an as low pressure as is reasonable.

Each of the methods that a PWS has to address regarding real losses also has an associated cost. In Figure 3-2, the CARL sets the existing losses and associated costs and the UARL establishes the loss reduction that a PWS can achieve. The area between is potentially recoverable real losses. The balance of what makes economic sense for a water loss reduction program for the water system lies between these two and is called the Economic Level of Leakage (ELL).

The ELL helps compare costs for making decisions whether a leak detection program will pay for itself or when to repair a pipe versus replacing it. The ELL is the point at which the cost of reducing leakage is equal to the benefit gained from leakage reductions. This can become a very involved process and requires comparing different scenarios. Figure 3-3 illustrates the general approach. The real cost of the volume of water that is lost is proportional to the time that the leak starts until it is repaired. If the leak management program allows for minimal field inspections, the probability of a leak going undetected for an extended amount of time increases. A program with frequent field visits minimizes the time to detect leaks and, therefore, reduces the lost revenue and volume of finished water.

The cost to detect and find the leak should also be accounted for in the final estimate. A program with infrequent leak detections will have a very low detection cost per year. Conversely, a program with a frequent detection cycle will experience high annual costs. It is important to point out that this cost does not include the cost of repair since these costs would be very similar regardless of the time it took to detect the leak.

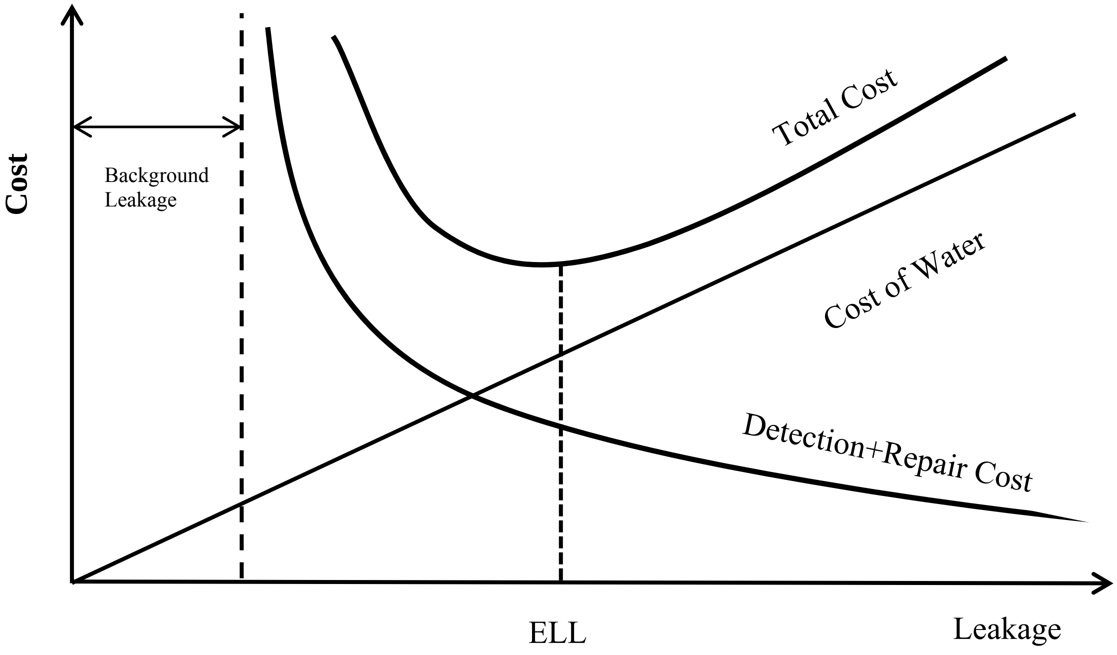


Figure 3-3. Example of an Economic Level of Leakage (ELL) Curve

The cost per year to conduct a field investigation diminishes exponentially as the number of detection cycles decreases. A parabolic cost curve is formed, rapidly falling from many cycles per year to achieve very low water loss to relatively low total cost per year for programs that are willing to have greater leak loss but only detect infrequently. However, even though a utility may elect to have a frequent detection cycle, there will be a minimum at which no amount of detection effort will find the leaks. At this point, the cost of detection line becomes asymptotic to the background leakage levels.

The total cost of leak detection is, therefore, the sum of these two opposing cost curves. The resultant saddle-curve provides a minimum program range at which the detection frequency is balanced with the amount of water loss from the system. This is known as the ELL range.

The identification process may not precisely point to the leak location, but significantly reduce the uncertainty and thus enable a field crew to achieve better detection rates more quickly within the areas where the leakage has been predicted.

It is important to know how much leakage is occurring on customer supply pipes compared with the other parts of the water network (e.g. trunk mains and local distribution mains) because of questions about:

- Where detection efforts should be directed for maximum gain?
- What are the environmental factors that might influence loss rates?
- For which areas and types of pipes is renewal the most economical option?

3.5. Causes of Leaks

Leakage occurs in different components of the water distribution system: transmission pipes, distribution pipes, service connection pipes, joints, valves, and fire hydrants. Causes of leaks include the following, with the details provided below: corrosion, soil corrosivity, excessive water pressure, material defects, water hammer, excessive loads and vibration from road traffic and stray electric current.

3.5.1. Corrosion

Corrosion is the disintegration of an engineered material into its constituent atoms due to chemical reactions with its surroundings. In the most common use of the word, this means the electrochemical oxidation of metals in reaction with an oxidant such as oxygen. Formation of an oxide of iron due to the oxidation of the iron atoms in a solid solution is a well-known example of electrochemical corrosion, commonly known as rusting. This type of damage typically produces oxide(s) and/or salt(s) of the original metal. In other words, corrosion is the wearing away of metals due to a chemical reaction.

One of the most common problems affecting domestic water supplies is corrosion, which is a chemical process that slowly dissolves metal, resulting in deterioration and failure of water distribution pipes. Corrosion is a natural process that occurs when metals are in contact with oxygen and react to form metal oxides. All water is corrosive to some degree as it contains some amount of dissolved oxygen. The rate of corrosion depends on a number of factors including acidity or low pH, electrical conductivity, oxygen concentration, and water temperature.

In addition to corrosion, the dissolution of metals occurs when the water is extremely low in dissolved salts or in the presence of certain water-borne ions. All materials have a particular level of solubility and, in the case of corroded plumbing, the concentration of the copper or other plumbing material metal is lower in the water than that of the material's solubility. As a result, the plumbing material is gradually dissolved. While this process is usually very slow, certain water-borne ions can react with and bind the recently dissolved metal allowing for more rapid loss. While corrosion and dissolution are fundamentally different, the end result is similar and so both are often discussed together under the general term corrosion.

The pH value is used to measure acidic and alkaline materials in water. The pH scale ranges from 0 to 14, with a pH of 7.0 representing the neutral point where acid and alkaline materials are in balance. Water with pH values below 7.0 is dominated by acidic materials, while water with pH values above 7.0 is alkaline.

The terms alkalinity and pH often are confused. Total alkalinity is a measure of the total bases in water that can neutralize acid. This includes bicarbonates, carbonates, hydroxides, and even some phosphates and silicates. Alkalinity is reported in units of milligrams per liter (mg/L) of calcium carbonate.

For ideal corrosion control, water should have moderate alkalinity (40 to 70 mg/L) and a pH between 7.0 and 8.2. Water with pH values below 6.5 will be corrosive, especially if alkalinity is also low. However, water with pH values above 7.5 can also be corrosive when alkalinity is low.

Corrosion is more likely and more rapid at higher water temperatures. The rate of corrosion increases by a factor of three to four as the water temperature rises from 60°F to 140°F. Above 140°F, the rate of corrosion doubles for every 20°F increase in water temperature. Corrosion occurs in the presence of moisture (Figure 3-4). For example, when iron is exposed to moist air, it reacts with oxygen to form rust (Equation 3-5).

$$\text{Rust} = Fe_2O_3 \cdot XH_2O \tag{3-5}$$

The amount of water complexed with the iron (III) oxide (ferric oxide) varies, as indicated by the letter "X". The amount of water present also determines the color of rust, which may vary from black to yellow to orange brown. The formation of rust is a very complex process that is thought to begin with the oxidation of iron to ferrous (iron "+2") ions (Equation 3-6).

$$Fe \text{ ------->} \ Fe^{+2} + 2\,e^{-} \tag{3-6}$$

Both water and oxygen are required for the next sequence of reactions. The iron (+2) ions are further oxidized to form ferric ions (iron "+3") ions (Equation 3-7).

$$Fe^{+2} \text{ ------------>} \ Fe^{+3} + 1\,e^{-} \tag{3-7}$$

The electrons provided from both oxidation steps are used to reduce oxygen as shown in Equation 3-8.

$$O_2\,(g) \;+\; 2\,H_2O \;+\; 4e^- \;\text{------>}\; 4\,OH^- \qquad (3\text{-}8)$$

The ferric ions then combine with oxygen to form ferric oxide [iron (III) oxide], which is then hydrated with varying amounts of water. The overall equation for the rust formation as shown in Equation 3-9.

$$4Fe(OH)_2(s) + O_2(g) \;\text{----->}\; 2Fe_2O_3 \bullet H_2O(s) + 2H_2O(l) \qquad (3\text{-}9)$$

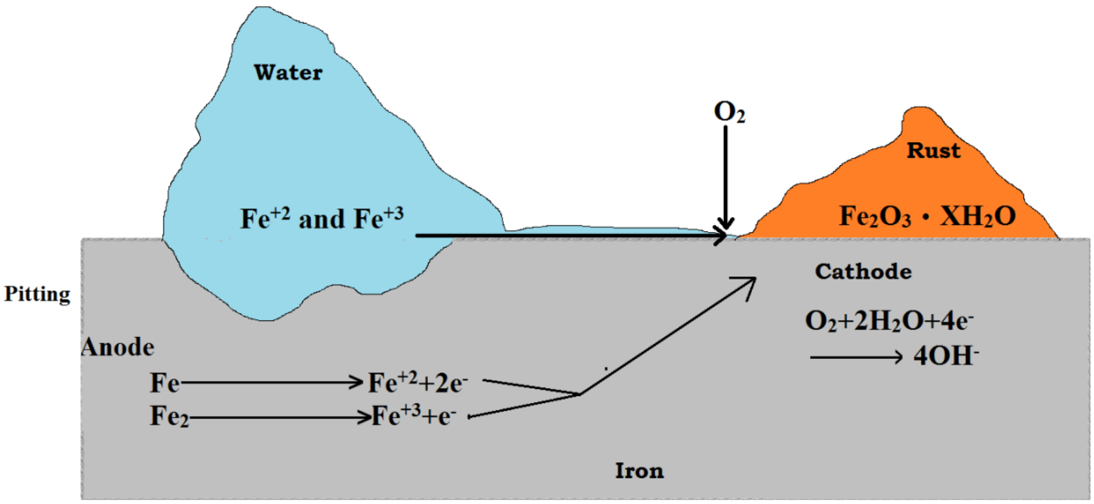


Figure 3-4. Chemistry of Corrosion

The main cause of leaks in water mains is external corrosion. “Corrosion is the root, if not the immediate, cause of most breaks in metal pipes. Metals tend to want to return to their ore state” (Walski, 1984). Pipes in wet, humid soils are more susceptible to external corrosion than pipes in drier soils. Low redox potential and low resistivity in wet soils lead to significant corrosion problems. Although corrosion is a natural process, stray electrical currents may contribute significantly to corrosion rates in metal pipes. The problem of pitting, which causes corrosion holes, must be taken into account when modeling leakage in a pipe network.

Not only does corrosion actually cause holes to form in pipe walls; corrosion also debilitates the strength of the pipe to the point that other problems may cause significant damage. “It has been recognized by water utility personnel that the majority of the breaks occur at location where the pipe wall has been weakened. Such weakening is the result of graphitic corrosion of cast iron and, although the actual failure may be due to stress, corrosion can be shown to be the real cause” (Fitzgerald, 1968). Often external corrosion is the underlying factor in most leak or break situations.

3.5.2. Soil Corrosivity

For design and corrosion risk assessment purposes, it is desirable to estimate the corrosivity of soils. One of the simplest classifications is based on a single parameter, which is soil resistivity. Soil resistivity is a measure of how much the soil resists the flow of electricity. It is a critical factor in design of systems that rely on passing current through the Earth's surface. A typical fracture surface with severe loss of the pipe wall thickness due to graphitization corrosion is shown in Figure 3-5.

The factors that contribute to the corrosive potential of soil are aeration, moisture content (and/or time of wetness), temperature, pH, and resistivity are the primary telltales. The following is a more detailed description of the manner in which each of the above factors influences soil corrosivity.

Figure 3-5. Pipe Graphitization Corrosion

Aeration –is the amount of air trapped within the soil. Aeration is an important factor in corrosion as it is a factor in water retention and evaporation rates. Well aerated soil is more favorable from a low corrosivity standpoint because this generally leads to lower water retention and higher evaporation rates.

The particle size and gradation- within the soil plays a major role in determining the amount of aeration. Sandy soils are generally desirable, as the relatively large particles allow for better aeration, and facilitate faster evaporation rates after water has been introduced into the soil. A quick way to classify soils in terms of their aeration is by examining their color. Reddish, brown, or yellow soils indicate good aeration, while gray soil is indicative of poor aeration.

pH (acidity) - Soils can have a wide range of acidity, reaching anywhere from 2.5 to10. As pH levels of 5 or below can lead to extreme corrosion rates and premature pitting of metallic objects, a neutral pH of approx. 7 is most desirable to minimize this potential for damage. The intrinsic pH level of a soil can also be affected by rainfall.

Moisture Content & Resistivity – Moisture content is a more important factor in soil corrosivity than any other variable. As water is one of the three components necessary for electrochemical corrosion (the other two being oxygen and metal), corrosion will not occur if the soil is completely dry. Experimental evidence dictates that increased moisture content decreases resistivity of soils, in turn increasing their corrosive potential. When the saturation point of the soil is reached, additional moisture has little or no effect on resistivity

Electrical conduction in soil is essentially electrolytic and for this reason the soil resistivity depends on moisture content, salt content and temperature above the freezing point (Figure 3-6).

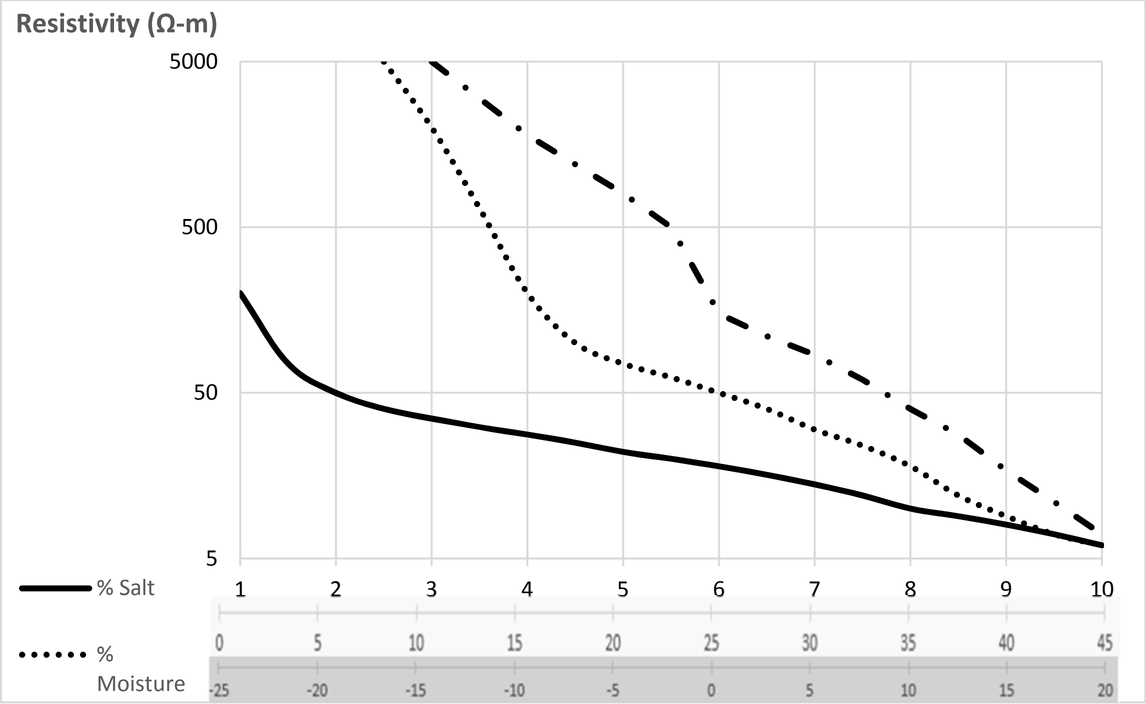


Figure 3-6. Effects of Moisture, Temperature and Salt upon Soil Resistivity

Soil resistivity is one of the driving factors determining the corrosiveness of soil. The soil corrosiveness is classified based on soil electrical resistivity. The soil corrosivity severity is measured in ohm-cm as of the British Standard BS-137 (Table 3-1).

Table 3-1. Soil Corrosivity Severity

Soil Resistivity Range (Ohm-cm)		Soil Corrosivity Severity
From	To	
Zero	1000	Severe
1000	5000	Corrosive
5000	10000	Moderately Corrosive
10000	> 10000	Slightly Corrosive

3.5.3. Excessive Water Pressure

Water pressure is measured according to the force need to move the water from water mains and into pipes. It is measured in pounds per square inch (psi). Water pressure is a term that is used to describe the flow strength of water through a pipe or other type of channel. Water pressure depends on water flow. The more water being pushed through a pipe, the more pressure there will be naturally.

Classically, water pressure ranges between 40 psi and 100 psi. Most water pressure regulators have an adjustable dial that can be used to increase the water pressure in the event that water is merely trickling out of taps, and to decrease the pressure if the water pressure is too strong.

Excessive water pressure can lead to water main breaks and cause leaks in water system. Water pressure management aims to adjust water pressure levels in the supply system to achieve more consistent pressure levels which will reduce the number of water-main breaks, improve the reliability of the water supply system and conserve water.

The rates of the real losses from WDNs vary with pressure (Giustolisi et al. 2008), as pressure changes, the area of leakage particularly at joints and fittings and on non-metal pipes change. The pressure of surges and high pressures influence the rate at which new leaks occur (Lambert 2000). In order to reduce the leakage levels from WDNs, pressure management is now recognized as one of the most efficient and cost effective techniques (Ulanicki et al. 2008).

Water Utilities design potable WDNs to provide a minimum level of service pressure throughout the day at the critical point in the network. The critical point is generally either the highest point in the system or the point most distant from the source although it may be a combination of the two depending upon local topography and various other factors. Since most systems are designed to provide minimum pressure throughout the day, they are generally designed to meet this pressure requirement during periods of peak demand when the friction losses are at the highest and inlet pressures are at their lowest. Because of this design methodology, most systems experience higher pressures than necessary during the remaining nonpeak demand periods. This is evident from the fact that the major burst pipes tend to occur during the late evening and early morning periods when system pressures are at their highest. Therefore, the pressure in a WDN is considerably higher than required during the most of the time increasing the leakage level.

The main objective of pressure management is to minimize the excess pressure in a WDN, which will reduce leakage as well as the frequency of burst pipes. Significant savings can be made where pressure management is extremely successful (Girard and Stewart 2007). With the aid of pressure reducing valves (PRVs) and the recent electronic and hydraulic controllers, it is now possible to apply the pressure management policies that can reduce the leakage to its possible minimum value.

3.5.4. Material Defects

A casting defect is an irregularity in the metal casting process that is undesired. Some defects can be tolerated while others can be repaired. Otherwise, they must be eliminated. They are broken down into the following four main categories.

3.5.4.1. Gas Porosity

Gas porosity is the formation of bubbles within the casting after it has cooled. This occurs because most liquid materials can hold a large amount of dissolved gas, but the solid form of the same material cannot, and so the gas forms bubbles within the material as it cools. Gas porosity may present itself on the surface of the casting as porosity or the pore may be trapped inside the metal, which reduces strength in that vicinity. Nitrogen, oxygen, and hydrogen are the most encountered gases in cases of gas porosity. In aluminum castings, hydrogen is the only gas that dissolves in significant quantity, which can result in hydrogen gas porosity. For casting that is a few pounds in weight the pores are usually 0.0004 inch to 0.02 inch in size. In larger casting, pores can be up to 0.04 inch in diameter.

To prevent gas porosity, the material may be melted in a vacuum, in an environment of low-solubility gases, such as argon or carbon dioxide, or under a flux that prevents contact with the air. To minimize gas solubility, the superheat temperatures can be kept low. Turbulence from pouring the liquid metal into the mold can introduce gases, so the molds are often streamlined to minimize such turbulence. Other methods include vacuum degassing, gas flushing, or precipitation. Precipitation involves reacting the gas with another element to form a compound that will form dross that floats to the top. For instance, oxygen can be removed from copper by adding phosphorus, or aluminum or silicon can be added to steel to remove oxygen. A third source consists of reactions of the molten metal with grease or other residues in the mold.

3.5.4.2. Shrinkage Defects

Shrinkage defects occur when feed metal is not available to compensate for shrinkage as the metal solidifies. Shrinkage defects can be split into two different types: open shrinkage defects and closed shrinkage defects. Open shrinkage defects are open to the atmosphere and, therefore, as the shrinkage cavity forms air subsequently compensates. Open air defects could be either pipes or caved surfaces. Pipes form at the surface of the casting and burrow into the casting, while caved surfaces are shallow cavities that form across the surface of the casting.

Closed shrinkage defects, also known as shrinkage porosity, are defects that form within the casting. Isolated pools of liquid form inside solidified metal, which are called hot spots. The shrinkage defect usually forms at the top of the hot spots. They require a nucleation point, and so impurities and dissolved gas can induce closed shrinkage defects. The defects are broken up into macroporosity and microporosity, where macroporosity can be seen by the naked eye and microporosity cannot.

3.5.4.3. Pouring Metal Defects

Pouring metal defects include misruns, cold shuts, and inclusions. A misrun occurs when the liquid metal does not completely fill the mold cavity, leaving an unfilled portion. Cold shuts occur when two fronts of liquid metal do not fuse properly in the mold cavity, leaving a weak spot. Both are caused by either a lack of fluidity in the molten metal or cross-sections that are too narrow. The fluidity can be increased by changing the chemical composition of the metal or by increasing the pouring temperature. Another possible cause is back pressure from improperly vented mold cavities.

Misruns and cold shuts, are closely related and both involve the material freezing before it completely fills the mold cavity. These types of defects are serious because the area surrounding the defect is significantly weaker than intended. The castability and viscosity of the material can be important factors with these problems. Fluidity affects the minimum section thickness that can be cast, the maximum length of thin sections, fineness of feasibly cast details, and the accuracy of filling mold extremities.

Various ways of measuring the fluidity of a material are used, although it usually involves using a standard mold shape and measuring the distance that the material flows. Fluidity is affected by the composition of the material, freezing temperature or range, surface tension of oxide films, and, most importantly, the pouring temperature. The higher the pouring temperature is, the greater the fluidity will be. However, excessive temperatures can be detrimental, leading to a reaction between the material and the mold; in casting processes that use a porous mold material the material may even penetrate the mold material.

3.5.4.4. Metallurgical Defects

Metallurgical defects could be hot tears or hot spots. Hot tears, also known as hot cracking, are failures in the casting that occur as the casting cools. This happens because the metal is weak when it is hot and the residual stresses in the material can cause the casting to fail as it cools. Proper mold design prevents this type of defect.

Hot spots are areas on the surface of casting that become very hard because they cooled more quickly than the surrounding material. This type of defect can be avoided by proper cooling practices or by changing the chemical composition of the metal.

3.5.5. Water Hammer

Water hammer is a pressure surge or wave resulting when a fluid (usually a liquid but sometimes also a gas) in motion is forced to stop or change direction suddenly (momentum change). Water hammer commonly occurs when a valve is closed suddenly at an end of a pipeline system, and a pressure wave propagates in the pipe. It may also be known as hydraulic shock. This pressure wave can cause major problems, from noise and vibration to pipe collapse. It is possible to reduce the effects of the water hammer pulses with accumulators and other features.

3.5.5.1 Sources of Water Hammer

The first variable is the length of the pipe the water is traveling through. We cannot do much about the length of the pipes, but it is an important factor in creating water hammer, and so it is useful to take a look at it, especially as it relates to the pipe size. For example, in some situations you can force a high rate of flow through a small pipe without problems, provided that the length of the pipe is s few feet short. The shorter the pipe, the smaller it can be. Knowing this will help when we try to identify the source of the water hammer.

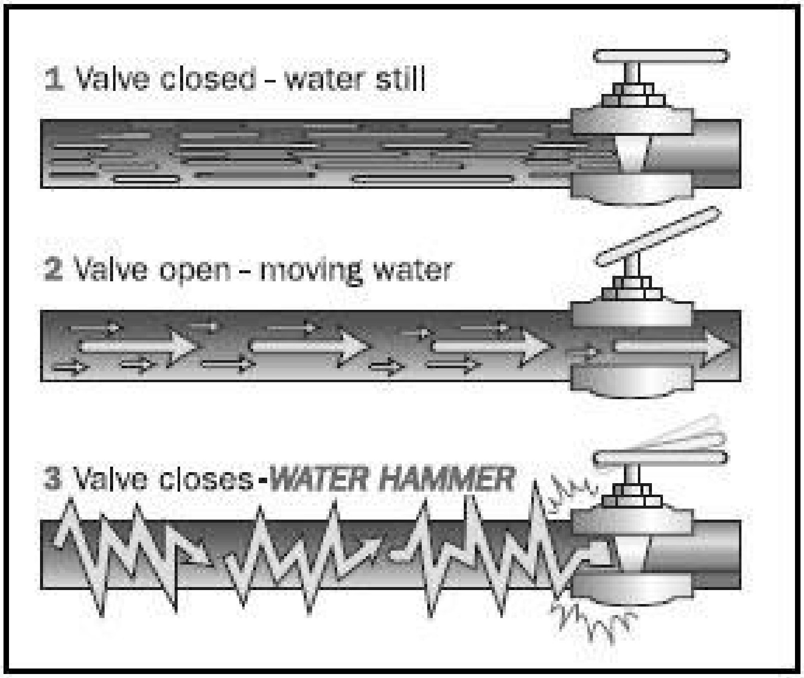


Figure 3-7. Water Hammer (Plumbing Mart 2013)

The second variable is time, or specifically how fast the water is being stopped. When a closing valve is causing water hammer, time is how long it takes for the valve to close. Most valves take several seconds to close. Theoretically, this would not cause a problem, as several seconds is very slow when dealing with water hammer. The valve may take a few seconds to go from full open to full closed, but it has a tendency to snap closed. Realistically, the actual closing time of a typical valve is around 1/2 to 1 second. However, it varies greatly, even when testing the same valve. A valve closes much faster if a higher water pressure is present. It also closes faster as you increase the flow through the valve (increasing the flow creates a greater pressure differential across the valve, which causes it to close faster). Therefore, a valve that would not cause a water hammer problem at a low flow and low pressure will cause all kinds of problems if you increase the flow through the valve and/or the water pressure.

The third factor that influences water hammer is the velocity of the water. The faster the water is traveling in the pipe, the greater the water hammer will be. It is this last factor that is the easiest for us to correct in a sprinkler system, and so most of the suggested solutions for water hammer will be aimed at reducing the water velocity.

3.5.5.2. Controlling Water Hammer

The most effective means of controlling water hammer is a measured, compressible cushion of air that is permanently separated from the water system. Water hammer arresters employ a pressurized cushion of air and a two O-ring piston, which permanently separates this air cushion from the water system. When the valve closes and the water flow is suddenly stopped, the pressure spike pushes the piston up the arrester chamber against the pressurized cushion of air. The air cushion in the arrester reacts instantly, absorbing the pressure spike that causes water hammer.

3.5.6. Excessive loads and vibration from road traffic

In road transport, an oversize load (or overweight load) is a load that exceeds the standard or ordinary legal size and/or weight limits for a specified portion of road, highway or other transport infrastructure, such as air freight or water freight. A load that exceeds the per-axle limits, but not the overall weight limits, is not considered overweight. Examples of oversize/overweight loads include construction machines (cranes, front loaders, backhoes, etc.), pre-built homes, containers, and construction elements (bridge beams, generators, windmill propellers, industrial equipment). The legal dimensions and weights vary between countries and regions within a country.

Heavy surface loads will subject the pipe to high stresses and, therefore, to faster deterioration in the long term. Longitudinal breaks caused by transverse stresses are typically the result of either hoop stress due to pressure in the pipe, ring stress due to soil cover load, ring stress due to live loads caused by traffic, increase in ring loads when penetrating frost causes the expansion of frozen moisture in the ground, or a combination of one or more of the above.

3.5.7. Stray Current Corrosion

Stray current corrosion is caused by current flow through paths other than the intended circuit or by any extraneous current in the earth. Metal structures buried in the ground, like pipelines, can often provide a better conducting path than the soil for earth-return currents from electric rail and tramway systems, electrical installations, and cathodic protection systems on nearby pipes. These routes exhibit higher conductivity than a sheathed earthling cable. Accelerated corrosion of the pipeline may occur at the point where the positive current flow leaves the pipe and enters into the earth.

The term “stray current corrosion” differs from other forms of corrosion in that the current, which causes the corrosion, has a source that is external to the affected structure. It may include the following different types of currents on buried or submerged metallic structures:

- Stray currents from direct current (DC) systems such as railways, trolley bus systems, cathodic or anodic corrosion protection systems, welding equipment in shipyards, and household appliances.

- Interference currents such as HVDC (high-voltage direct current) power lines with a full or partial ground return.
- Stray currents from alternating current (AC) systems such as AC currents from certain household appliances.

Modern ductile iron pipe are manufactured in 18-ft and 20-ft nominal lengths, and a rubber-gasket jointing system may be employed to join successive lengths into a continuous pipeline. Joints with gaskets in this matter offer resistance that may vary from a fraction of an ohm to several ohms but, nevertheless, are of sufficient magnitude that ductile iron pipelines are considered to be electrically discontinuous (and are, therefore, unsuitable for cathodic protection without substantial modification). The rubber-gasket joints limit attack of ductile iron by long-line stray currents, but not necessarily by local currents.

Stray AC current could initiate and/or accelerate corrosion of unprotected metals by exaggerating the potential of the existing anodes and cathodes on the surface, and/or by depolarization of existing bimetallic or galvanic cells. Stray currents also could be introduced in a cast iron pipeline if it ran parallel to high-voltage cables, the alternating current apparently being partly rectified by residual oxide films on the pipe.

3.6. Summary of Main Causes

The main cause of leaks in water mains is external corrosion. “Corrosion is the root, if not the immediate, cause of most breaks in metal pipes. Metals tend to want to return to their ore state" (Walski, 1984). Savic and Walters (1999) suggest that the causes of water main failures may be split into quality, age, type of environment, quality of construction workmanship, and service condition.

Wood et al. (2009) reported that in addition to historical pipe breakage data, factors that may be important for predicting water main breaks may need to be obtained, including soil type, surface, bedding, and backfill material, type of road usage, or typical flow in area of break. The causes of water leakage and its impact and severity degree are summarized in Table 3-2.

Table 3-2. Impact of Causes of Water Leakage and Severity Degree

No.	Cause	Impact	Degree of Impact
1	Corrosion	Deterioration of Pipes	High
2	Soil Corrosivity	Loss of pipe wall thickness	High
3	Excessive Water Pressure	Water main breaks	High
4	Material Defects	Pipe Break in Early Phase	Low
5	Water Hammer	Pipe collapse	Moderate
6	Excessive loads and vibration from road traffic	Pipe higher stress	Low
7	Stray Electric Current	Initiate and/or accelerate corrosion of unprotected metals	Low

Chapter 4. Proposed Model

In this chapter a probabilistic model that estimates the recoverable leakage of water distribution networks is proposed. The model examines the key causes that lead to high percentages of leakage in the different components of the water distribution network. Directional cosines analysis examines the importance of the random variables in the new probabilistic estimation model, i.e., the sensitivity of the model to perturbation in the random variables in the new probabilistic estimation model.

4.1. Problem Definition

The Literature Review, as provided in Chapter (1), revealed several models that deal with the leakage of water distribution networks forecasting, estimating, and implementing. However, rather little work has been performed in developing a model that estimates the recoverable leakage of water distribution networks when considering the different factors that may be important for predicting a more accurate estimate for the recoverable leakage.

Leakage is usually the major cause of water loss in water distribution systems. Environmental Protection Agency (EPA) in 2011 reported 240,000 water main breaks per year in the United States. The number of breaks increases substantially near the end of the respective system's service life. Large utility breaks in the Midwest increased from 250 per year to 2,200 per year during a 19-year period.

The City of Baltimore, Maryland in 2003 reported 1,190 water main breaks, which is more than three per day. The U.S. Geological Survey estimates that the water lost from water distribution systems totals 1.7 trillion gallons per year at a national cost of $2.6 billion per year (EPA, 2003).

In addition to the historical pipe breakage data, Wood et al. (2009) reported that it may be important to discover the causes in order to better predict water main breaks. These causes include the soil type, corrosivity, surface, bedding, backfill material, type of road usage, and typical flow in the area of the break.

Determining the key causes that lead to high percentages of leakage in different components of the water distribution network, will help water utilities perform a predictive or preventive action plan rather than reacting to the failure and losses occurring due to the leakage of the water distribution network.

4.2. Model Definition

The proposed probabilistic estimation model for the recoverable leakage of water distribution networks, as shown in Figure 4-1, was developed by receiving physical and historical data from owners of WDNs. The analysis of the literature review was used in determining the main causes that increase the network leakage.

The model received the deterministic and stochastic description of the leakage of the different distribution networks received from the water utility companies included in the research survey. The soil, water, and network characteristics for each of these water distribution networks were collected, analyzed, and tallied.

The developed model was validated using the detailed leakage component parameters from two case studies as provided in Chapter 5. The output results were analyzed and the statistical parameters of the model were then used in a Monte Carlo simulation to generate output statistical distributions. The generated output was analyzed to validate the developed model.

4.3. Development of a Probabilistic Model

Break prediction models have been developed in order to help the water industry understand how pipes deteriorate and the pipe break potential in the future. These models are typically grouped into two classes: statistical and physical–mechanical models (Kleiner and Rajani, 2001). Statistical models use historical pipe break data to identify the break patterns and extrapolation of these patterns to predict future pipe breaks, or degrees of deterioration. Physical–mechanical models predict failure by simulating the physical future pipe breaks or degrees of deterioration. Physical–mechanical models predict failure by simulating the physical effects and loads on pipes and the capacity of a pipe to resist failure over time (Rajani and Makar, 2000).

Break prediction models presented in the literature are either deterministic or probabilistic models (Kleiner and Rajani, 2001). Deterministic pipe breakage models are developed by fitting pipe breakage data to various time dependent equations. On the other hand, probabilistic models treat all variables as random variables, with specific probability distributions, in order to predict both the failure potential and the extent of failure

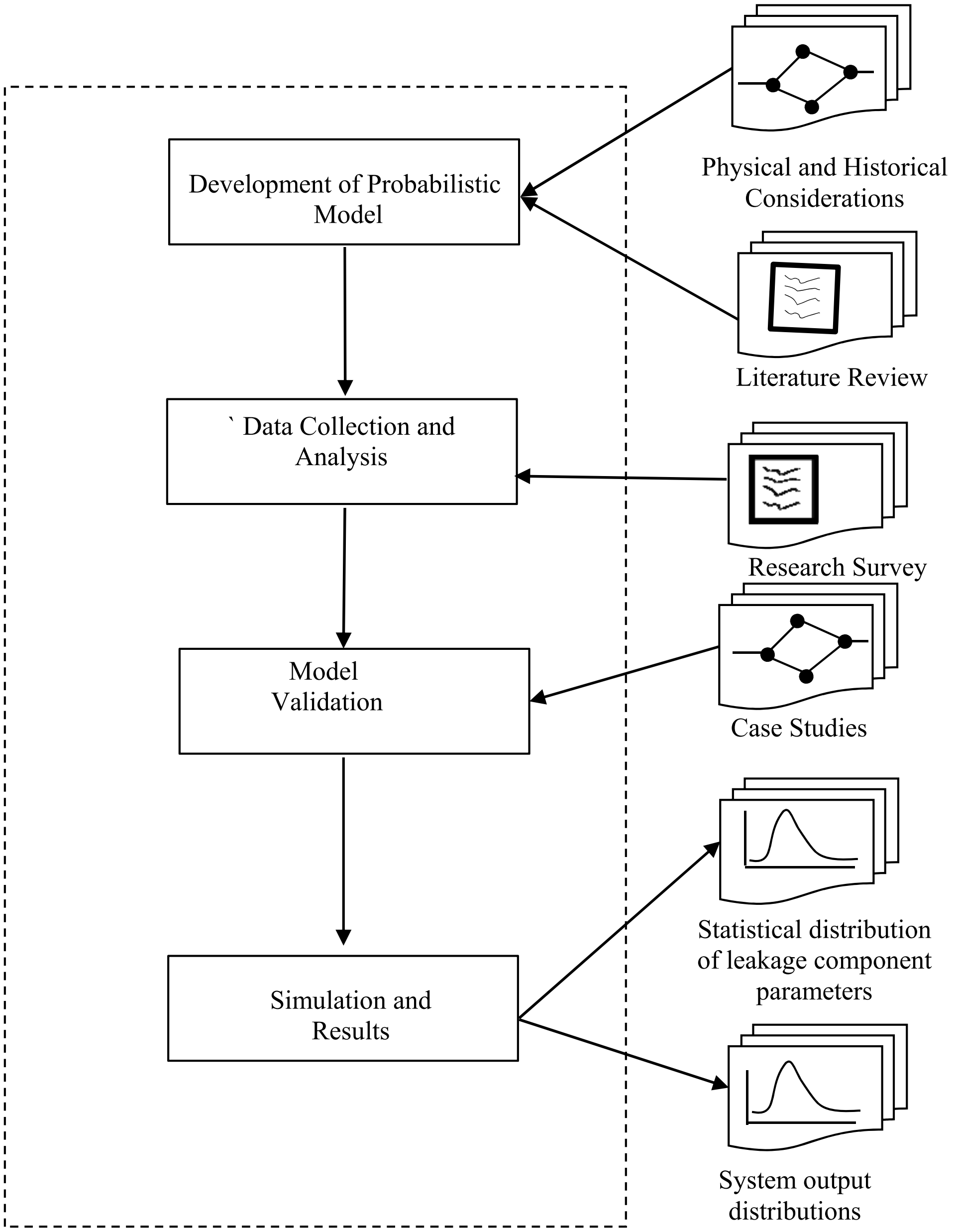


Figure 4-1. Proposed Model

The Recoverable Leakage is the difference between the Current Annual Volume of Real Losses (CARL) and the Unavoidable Annual Real Losses (UARL) Equation 4-1. In this dissertation, a probabilistic model for the recoverable leakage is proposed in which the pressure, Connection Density, Corrisivity and length of pipes are to be modeled as random variables, as will be discussed section 4.4.

$$\text{Recoverable Leakage} = \text{CARL} - \text{UARL} \tag{4-1}$$

System Input Volume	**Authorized Consumption**	**Billed Authorized Consumption**	**Billed Metered Consumption**	**Revenue Water**
			Billed Un-metered Consumption	
		Unbilled Authorized Consumption	**Unbilled Metered Consumption**	**Non-Revenue Water (NRW)**
			Unbilled Un-metered Consumption	
	Water Losses	**Apparent Losses (Commercial Losses)**	**Unauthorized Consumption**	
			Customer Meter Inaccuracies and Data Handling Errors	
		Real Losses (Physical Losses)	**Storage Leaks and Overflows from Water Storage Tanks**	
			Service Connections Leaks up to the Meter	

Figure 4-2. AWWA Water Balance Table

The CARL is the physical volume of water that is lost from the system (shown in gray in Figure 4-2). The water that is lost is due to leaks in the transmission and distribution systems, losses at the utility’s storage tanks, and leaks in the service lines from the main to the point of customer usage. CARL can be estimated by adding the burst leakage and the background leakage as shown in Equation 4-2 and Figure 4-3. The CARL is given in gallons/day averaged over a one-year period.

$$\text{CARL} = \text{Burst Leakage} + \text{Background Leakage} \tag{4-2}$$

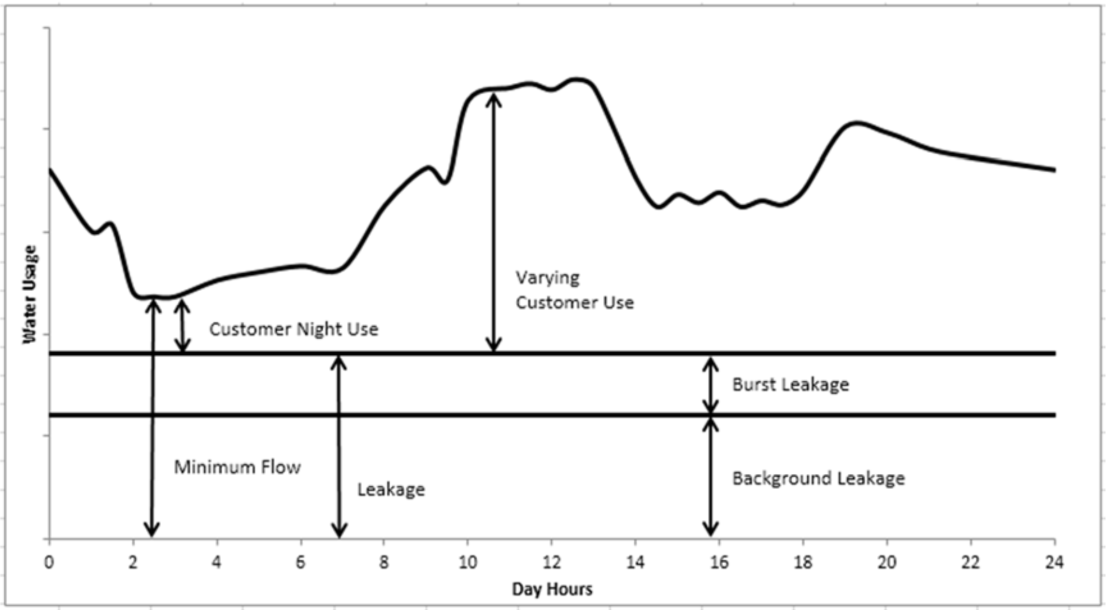


Figure 4-3. A Typical 24-hour Flow Profile of the Components of Leakage

Water utility personnel recognized that the majority of breaks occur at locations where the pipe wall has been weakened. Such weakening is the result of graphitic corrosion. Although the actual failure may be due to stress, corrosion can be shown to be the real cause (Fitzgerald, 1968). External corrosion is the underlying factor in most leak or break situations. Although corrosion is the most critical factor in pipe leakage, several other factors work either with corrosion or separately to cause leaks or breaks in water mains. These factors include physical pipe characteristics, soil type and behavior, water main pressure in the system, and installation procedures (Stathis 1999).

It is evident from the previous literature review that International Water Association's (IWA) model (Appendix C) for estimating UARL does not account for soil corrosivity, pipe burial depth, and climate; these have significant effects for the leakage calculation in water distribution systems.

4.4. Data Collection and Analysis

Data Collection were divided into two phases. The first phase was to perform a research survey to collect WDN leakage results from different water utility companies throughout the United States with different climate and soil characteristics.

The second phase was to perform a full data collection including detailed pumped and consumed water, break incidents, and water audit for two case studies: the City of Baltimore-Department of Public Works (COB) and Washington Suburban Sanitary Commission (WSSC) as described in Chapter 5. These data will be used for the validation of the developed model.

4.4.1. Research Survey

To summarize the current status of the municipal water accounting and water loss management practices of utilities around the US, an informational survey needed to be conducted. A written survey was sent by electronic mail in the spring of 2013 to a sample from the group of public water systems (PWS) in 45 different states in the US.

A survey called "Leakage of Water Networks Research" was conducted in order to include several questions regarding water loss and accounting that might be considered. The questions were planned to be as simple as possible to maximize the number of responses from the water utility companies. The final survey is attached in Appendix A.

The survey was designed to be completed by a person within the utility management that would have good knowledge of the water loss accounting as well as the water loss control measures that the utility currently practiced. This preference was reinforced by addressing the survey to the correct personnel in each of the utility companies. The nature of the survey and the wording of the questions assumed that the person responding had specific experience and knowledge in the water utility management field, but care was taken to prepare the questions that kept jargon or acronyms to a minimum.

The questions were written as neutral as possible to minimize bias and avoid leading the respondent toward any specific answer. The survey was sent to a sample of public water systems in 45 US states.

The data from the received surveys were entered into an Excel file. Then, each of the categorical responses to the survey questions was analyzed by determining the proportion of positive, negative, and non-responses. The data were reported and analyzed for easier comparison. The survey responses are listed in Table B-1 in Appendix B.

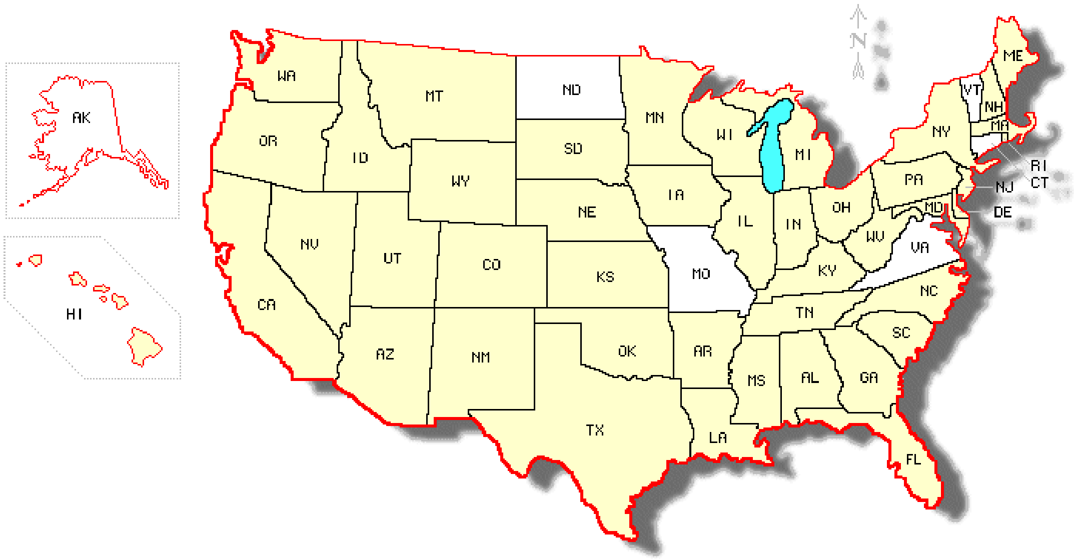


Figure 4-4. US States - Survey Sent

4.4.1.1. Survey Response

In the research survey, 212 surveys were e-mailed to water utility companies in 45 US states and 37 responses were received from 21 states. The recap totals of the target group, sample group, and surveys e-mailed and received are shown in Table 4-1.

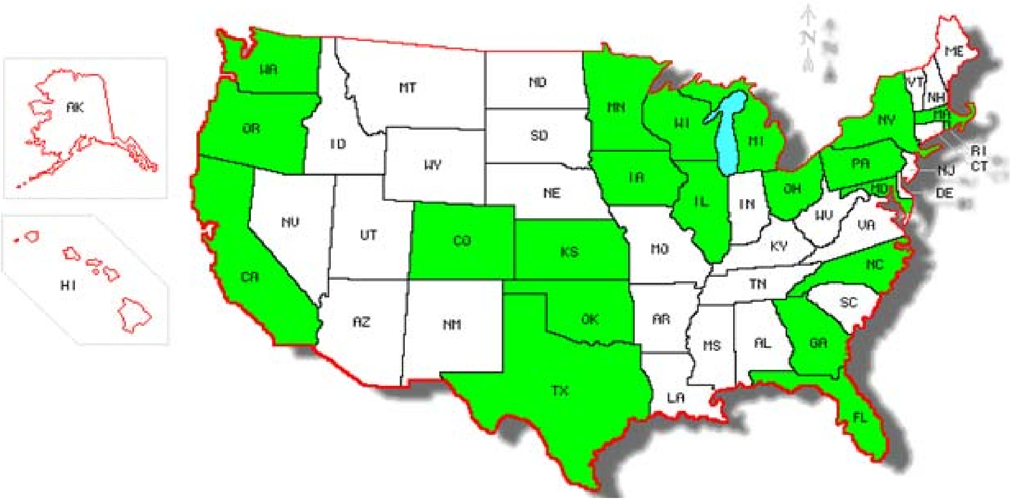


Figure 4-5. US States - Received Data

4.4.1.2. Research Survey Analysis

The received data were collected, reviewed, and checked for accuracy. One of the responses showed a negative loss, which indicates inaccurate data; these data were eliminated from the analysis (shaded in Table 4-1).

To analyze the response received from the utility companies, the following random variables were collected for each utility company:

- Length of Miles of Water Mains
- Number of Service Connections
- Length of Customer Service Line
- Average Pressure
- Average Soil Resistivity (Corrosivity)
- Current Annual Real Loss (CARL)
- Apparent Loss.

One of the most significant factors affecting the level of leakage in a water distribution network is the general condition of the mains and service pipes. The condition of the infrastructure is inherited from previous generations, and it cannot be improved significantly without major capital investment in renewal and refurbishment works (Farley and Trow, 2003). IWA's component analysis equations show that the greatest proportion of annual real loss volume occurs in service connections. This conclusion is supported by comparisons of new leak frequencies per mile of pipe, as well as practical experience (Lambert et al.).

Table 4-1. Research Survey Collected Data

Utility Company No	State	Produced	Metered	% Loss	Units	Year	Audit
1	OH	4,748,873,000.00	4,177,601,994.00	12.03%	MG/yr	2012	Yes
2	PA	3,101,414,893.00	2,668,453,300.00	13.96%	MG/yr	2012	No
3	TX			8.00%	MG/yr	2010	No
4	PA	23,061,592.00	18,618,942.00	19.26%	GPD	2012	No
5	CO			6.10%	MG/yr	2011	No
6	CO			5.03%	MG/yr	2011	No
7	TX	646,725,204.00	524,344,200.00	18.92%	MG/yr	2003	No
8	KS	22,303.00	19,870.00	10.91%	MG/yr	2012	Yes
9	Iowa	16,981.16	15,309.76	9.84%	MG/yr	2012	Yes
10	GA	24,704.00	22,080.00	10.62%	MG/yr	2012	Yes
11	GA	21,195.70	18,567.50	12.40%	MG/yr	2012	Yes
12	IL	8.20	7.05	14.02%	MG/yr	2012	No
13	MN	497,183,000.00	438,609,206.00	11.78%	MG/yr	2012	No
14	MA	195.00	188.00	3.59%	MGD	2011	No
15	FL			2.76%	MG/yr	2012	Yes
16	CA	20,462.60	19,416.00	5.11%	Acre-feet	2012	Yes
17	NY	59.20	52.30	11.66%	MGD	2012	No
18	Ohio	4,780,000.00	3,918,332.00	18.03%	Thous. Gallons	2012	No
19	OK	42,404,640.00	35,554,308.00	16.15%	Thous. Gallons	2012	Yes
20	NY	38.22	32.76	14.29%	MG/yr	2012	No
21	RI	26,025,548.00	19,468,461.00	25.19%	MG/yr	2012	No
22	MD	7,845.00	7,318.00	6.72%	MG/yr	2011	Yes
23	MI	12,653.35	11,752.85	7.12%	MG	2012	No
24	NC	9,781,061,000.00	8,960,422,709.00	8.39%	Gallons	2012	No
25	WI	1,782.00	1,542.00	13.47%	MG/Yr	2011	Yes
26	NC	101.25	82.51	18.51%	MGD	2012	No
27	WA	120.50	111.70	7.30%	MGD	2012	No
28	WA	1,223,300.00	1,243,000.00	-1.61%	Gallons	2012	No
29	CA	20,586.20	19,685.40	4.38%	Acre-ft/year	2012	Yes
30	WA	2,451,174.00	2,335,441.00	4.72%	Cubic Feet	2012	No
31	NC	2,514.95	2,322.10	7.67%	MG/yr	2012	Yes
32	OK	398,574.92	379,841.13	4.70%	MG/yr	2012	No
33	OR	999,311,800.00	980,912,706.00	1.84%	MG/yr	2012	No
34	OH	50,286.05	41,554.02	17.36%	MG/yr	2011	No
35	TX	11,719,880,344.00	9,845,099,200.00	16.00%	MG/yr	2012	No
36	GA	572.82	552.71	3.51%	MG/Yr	2013	Yes
37	MD	1,855.10	1,448.30	21.93%	MG/yr	2012	Yes

Table 4-2. Percentage of Water Loss Statistics for the Entire Survey Sample

Parameter	Percentage of Water Loss
Median	10.77%
Mean	10.92%
Standard Deviation	5.96%

Detailed data for 11 of the utility companies from the research survey that supplied water detailed audits data of their water mains and service pipes were collected and are summarized in Table 4-3. Those data were used to calculate the Current Annual Real Loss (CARL) and the Infrastructure Leakage Index (ILI) of the WDNs. One of the results of the apparent loss had an outlier value of 99.97% that was probably due to a calculation error in estimating the apparent losses. The median value of the apparent loss percentage is 23.12%, which is close to the average US values as shown in Figure 4-6.

Table 4-3. Water Loss Data for Research Survey

No.	Total Water Loss (MG/Yr)	Apparent Loss (MG/Yr)	Apparent Loss %	Real Loss (CARL)	CARL %
1	161.4	55.70	34.51%	105.7	65.49%
2	1249.87485	1249.55	99.97%	0.325851	0.03%
3	2615.404	312.99	11.97%	2302.415	88.03%
4	569.649	18.95	3.33%	550.703	96.67%
5	1,037.35	251.36	24.23%	785.99	75.77%
6	2623.901	743.58	28.34%	1880.326	71.66%
7	217.725	47.92	22.01%	169.801	77.99%
8	293.518107	179.37	61.11%	114.148	38.89%
9	2,433.10	455.58	18.72%	1977.515	81.28%
10	20.11	5.21	25.91%	14.896	74.09%
11	588.14	238.42	40.54%	349.7259	59.46%

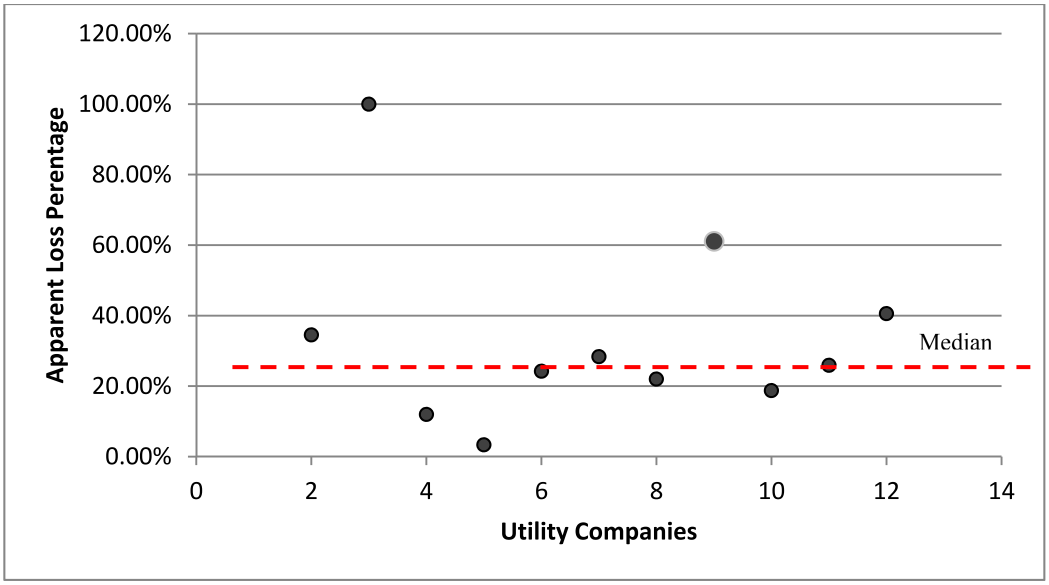


Figure 4-6. Apparent Loss Percentage of Total Water Loss

Table 4-4. Values of Connection Density and CARL/Length of Mains

No.	Connection Density (#/miles)	CARL/Length of Mains (MG/Yr/Mile)
1	86.95	1.36
1	53.57	0.27
2	0.70	0.01
3	60.03	0.79
4	23.53	0.32
5	57.40	0.56
6	65.22	0.51
7	77.46	0.85
8	87.68	0.29
9	51.87	0.75
10	35.15	0.25
11	88.64	0.59

Two important analyses, Connection Density and the CARL/Main Length, were performed in Table 4-4 for the utility companies in the research survey to analyze the ILI and UARL values. By analyzing the different components of the UARL in Equation 4-5 with the ILI using the data received from the utility companies in Table 4-4, the following observations were found:

1. The average operating pressure has a negative relationship with the infrastructure leakage index as shown in Figure 4-7.

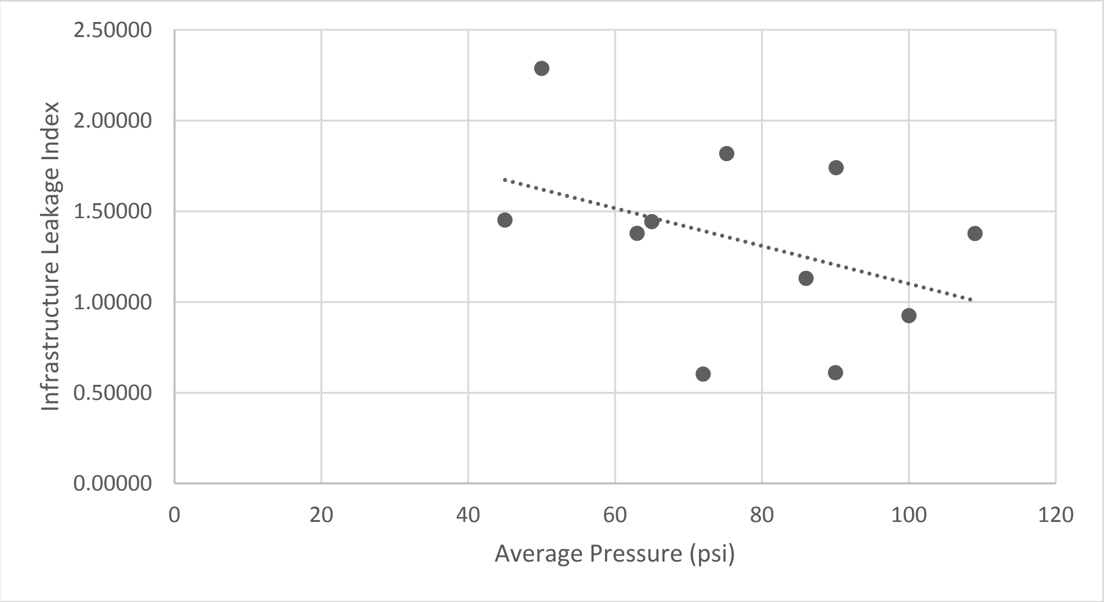


Figure 4-7. Infrastructure Leakage Index versus Average Pressure

2. The variation of ILI shows a positive relationship with connection density: the higher the density, the higher the ILI (Figure 4-8). Pearson, 2010, had identified that it was likely the result of:
    a. Difficulty of leak detection in more complex urban areas

b. The longer repair times due to road access restrictions

c. The connection asset lifetime, especially for the old connections

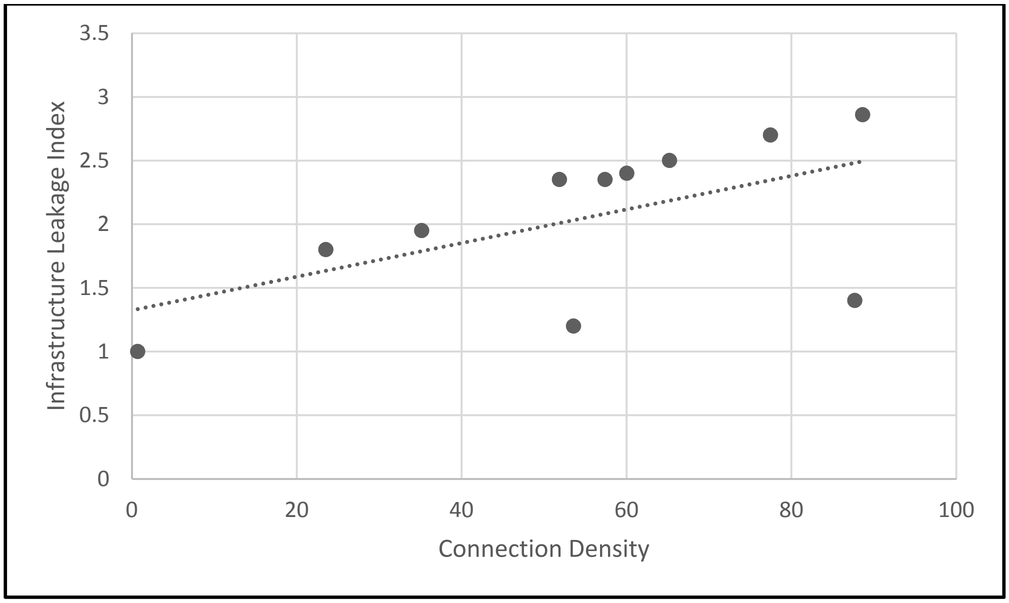


Figure 4-8. Connection Density versus Infrastructure Leakage Index

3. Besides the average age of the network, the structure of the distribution system is the most important influencing factor for the performance indicator Infrastructure Leakage Index (ILI). With increasing service connection density, as well as with increasing network delivery rate, the losses per mile of main length increase (Kölbl, 2014). CARL divided by the Main Length has a positive relationship on ILI, as shown in Figure 4-9.

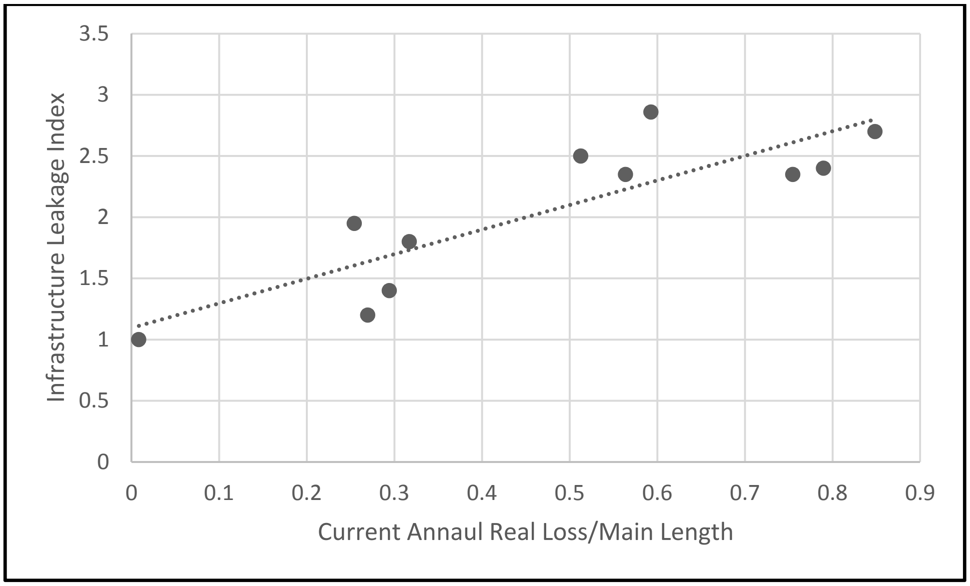


Figure 4-9. Current Annual Real Loss/ Main Length versus Infrastructure Leakage Index

4. The degree of common variation between ILI and the percentage of CARL is low. No correlation between the two variables was observed as shown in Figure 4-10. Low percentage of water loss is not necessarily an indication for good real loss management (Winarni, 2012).

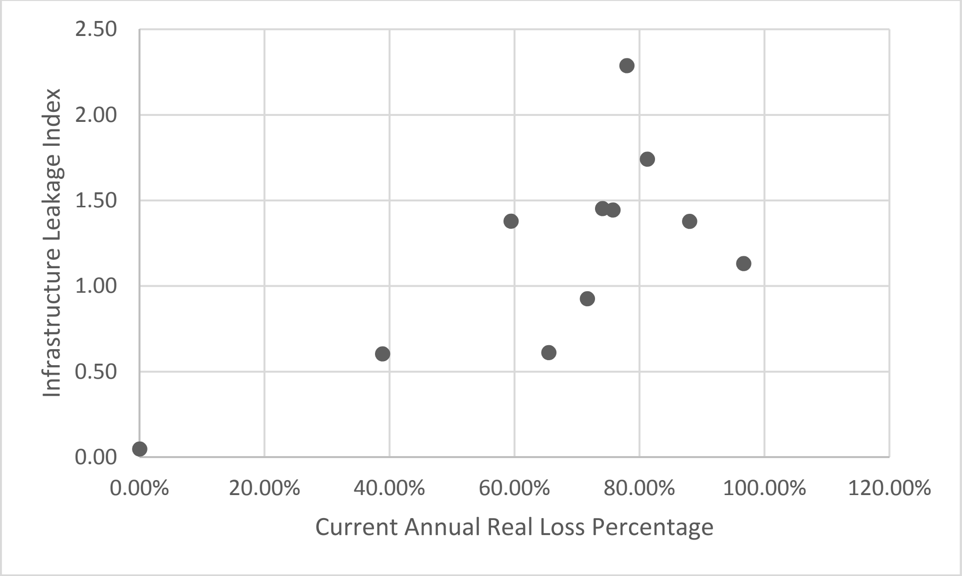


Figure 4-10. Current Annual Real Loss vs. Infrastructure Leakage Index

5. The IWA's equation introduced in Equation 3-3 used for calculating UARL reintroduced here in Equation 4-3 does not account for soil corrosivity.

$$\text{UARL} = (5.41\ \text{Lm} + 0.15\ \text{Nc} + 7.5\ \text{Lp}) \times \text{P} \qquad (4\text{-}3)$$

4.4.2 The Corrosivity Factor

The IWA's equation used for calculating UARL is based on clearly stated auditable assumptions for the frequencies and durations of the different types of leaks and their typical flow rates related to pressure. The actual calculations are detailed in Appendix C. The UARL equations require data on four key system-specific variables:

- Length of mains
- Number of service connections
- Location of the customer meter upon service connection (relative to the property line, or curb-stop in North America)
- Average operating pressure (when the system is pressurized).

Equations for calculating UARL for individual systems were developed and tested by the IWA Water Losses Task Force (Lambert et al., 1999), allowing for:

- Background leakage – small leaks with flow rates too low for ultrasonic detection if non-visible
- Reported leaks and bursts – based on frequencies, typical flow rates, target average durations

- Unreported leaks and bursts – based on frequencies, typical flow rates, target average durations
- Pressure – leakage rate relationships (a linear relationship being assumed for most large systems).

This equation (Equation 4-3) is based on the component analysis of Real Losses for well-managed systems with good infrastructure. It has proved to be robust in diverse international situations (Lambert and McKenzie, 2002). It is a reliable predictor of real losses for systems with more than 5,000 service connections, connection density (Nc/Lm) of more than 20 per km, and an average pressure of more than 25 meters.

The component analysis employed is a statistical procedure that uses transformation to convert a set of observations of possibly correlated variables into a set of values of linearly uncorrelated variables. It is evident from the literature review that IWA's model for estimating Unavoidable Annual Real Losses (UARL) does not account for soil corrosivity. The UARL equation can be modified by adding a new soil corrosivity factor (Cr) that takes the soil corrosivity into consideration with the main length and the number of connections as shown in Equation (4-4). If the effect of soil corrosivity is known, it can provide useful information for the selection of pipeline paths, methods of corrosion control during design, and the maintenance of underground metallic structures.

$$UARL = \left((5.41Lm + 0.15Nc)Cr + 7.5Lp\right) \times P \qquad (4\text{-}4)$$

4.5. Regression Analysis

Underground corrosion analysis is difficult to perform when a variety of parameters are involved in the corrosion process. Statistical methods provide a rational approach to this problem, interpreting the results of corrosion experiments and tests.

Linear Regression is used in this section to develop a relationship between the UARL and the soil corrosivity. The variables tested in this analysis are the Network Connection Density, UARL, Average Pressure, Water Main Length, and Length of Service Connections from the research survey. The criterion variable is the average soil resistivity (SR) and the predictor is the corrosivity factor (Cr).

Table 4-5. Water Distribution Networks Characteristics

No.	Length of Mains (miles)	Number of Service Connections	Length of Customer service Line (ft)	Average Pressure (psi)	Avg. Soil Resistivity (ohm-cm)
1	392	21,000	0	90	25000
2	40	28	0	85	3300
3	2,915	174,977	0	109	50000
4	1,737	40,864	35	86	12500
5	1,394	80,021	30	65	6700
6	3,667	239,146	0	100	100000
7	200	15,500	30	50	6700
8	388	34,020	0	72	50000
9	2,620	135,893	0	90.1	6700
10	59	2,060	0	45	25000
11	590	52,300	0	63	55000

For design and corrosion risk assessment purposes, the simplest estimate for soil corrosivity is based on a single predictor, soil resistivity (SR). Soil resistivity is a measure of how much the soil resists the flow of electricity. It is a critical factor in the design of systems that rely on passing current through the Earth's surface. The average soil resistivity for the different water utility companies was collected in ohm-cm as shown in Table 4-5.

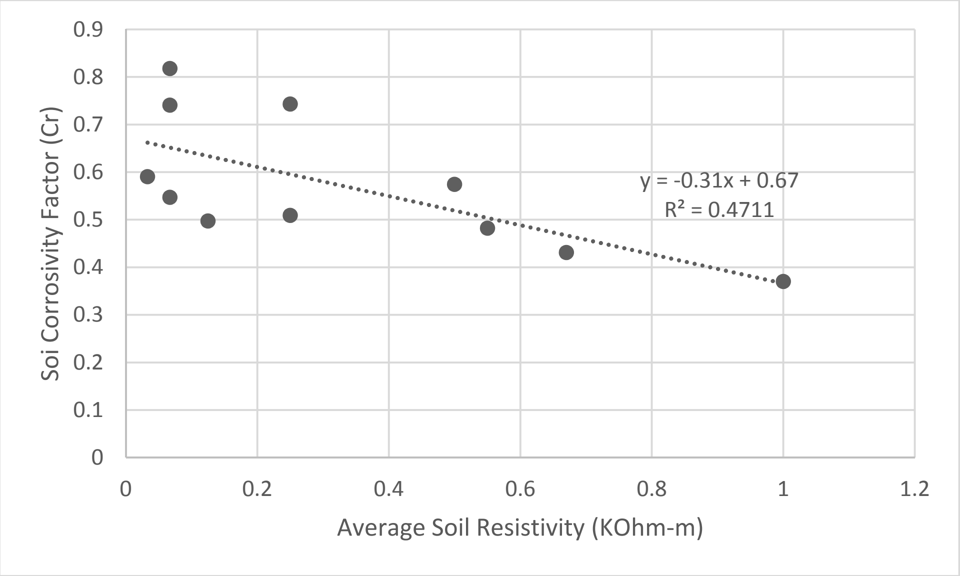


Figure 4-11. Average Soil Resistivity versus Soil Corrisivity Factor

4.5.1. Visual Inspection of Regression Graphical Data

After analyzing the research survey, a scatter plot was generated between the Soil Resistivity (SR) in KOhm-m and Soil Corrosivity Factor (Cr) (Figure 4-11). By performing a visual graphical analysis of the data on Figure 4-11 the following was identified:

1. The degree of common variation is moderate, evidently as the soil resistivity increases the corrosivity factor decreases.
2. The range of the sample data points is wide which gives stability to the relationship and the capability of the data sample to represent the distribution of the population.

3. Outliers are not observed.
4. A linear form of relationship is evident between the soil resistivity and the corrosivity factor.
5. A negative relationship was observed between these parameters in the scatter plot.

4.5.2. Evaluation of the Coefficients of the Regression Equation

The analysis of the Corrosivity Factor (Cr) resulting from Equation (4-5) showed that Cr had a Gaussian distribution with a mean value of 0.52 and standard deviation of 0.21.The regression coefficients of relating the response variable to the predictor variable were evaluated. The least squares regression equation has an intercept of 0.67 and a slope of -0.31, as shown in Equation (4-5).

$$Cr = 0.67 - 0.31 \times SR \tag{4-5}$$

The interpretation of the regression coefficients is:

- When the soil resistivity value is equal to zero, the soil corrosivity factor is equal to 0.67.
- When soil resistivity increases by one unit, the soil corossivity factor decreases by 0.31 units.

Regression data analysis were performed using Excel with a sample size of the 11 utility companies. To evaluate the reliability of the derived regression model the following were observed:

1. The coefficient of determination R^2 had a value of 0.47, meaning that the soil

resistivity explains 47% of the variability of the corrosivity factor Cr while approximately 53% remains unexplained. Thus, increasing the sample size for more utility companies should be examined in further studies to obtain a better idea of linear association between the random variables.

2. The standard error of estimate measures the dispersion (or variability) around the line of means. The standard error of estimate (S_e) was calculated as 0.108. When comparing Se with the bounds of zero and the standard deviation of the corrosivity factor (S_Y), we find $0<S_e=0.108<S_Y=0.21$. Therefore the regression analysis has improved the reliability of prediction.
3. For testing the F statistic for the analysis of the variance: H_0: No statistically significant relationship between soil resistivity and corossivity versus. H_a: A statistically significant relationship between soil resistivity and corossivity (or soil resistivity is a significant predictor of corossivity). The p-value of the F(1,9) statistic in the analysis of variance was 0.02, which indicates that at the $\alpha=0.05$ significance level, we could reject the null hypothesis (H_0) and conclude that the relationship between the two variables is significant. This indicates that the model applied can be used to predict the corrosivity factor from soil resistivity.
4. Regarding the normality of the response variable, the Shapiro-Wilk test for normality, yielded a p-value=0.5075>0.05 so we fail to reject the null hypothesis that the response is normally distributed and therefore the normality assumption is not violated.

4.5.3. Hypothesis Testing of the Slope Coefficient

The slope coefficient (b_1) represents the effect of change in the predictor variable on the response variable. The hypothesis testing is useful to indicate the quality of the regression. The null hypothesis $H_0: \beta_1 = 0\ versus\ the\ two\ sided\ alternative H_a: \beta_1 \neq 0$ will be tested. The null hypothesis will be tested with the test statistic equation (Equation 4-6), where Se,b_1 is the error variance of the slope.

$$t = \frac{b1 - \beta 1}{Se,b1} \tag{4-6}$$

The degrees of freedom of this statistic are df = n-2 = 11-2 = 9, and the standard error of the slope is 0.1081, the value of the test statistic was computed as -2.83 with a p-value=0.02. For a level of significance of α=5% and 9 degrees of freedom, we reject the null hypothesis since p-value < α. We therefore conclude that soil resistivity is a significant predictor of corossivity.

In addition to the hypothesis test for the slope, we can observe the 95% confidence interval. The 95% confidence interval for the slope is (-0.5507, -0.0615) which means that we are 95% confident that the true population slope is between these two numbers. Since this confidence interval does not include the value of zero, it is equivalent to a two-sided hypothesis test with 5% significance level that rejects the null hypothesis and concludes that the slope is significant.

4.5. Sensitivity Analysis of Random Variables

Sensitivity analysis is used to test the robustness of the results of the proposed model. The need for the sensitivity analysis stems from acknowledging the presence of uncertainty in the model. Directional Cosines was used for the assurance of the level of sensitivity of the random variables of the UARL equation (Equation 4-4). The statistical parameters of the different random variables obtained from the research survey results are shown in Table 4-6.

Table 4-6. Statistical Parameters of the Random Variables

Random Variables	Mean Value	Standard Deviation	Distribution Type
Main Length (Lm)	1,272.86	1,290.11	Normal
Number of Connections (Nc)	72,346.27	78,327.66	Normal
Corrosivity Factor (Cr)	0.52	0.21	Normal
Length of Service Connections (Lp)	73.97	150.89	Normal
Pressure (P)	77.77	20.39	Normal

The directional cosines can be measured by Equation 4-7, where α_i is the directional cosine, x are the random variables in the UARL equation and σxi is the standard deviation of the i^{th} random variable.

$$\alpha\text{i} = \frac{\frac{\text{df}}{\text{dxi}}\sigma x_{\text{i}}}{[\sum_{i=1}^{n}(\frac{\text{df}}{\text{dxi}}\sigma x_{\text{i}})^2]^{1/2}} \tag{4-7}$$

Table 4-7. Directional Cosines for Random Variables

Random Variables	Directional Cosine (α_i)
Number of Connections (Nc)	0.72
Corrosivity Factor (Cr)	0.44
Main Length (Lm)	0.43
Pressure (P)	0.30
Length of Service Connections (Lp)	0.13

By substituting in Equation 4-7 and using the mean values as the design points and evaluating the directional cosines values output in Table 4-7 we found the following:

1. The larger the α_i the greater the importance of the random variables. The highest directional cosine values were for the Number of Connections (Nc) and the Corrosivity Factor (Cr).
2. It is evident that after adding the corrosivity factor Cr to the UARL equation, the directional cosine value (0.44) for it was the second highest in importance (Figure 4-12) which shows the great impact of the corrosivity in measuring the UARL.

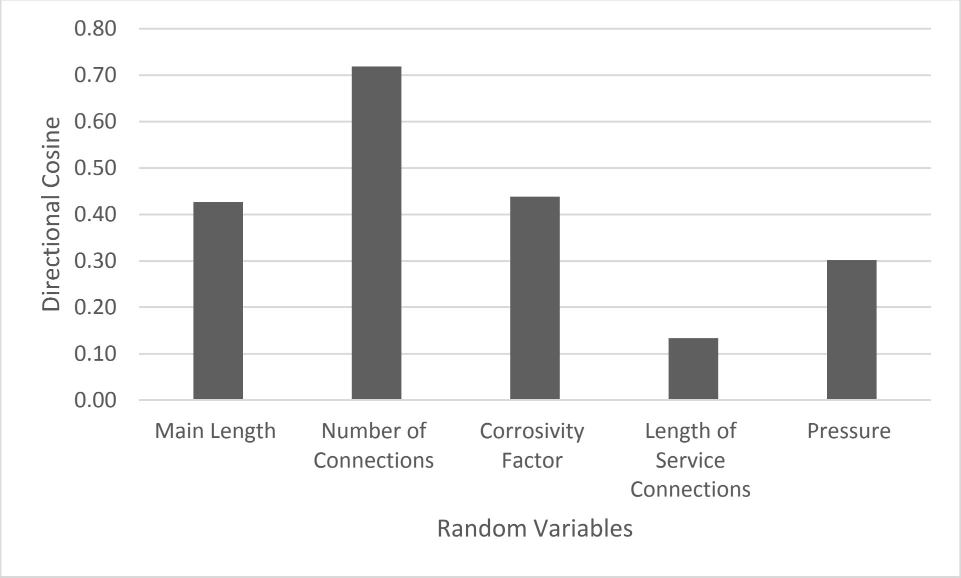


Figure 4-12. Directional Cosines for the Random Variables

3. All of the directional cosine values were between zero and one, and the sum of their squares is one thereby confirming the appropriate implementation of the directional cosine analysis.

The current analysis investigated the extent to which soil corrosivity would impact the estimate of the UARL using the soil resistivity to determine the degree of impact. From these results, it is evident that the corrosion behavior of water networks in soil is closely related to environmental factors. It is possible to extract key variables related to corrosion such as soil corrosivity using the linear regression correlation analysis technique.

This gives us the ability to modify the UARL equation (Equation 4-4) to give a better estimate for the ILI and the recoverable leakage model. This model will be validated using the two Case Studies and a simulation of the results will be performed in Chapter 5 using Monte Carlo Simulation.

Chapter 5. Model Validation (Case Studies)

In this chapter, deterministic and stochastic data were collected for the elements of the water distribution networks under analysis for two case studies; the City of Baltimore – Department of Public Works (COB) and the Washington Suburban Sanitary Commission (WSSC). Those two case studies were picked because they are both large and old cities with an aged infrastructure.

The collected data were the detailed pumped and consumed water, pipe description, pipe sizes, break frequency, break period, break water flow, type of leak, amount of leak, water characteristics, soil characteristics, pipe average pressure, pipe lengths, flow rate, number of service connections, and connection density.

5.1. City of Baltimore – Department of Public Works

Baltimore is the largest city in the U.S. state of Maryland and the 26th largest city in the country. It is located in the central area of the state along the tidal portion of the Patapsco River, an arm of the Chesapeake Bay. It has an area of 92.05 sq. miles and a population of 621,000.

The Department of Public Works in City of Baltimore supplies high quality drinking water to 1.8 million customers in Baltimore Metropolitan Area and it protects and manages three Reservoir Watersheds; Loch Raven, Liberty, and Prettyboy. The bureau operates three Water Filtration Plants; Montebello I, Montebello II, and Ashburton and filters and distributes an average of 225 million gallons of drinking water daily (Figure 5-1).

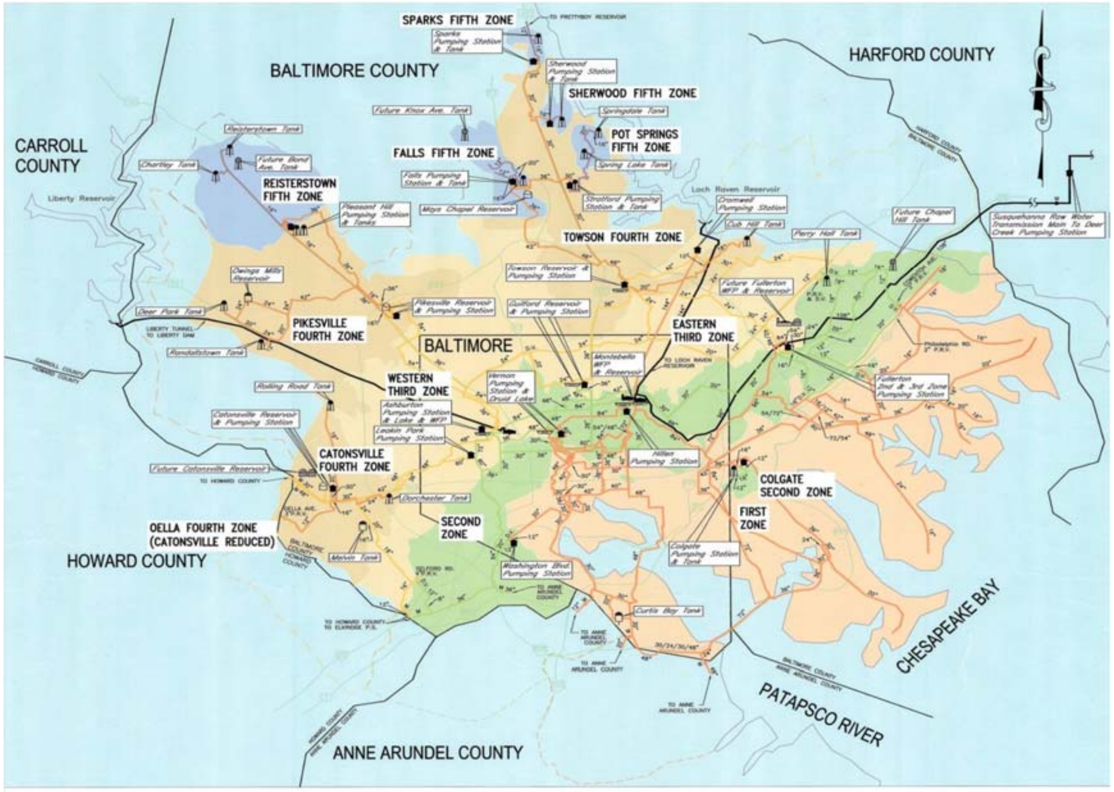


Figure 5-1. City of Baltimore Distribution System

The City of Baltimore operates 20 finished water pumping stations and one raw water pumping station (Deer Creek), operates 27 finished drinking water towers, operates 2 major chlorinators and 16 remote chlorinators, maintains 3,800 miles of water mains and an additional 700 miles of public water connections, and maintains 9,100 fire hydrants in the city and 13,750 fire hydrants in the county (Figure 5-2).

The Department of Public Works serves 5 other counties besides the City of Baltimore; Anne Arundel County, Baltimore County, Carroll County, Harford County, and Howard County.

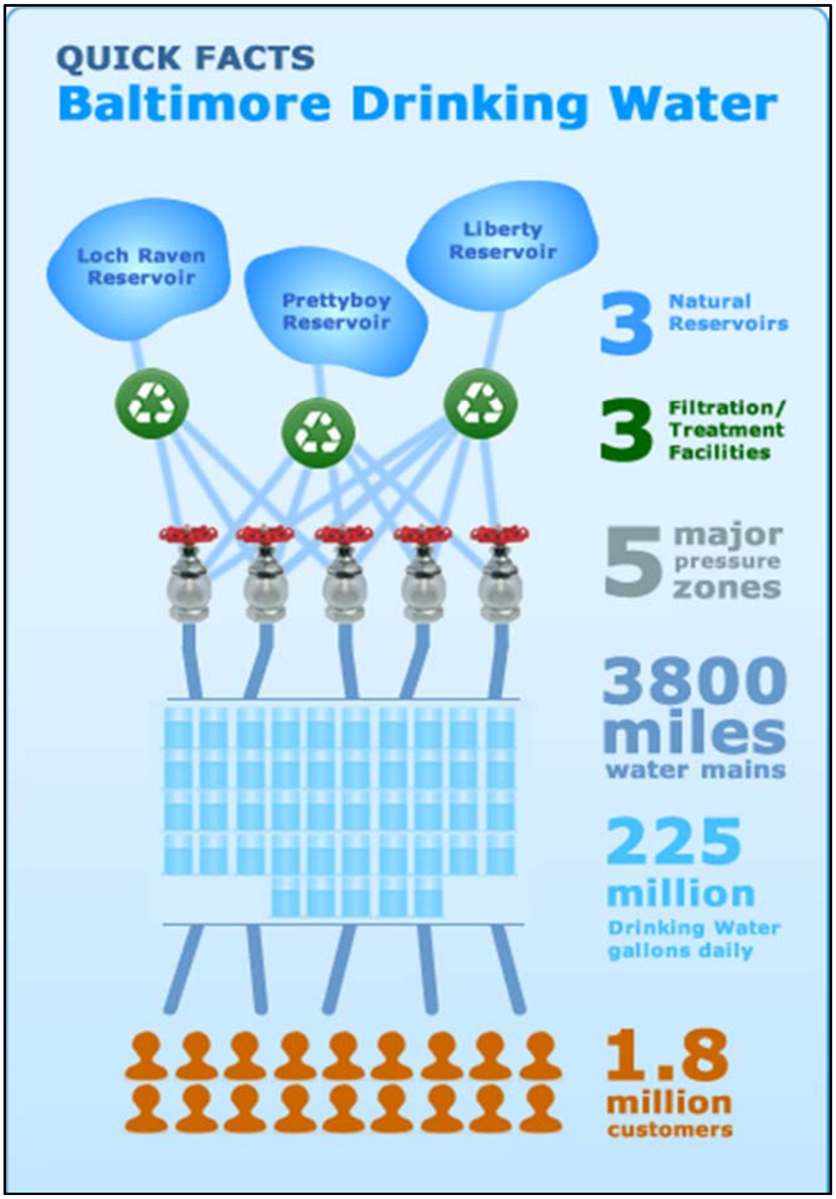


Figure 5-2. Baltimore City Water Facts

5.1.1. Distribution

The distribution system delivers treated water to Baltimore area consumers throughout the metropolitan area. The Central System service area is approximately 560 square miles and provides potable water through 3,800 miles of water-mains.

The system distributes water through a network of water mains ranging in size from three inches to twelve feet in diameter. Most of these mains are constructed of cast iron, but some of the larger mains are steel or reinforced concrete. Currently, more than 3,800 miles of mains are in service in the distribution system. These mains connect a series of pumping stations, reservoirs, and elevated storage tanks, which supply water to Baltimore City and parts of Baltimore County, Howard, and Anne Arundel Counties. Within the network of mains, five major pressure zones are maintained to provide adequate water pressure and supply to the consumers.

Under the present operating system, the Montebello Filtration plants (Figure 5-3) supply water to the First Zone by gravity, and the Second and Third Zones by pumping. The Ashburton Filtration Plant supplies water to the Second Zone by gravity, and to the Third, Fourth, and Fifth Zones by pumping.

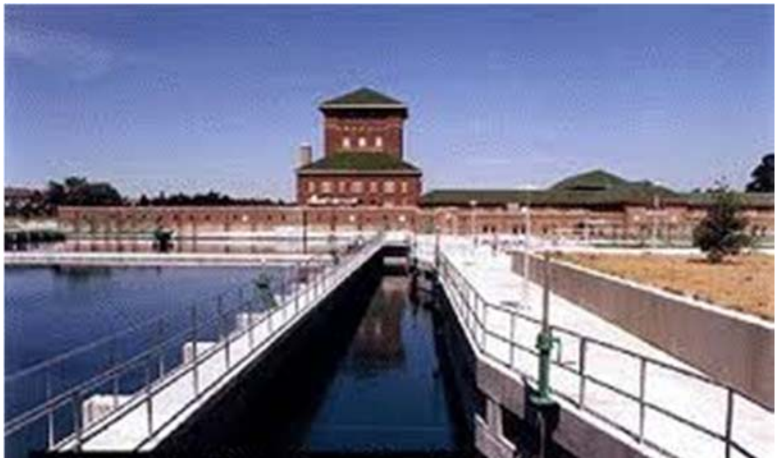


Figure 5-3. Montebello Filtration Plant (City of Baltimore)

5.1.2. Water Filtration Plants

The City operates three water filtration plants to meet the current and future demands of the metropolitan area’s 1.8 million consumers. Montebello Plants I and II are normally supplied by the Gunpowder Falls Reservoirs. Water from Loch Raven flows by gravity to the Montebello plants through a 12’ tunnel. The capacity of Plant I is 128 million gallons per day (MGD), while Plant II is rated at 112 MGD.

In times of drought, the Deer Creek Pumping Station supplements Loch Raven by pumping water from the Susquehanna 37 miles through a 9’ transmission main to Montebello. The third filtration plant, Ashburton, located on the west side of the City, is supplied by Liberty Reservoir through a 10’ wide tunnel 13 miles long. This plant can treat up to 165 MGD.

The city’s water supply system must not only meet everyday water demands but also the maximum projected needs of consumers. The combined safe treatment capacity of the three plants is over 400 MGD.

5.1.3. Baltimore City Water Loss

The Water Loss data for the period 2007 to 2011 was collected and analyzed for the percentage of the total water loss on the network and for the percentage of water loss from treatment plant till the pump stations which is a subset of the total loss as shown in Figure 5-4. It is clear from the figure that a higher water loss percentage is in the distribution pipes after the pumping stations. This higher water loss percentage is due to the higher pipe connections after the pumping stations.

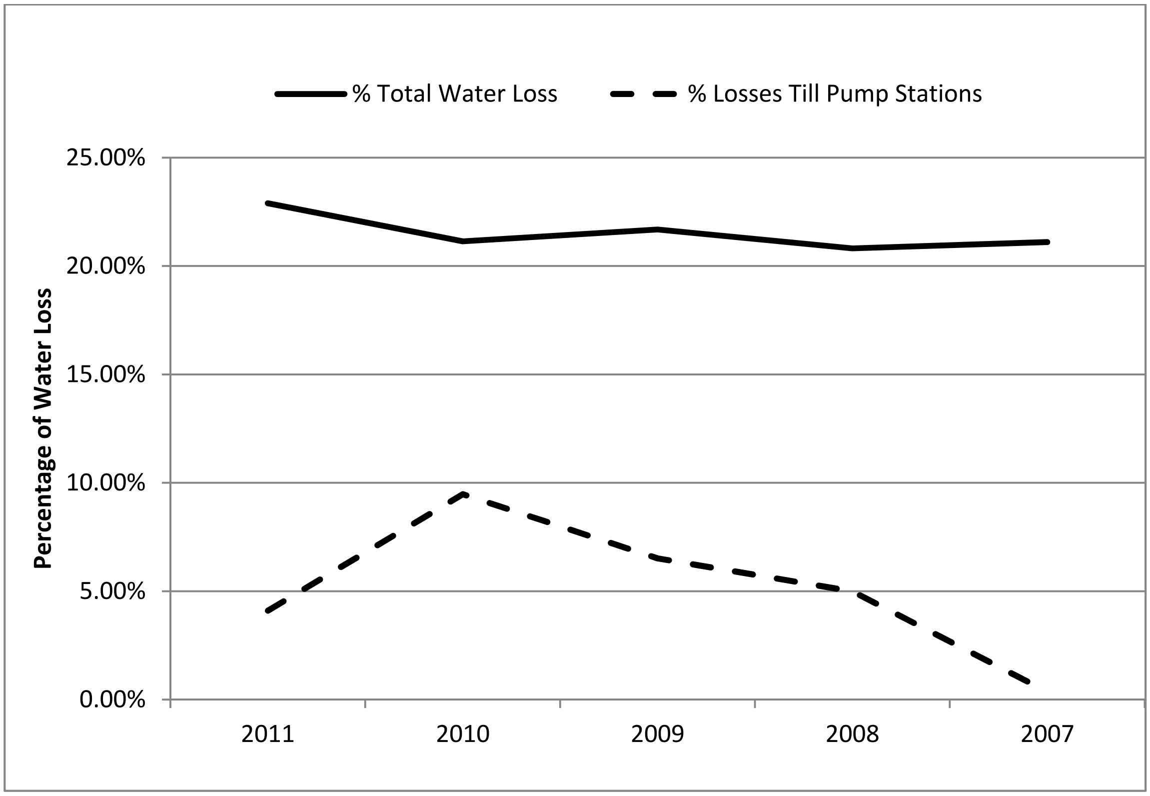


Figure 5-4. Water Loss Percentage in City of Baltimore

The number of breaks per Fiscal Year was also collected between periods FY 08 through FY 12 as shown in Figure 5-5. The graph shows the correlation between the average temperature and the number of breaks where the number of breaks reaches its peak in the months of January and February where the temperature reaches its lowest. In addition, during FY 09 and FY 10, when the temperature during the winter season was higher than the average and more stable than other years, fewer pipe breaks were reported during this period.

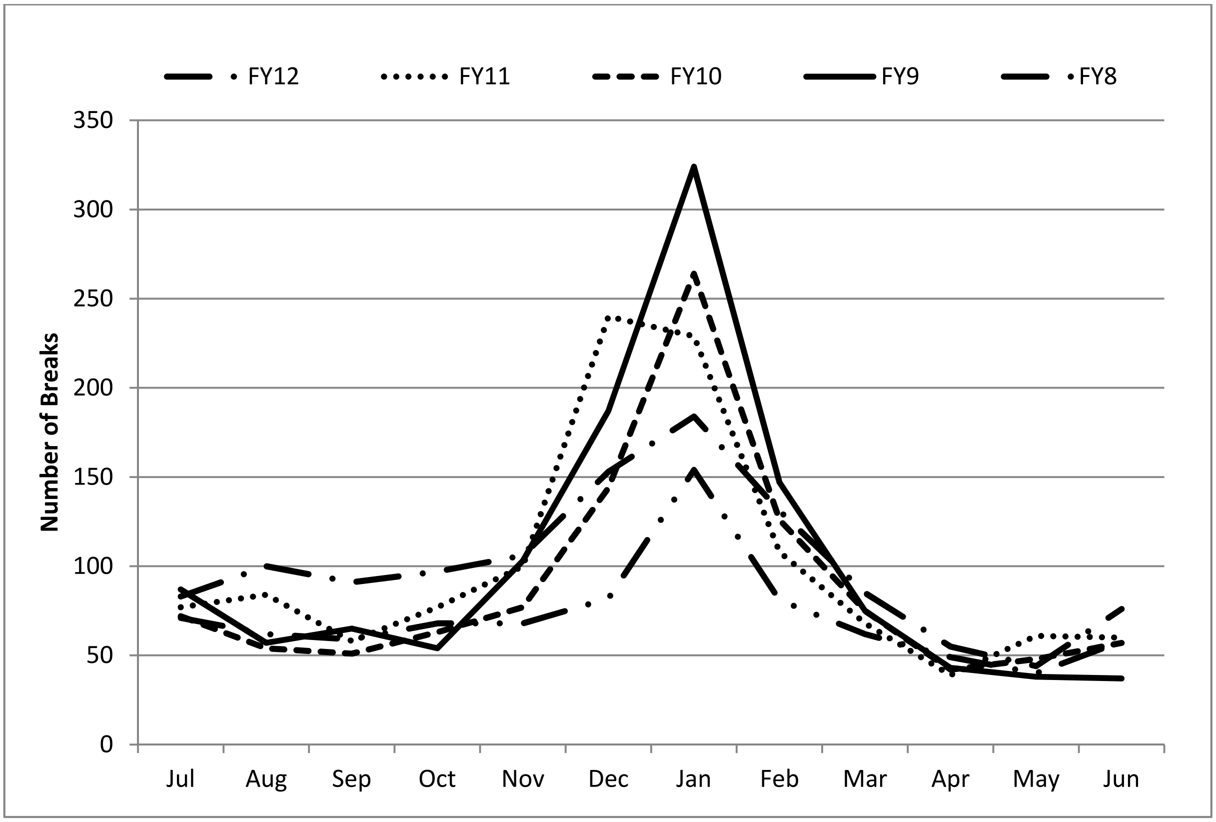


Figure 5-5. Number of Breaks by Fiscal Year in City of Baltimore

5.2. Washington Suburban Sanitary Commission (WSSC)

The Washington Suburban Sanitary Commission (WSSC) was established on May 1st, 1918. WSSC is the 8th largest water and wastewater utility in the nation, serving nearly 1.8 million residents and approximately 460,000 customer accounts in Prince George's and Montgomery counties over an area of nearly 1,000 square miles (Figure 5-7). WSSC operates and maintains eight water and wastewater plants, more than 5,500 miles of fresh water pipeline and nearly 5,400 miles of sewer pipeline.

Figure 5-6. WSSC Water

WSSC Operates and Maintains:

- 3 reservoirs – Triadelphia, Rocky Gorge, and Little Seneca with a total holding capacity of 14 billion gallons (Note: Jennings Randolph Reservoir holds an additional 13 billion gallons of water shared with Fairfax Water and the Washington Aqueduct)
- 2 water filtration plants – the Patuxent (max. 56 million gallons per day MGD) and the Potomac (max. 285 MGD) plants produce an average of 167 million gallons per day (MGD) of safe drinking water
- More than 5,500 miles of water main lines and nearly 5,400 miles of sewer main lines.

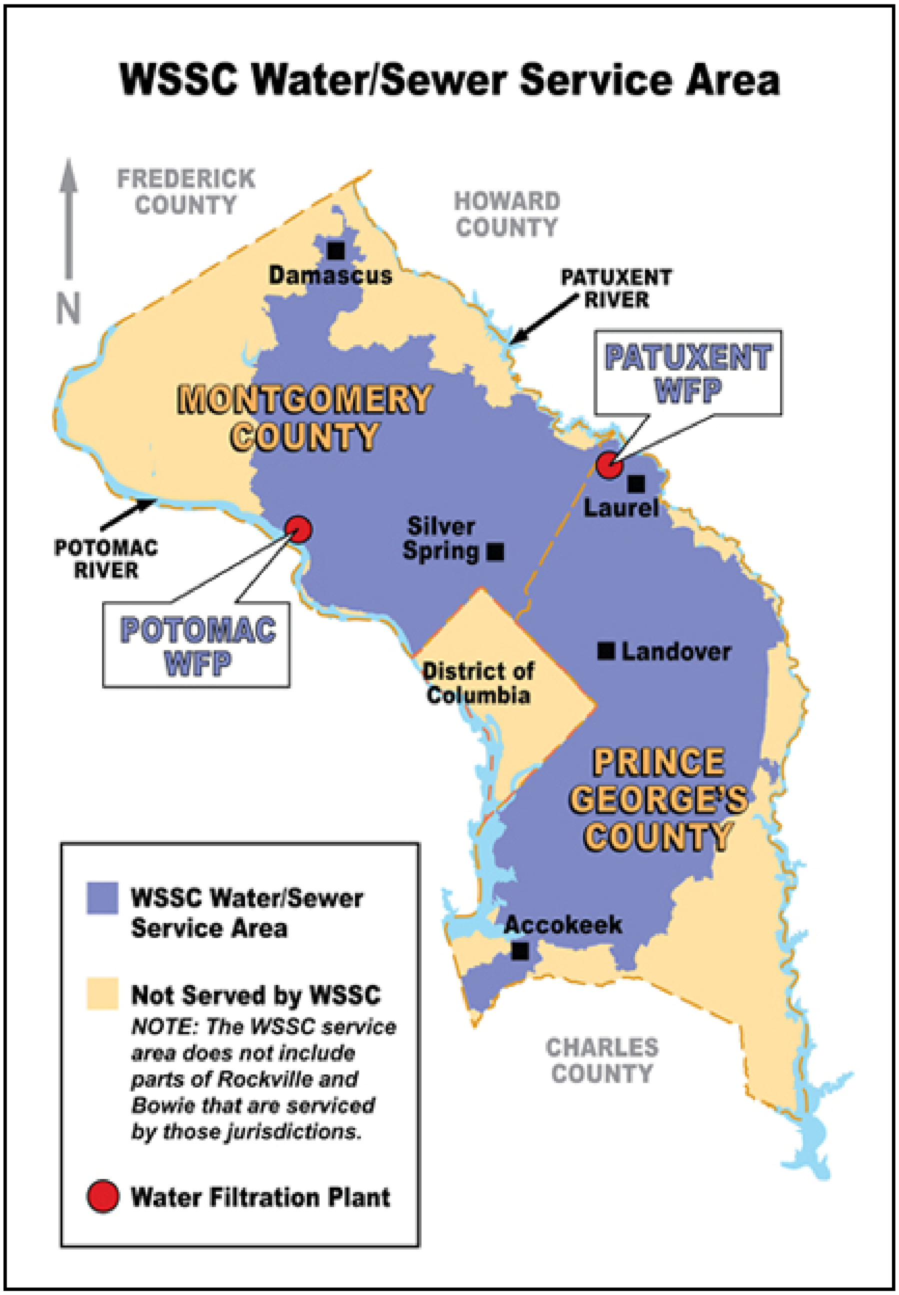


Figure 5-7. WSSC Service Area (WSSC 2010)

5.2.1. WSSC Aging Infrastructure

WSSC has been serving the residents of Montgomery and Prince George's counties since 1918. WSSC is now faced with the critical challenge of old and failing infrastructure. WSSC maintains approx. 5,573 miles of water mains.

Figure 5-8. Pipe Break (WSSC 2010)

As WSSC moves toward 100 years of service, they are faced with aging (deteriorating) pipes and valves. As of December 2009, nearly 26% (approx. 1,443 miles of water mains out of the nearly 5,573 miles they maintain) are more than 50 years old. Approximately 1,973 miles of mains (35%) are between 31 and 50 years old; 547 miles of pipe (10%) are 25-30 years old; the remaining 1,610 miles of pipe (29%) were installed in the last 25 years.

The older pipes (installed before 1931 and up to 1975) are either cast iron or asbestos cement, and have reached their natural life span (Figure 5-9). The aging process is driven by the corrosion of the metallic pipes by the soil and by water in cases of internally unlined pipes.

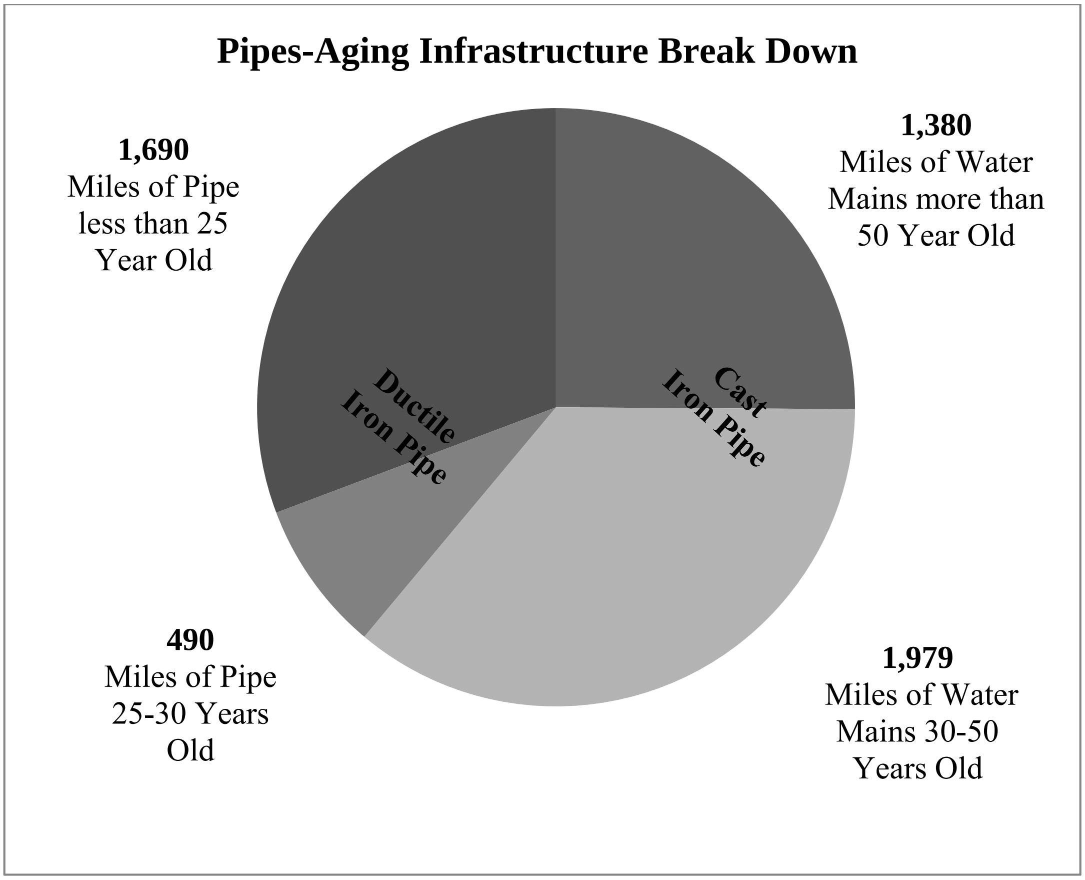


Figure 5-9. WSSC Aging Infrastructure Breakdown

5.2.2. Fire Hydrants

WSSC has 40,000-plus hydrants that are spread out over 5,300 miles of water pipe. Fire hydrants are made of cast iron materials and can last more than 50 years. However, the internal working parts, such as gaskets and seats, require routine maintenance or replacement (Figure 5-10).

Figure 5-10. Old and New Fire Hydrants

5.3. Water Audits

COB and WSSC both performed water audits in 2010. The Auditors performed a water audit in conformance with current AWWA Standards as described in the AWWA M 36 Manual, Water Audits, and Loss Control Programs. The audit is a hybrid top- down and bottom-up audit approach, indicating that available data and records supplied by the utility company were analyzed (top-down approach), but supplemented with the findings of detailed field investigations performed by the Auditors (bottom-up approach). The results of the water audits were collected and tabulated and used for the validation of the model.

5.3.1. City of Baltimore Water Audit

KCI Technologies (KCI) completed a water audit for the Baltimore Metropolitan Water District (BMWD) for the time period of March 1, 2009 to February 28, 2010. The primary goal of the water audit was to minimize operating costs and optimize revenue by assessing water accountability in the distribution system.

The auditors performed a water audit in conformance with the current AWWA Standards as described in the AWWA M 36 Manual, Water Audits and Loss Control Programs. The audit is a hybrid top-down and bottom-up audit approach, indicating that the available data and records supplied by the BMWD were analyzed (top-down approach). The audit was supplemented with the findings of detailed field investigations performed by the Auditors (bottom-up approach) as shown in Figure D-1 in Appendix D.

5.3.2. Washington Suburban Sanitary Commission Water Audit

The Washington Suburban Sanitary Commission (WSSC) conducted the most recent water system audit for FY10 (Figure D-2) using the help of JMT and determined that the unaccounted for water loss is 17.2% as shown in Table 5-1. In response to the findings of the water system audit, WSSC has prepared a Water Loss Reduction Plan to meet the requirements of MDE shown in Appendix D.

Table 5-1. Results of 2010 WSSC Water Audit

Water Audit Result	Quantity	Unit
Volume of Water From Own Sources (Raw Data)	61,590	MG/Yr
Adjustments to Water From Own Sources	-32	MG/Yr
Adjusted Volume of Water From Own Sources	61,558	MG/Yr
Water Exported	1,485	MG/Yr
Water Supplied	60,072	MG/Yr
Billed Metered Consumption	48,138	MG/Yr
Billed Unmetered Consumption	0	MG/Yr
Unbilled Metered Consumption	604	MG/Yr
Unbilled Unmetered Consumption	751	MG/Yr
Apparent Water Losses	3,293	MG/Yr
Real Water Losses	7,286	MG/Yr
Net Lost or Unmeasured Water	10,579	MG/Yr
Percentage of Lost or Unmeasured Water (Net Lost or Unmeasured Water/Water Produced)	17.2	%

5.4. Model Validation

The two Case Studies City of Baltimore (COB) and Washington Suburban Sanitary Commission (WSSC) were both used to validate the modified formulation for the Unavoidable Annual Real Loss (UARL) using the data for the leakage component parameters and the detailed conducted system water audit collected in Table 5-2.

Table 5-2. Case Studies Data Collected

	Unit	COB	WSSC
Water Supplied	MG/Yr	79,115.00	60,072.00
Authorized Consumption	MG/Yr	63,442.59	49,492.82
Total Water Loss	MG/Yr	15,672.40	10,579.00
Total Water Loss %	%	19.81%	17.61%
Apparent Loss	MG/Yr	12,754.30	3,293.00
Apparent Loss % (Of Total Loss)	%	81.38%	31.13%
Real Loss CARL	MG/Yr	2,918.10	7,286.00
Real Loss CARL % (of Total Loss)	%	18.62%	68.87%
Length of Mains	Miles	4,065.90	5,339.00
Number of Service Connections	Number	436,786	464,232.00
Connection Density	Number/Miles	107.43	86.95
Length of Customer service Line	ft	15.3	72.00
Lp	Miles	1,265.69	6,330.44
Average Pressure	psi	81.6	75.20
UARL	Gallons/Day	2,888.20	4,007.33
ILI(Reported)	Unitless	1.01	1.82
Avg. Soil Resistivity	Ohm-cm	25,000.00	30,000.00

Using the average soil resistivity values for COB and WSSC from Table 5-2 and substituting in Equation 4-5 that was determined from the regression analysis, the Corrosivity Factor (Cr) values were 0.58 and 0.57, respectively. By incorporating the Corrosivity Factor (Cr) and applying in the modified formulation for the UARL (Equation 5-1) the value for UARL for COB and WSSC were 1,768.45 Gallons/day and 2,763.42 Gallons/day respectively.

$$UARL = ((5.41Lm + 0.15Nc)\boldsymbol{Cr} + 7.5Lp) \times P \tag{5-1}$$

The modified ILI values for COB and WSSC, according to the modified values of UARL, were calculated as 1.65 and 2.64. By analyzing the new results of ILI for the case studies, the following was found:

1. City of Baltimore (COB):

The resulting ILI (1.65) is higher than the value in the Audit (1.01). However, when comparing both the Connection Density (107.4 conn/mile) and the CARL/Main Length (0.72 Gallons/day/mile), the ILI value is still untypical for large, older cities with aging infrastructure such as Baltimore and should have been with an even higher value.

Furthermore, the lack of confidence in the data that are available to calculate the volume of water supplied and the calculation for billed metered consumption are two major contributors to the low ILI. The first significant source of error in the water audit is the value calculated for volume from own source due to the inaccuracies in the water production figures. Not having an accurate figure for the volume of water supplied adversely affects the rest of the figures calculated by the software (KCI, 2011).

Another significant source of error is the value reported for billed metered consumption. The auditors received a data dump of the customer billing system as well as several automated billing reports (KCI, 2011). The Access database (data dump) was queried and compared with the automated billing reports. There were numerous discrepancies that can best be explained by the significant number of adjustments that are continuously occurring to customer accounts. The billed meter consumption varies depending on the day the report is generated and what adjustments have occurred. In addition, the auditors identified numerous apparent losses due to inaccuracies in customer billing system, which may also affect the accuracy of the billed meter consumption.

2. Washington Suburban Sanitary Commission (WSSC):

The resulting ILI (2.64) is higher than the value in the Audit (1.82) and, when comparing both the Connection Density (86.9 conn/mile) and the CARL/Main Length (1.36 Gallons/day/mile), the ILI is more reliable than the previous value in the audit. This number should also be slightly higher for a large, older city like Washington Metropolitan with its aging infrastructure.

WSSC should update the current water audit procedures related to water usage from WSSC facilities, fire departments, exempt accounts and charitable organizations so that consumption from these sources is classified as billed metered rather than unbilled metered to be in accordance with the AWWA Manual M36 guidelines (JMT, 2011).

WSSC should analyze a cross-section of individual accounts and track the water usage data for these accounts through the data handling flowcharts to identify potential losses. They should also analyze the billing system operations using the 9 billing system questions provided in the AWWA Manual M36. Apparent losses should be identified through the review of the customer billing process and provide these data to the water auditors for their use in determining the losses due to systematic data handling errors.

Therefore, for both COB and WSSC, the ILI values were refined to be more realistic, which shows the importance of accounting for the soil corrosivity when estimating the UARL. The two utility companies should still take some actions to update their current water audit practices in accordance with the AWWA Manual M36 guidelines and accordingly refine the values for the UARL and the ILI and to continue to perform the water audit annually.

5.5. Simulation and Results

Using the provided statistical distributions, n-sets of system variables were generated. Each set represents a possible system (Monte-Carlo) scenario. Each set includes the values for the length of the transmission main, number of service connections, total length of private pipe, soil corrosivity, and the average network pressure.

However, because of the lack of any available data to compute co-variances, the variables in the proposed methodology are each assumed to be independent. The different variables to measure the UARL have a Gaussian distribution. The different mean value and standard deviation values for the different variables are shown in Table 5-3.

Table 5-3. Mean and Standard Deviation for Different Variables

Variables	Mean	Standard Deviation
Main Length (Lm)	1,272.86	1,290.11
Number of Connections (Nc)	72,346.27	78,327.66
Corrosivity Factor (Cr)	0.52	0.21
Length of Service Connections (Lp)	73.97	150.89
Pressure (P)	77.77	20.39

This part of the model solves the distribution network in order to obtain the Unavoidable Annual Real Loss. The model will operate n-times using each of the n-sets of input variables representing the different Monte Carlo scenarios. The source code for the proposed methodology was written in MATLAB.

A Monte Carlo simulation was operated 1,000, then 10,000, and then 100,000 times. Increasing the number of Monte Carlo samples beyond 100,000 times did not seem to offer any significant additional accuracy. The first simulation was made without adding the corrosivity factor (Cr). The UARL had an output Generalized Extreme Value distribution with a mean value of 346.168 and a standard deviation of 396.651 and k equals -0.1403 as shown in Figure 5-11.

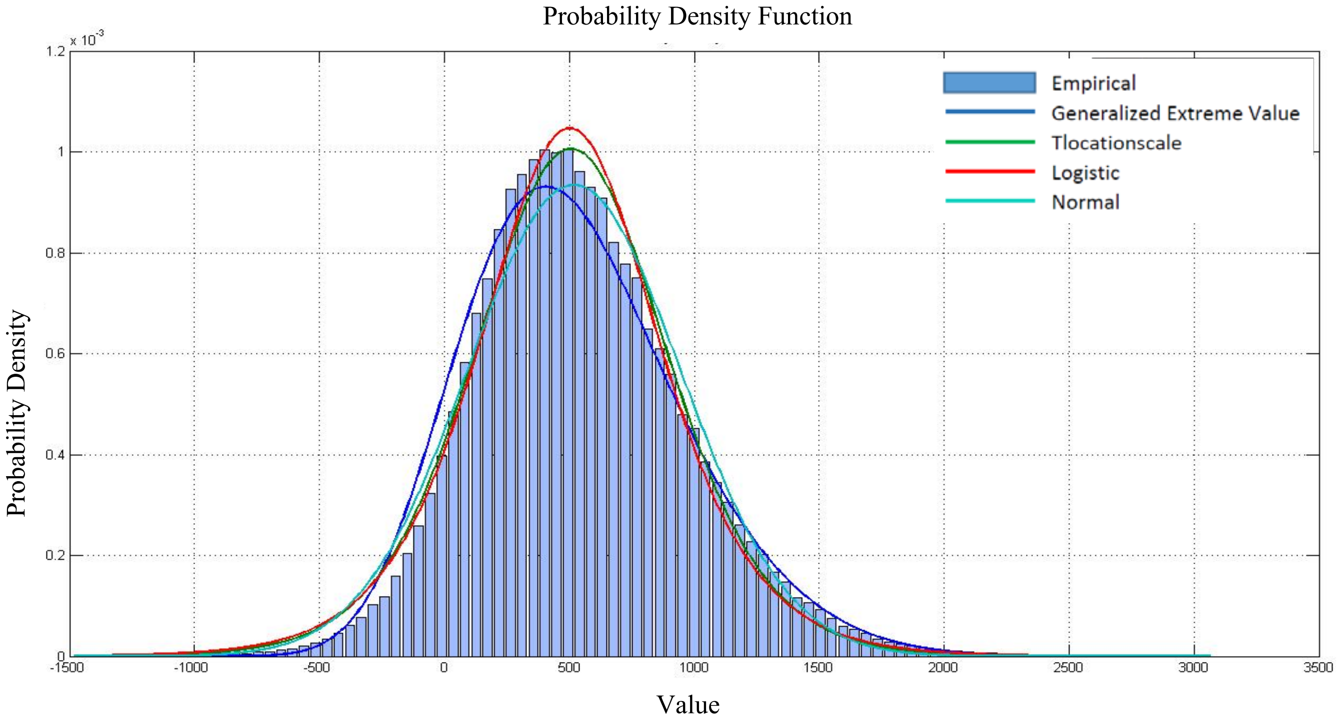


Figure 5-11. UARL Distribution without Cr

The second simulation was made after adding the corrosivity factor (Cr) to the UARL equation. Again the Monte Carlo simulation was operated 1,000, then 10,000, and then 100,000 times until the distribution had a minimal change. The UARL had an output Generalized Extreme Value distribution with a mean value of 169.696 and standard deviation of 227.445 and k equals -0.0868 as in shown in Figure 5-12.

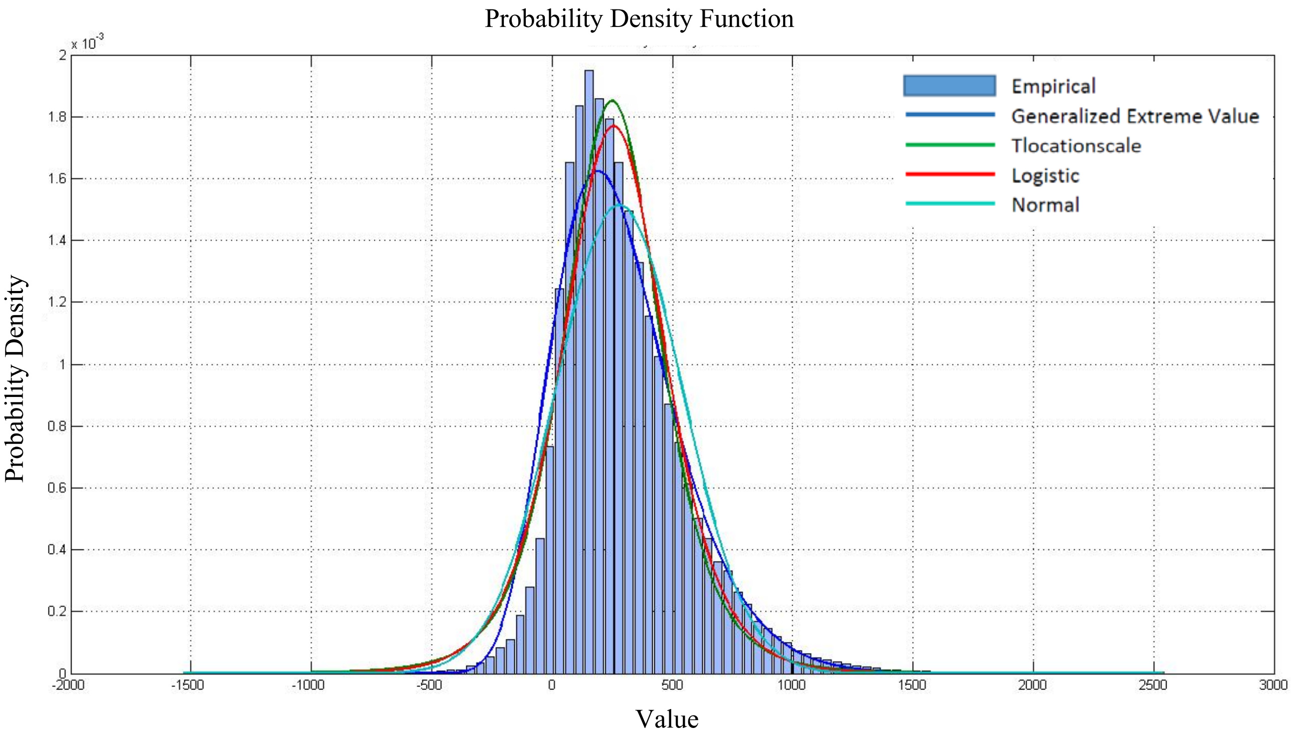


Figure 5-12. UARL Distribution with Cr

The output generalized extreme value distributions has k smaller than zero which corresponds to the Type III (Weibull distribution) whose tails are finite. It is evident that after adding the Cr in UARL estimation model the standard deviation value decreased from 396.651 to 227.445 which is approximately a 43% decrease, which has a significant effect on enhancing the estimating of the UARL. This shows that the corrosion behavior of water networks in soil is closely related to environmental factors, and it is possible to extract the key variables related to corrosion, such as soil corrosivity.

Chapter 6. Conclusion and Recommendations

6.1. Conclusion

Water Distribution Networks play a vitally important role in preserving and providing a desirable life quality to the public. In common engineering practice, water distribution systems are designed using only deterministic criteria: Determining the optimal configuration and network parameters that can meet the required flow and pressure rate are the result of hydraulic and cost-benefit analyses. The probability of system failure and other reliability statistics are very rarely included in such analyses.

A WDN should provide, during its economic life, the required quality and quantity of water at the required pressures. The system must be able to supply water during unusual conditions such as pipe breaks, mechanical failure of pumps and valves, power outages, malfunction of storage facilities, and uncertain demand projections.

The leakage rate and its high associated cost of failure have reached a level that now draws the attention of both policy and decision makers. As a result, dealing with the risk of water leakage has been undergoing a great change in concept from reacting to failure events to taking preventive actions that maintain water networks in good working conditions.

Leakage occurs in different components of the water distribution system: transmission pipes, distribution pipes, service connection pipes, joints, valves, and fire hydrants. Causes of leaks include corrosion, soil corrosivity, excessive water pressure, material defects, water hammer, excessive loads and vibration from road traffic, and stray electric current.

There have been many works reported in the literature that deal with uncertainties in the leakage of WDNs forecasting, estimating, and implementing. However, very little work has been performed in developing a model that estimates the recoverable leakage of water distribution networks when considering the different factors that may be important for predicting a more accurate estimate for the recoverable leakage.

In this dissertation, a probabilistic estimation model for the recoverable leakage of water distribution networks was presented, factoring in the key causes that lead to high percentages of leakage in different components of the water distribution network. Determining these key causes will help water utilities perform a predictive or preventive action plan rather than reacting to the failure and losses occurring due to the leakage of the water distribution network.

The model receives the deterministic and stochastic description of the leakage of the different distribution networks received from the water utility companies included in the research survey. The soil, water, and network characteristics for each of these water distribution networks were collected, analyzed, and tallied.

It is evident from the IWA's component analysis model and the literature review that IWA's model for estimating Unavoidable Annual Real Losses (UARL) does not account for soil corrosivity. The UARL equation can be modified by adding a new soil corrosivity factor (Cr) that takes soil corrosivity into consideration with the main length and the number of connections as shown in Equation (6-1).

$$UARL = ((5.41Lm + 0.15Nc)\boldsymbol{Cr} + 7.5Lp) \times P \tag{6-1}$$

Linear Regression was used to develop a relationship between the UARL and the soil corrosivity. The variables tested in this analysis are the Network Connection Density, UARL, Average Pressure, Water Main Length, and Length of Service Connections from the research survey. The criterion variable is the average soil resistivity (SR) and the predictor is the corrosivity factor (Cr). The least squares regression equation has an intercept of 0.67 and a slope of -0.31, as shown in Equation (6-2).

$$\text{Cr} = 0.67 - 0.31 \times \text{SR} \tag{6-2}$$

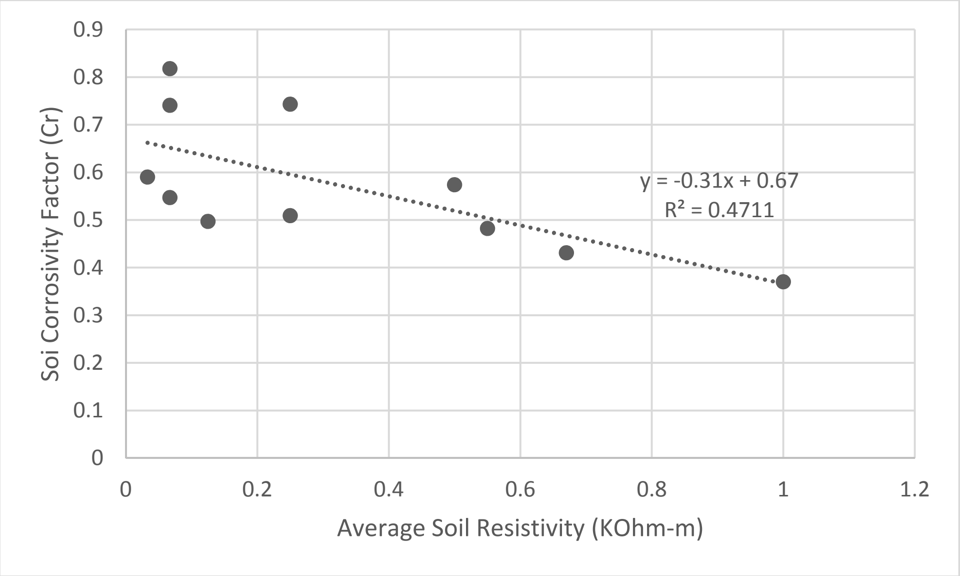


Figure 6-1. Average Soil Resistivity versus the Soil Corrosivity Factor

The p-value of the F(1,9) statistic in the analysis of variance was 0.02, which indicates that at the α=0.05 significance level, we could reject the null hypothesis (H_0) and conclude that the relationship between the two variables is significant. This indicates that the model applied can be used to predict the corrosivity factor from soil resistivity.

Hypothesis testing indicated the quality of the regression, the degrees of freedom of this statistic are df = n-2 = 11-2 = 9, and the standard error of the slope is 0.1081, the value of the test statistic was computed as -2.83 with a p-value=0.02. For a level of significance of α=5% and 9 degrees of freedom, we reject the null hypothesis since p-value < α. We therefore conclude that soil resistivity is a significant predictor of corossivity.

In addition to the hypothesis test for the slope, we can observe the 95% confidence interval. The 95% confidence interval for the slope is (-0.5507, -0.0615) which means that we are 95% confident that the true population slope is between these two numbers. Since this confidence interval does not include the value of zero, it is equivalent to a two-sided hypothesis test with 5% significance level that rejects the null hypothesis and concludes that the slope is significant.

Sensitivity analysis was used to test the robustness of the results of the proposed model. The need for the sensitivity analysis stems from acknowledging the presence of uncertainty in the model. Directional Cosines was used for the assurance of the level of sensitivity of the random variables of the UARL equation.

It is evident that after adding the corrosivity factor Cr to the UARL equation, the directional cosine value (0.44) for it was the second highest in importance (Figure 6-2) which shows the great impact of the soil corrosivity in measuring the UARL.

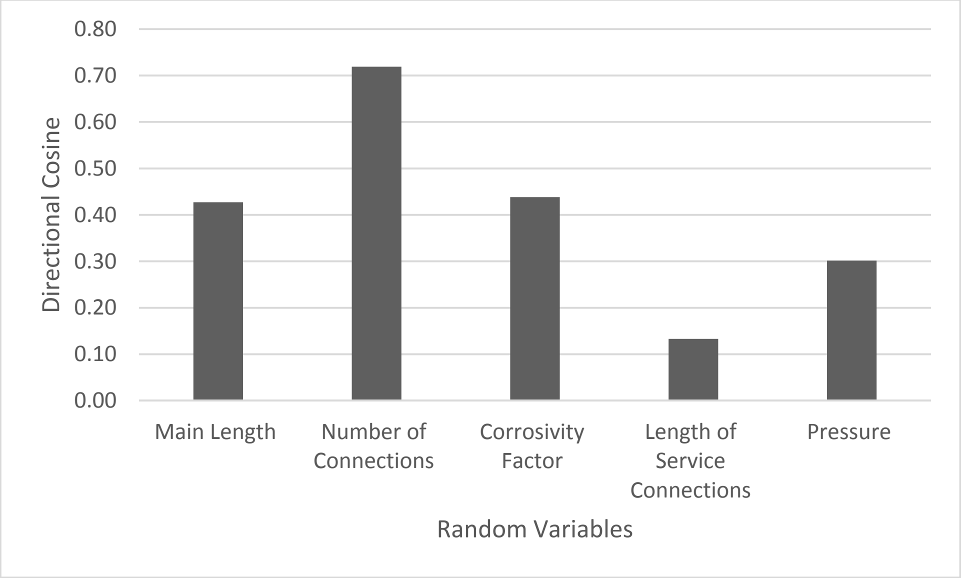


Figure 6-2. Directional Cosines for the Random Variables

The two case studies, City of Baltimore (COB) and Washington Suburban Sanitary Commission (WSSC), were used to validate the modified formulation for the Unavoidable Annual Real Loss (UARL) using the data for the leakage component parameters and the detailed conducted system water audit collected.

Using the average soil resistivity values for COB and WSSC and substituting in Equation 6-2, which was determined from the regression analysis, the Corrosivity Factor (Cr) was 0.58 and 0.57, respectively. By incorporating the Corrosivity Factor (Cr) and applying the modified formulation for the UARL (Equation 6-1), the values for UARL for COB and WSSC were 1,768.45 Gallons/day and 2,763.42 Gallons/day, respectively.

The modified ILI values for COB and WSSC according to the modified values of UARL were calculated as 1.65 and 2.64. By analyzing the new results of ILI for the case studies, the following was found:

1. *City of Baltimore (COB):*

   The resulting ILI (1.65) is higher than the value in the Audit (1.01) but compared to both the Connection Density (107.4conn/mile) and the CARL/Main Length (0.72Gallons/day/mile), the ILI value is untypical for large, older cities with aging infrastructure, such as Baltimore, and should have had an even higher value.

2. *Washington Suburban Sanitary Commission (WSSC):*

   The resulting ILI (2.64) is higher than the value in the Audit (1.82) and, compared to both the Connection Density (86.9 conn/mile) and the CARL/Main Length (1.36 Gallons/day/mile), the ILI is more reliable than the previous value in the Audit. This number should also be slightly higher for a large, older city like Washington Metropolitan with its aging infrastructure.

Therefore, for both COB and WSSC, although the ILI values were refined to be more realistic, the two utility companies should still take some actions to update their current water audit practices in accordance with the AWWA Manual M36 guidelines, accordingly refine the values for the UARL and the ILI, and continue to perform the water audit annually.

Using the provided statistical distributions, n-sets of system variables were generated. Each set represents a possible system (Monte-Carlo) scenario. Each set includes values for the length of the transmission main, number of service connections, total length of private pipe, soil corrosivity, and average network pressure. The source code for the proposed methodology was written in MATLAB.

A Monte Carlo simulation was operated 1,000, then 10,000, and then 100,000 times until the distribution had a minimal change. The first simulation was made without adding the corrosivity factor (Cr). The UARL had an output Generalized Extreme Value distribution mean value of 346.16 and a standard deviation of 396.65 and k equals -0.14 (Figure 6-2). The second simulation was made after adding the corrosivity factor (Cr). The UARL had an output Generalized Extreme Value distribution mean value of 169.69 and a standard deviation of 227.44 and k equals -0.08 (Figure 6-3).

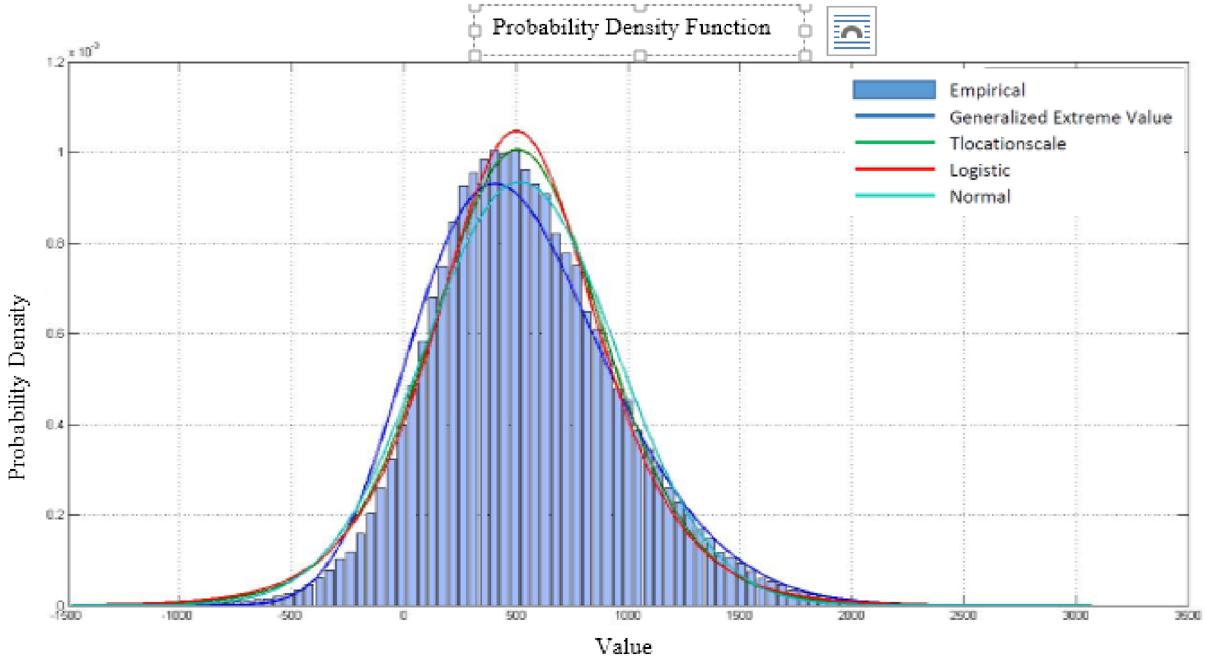


Figure 6-3. UARL Distribution without Cr

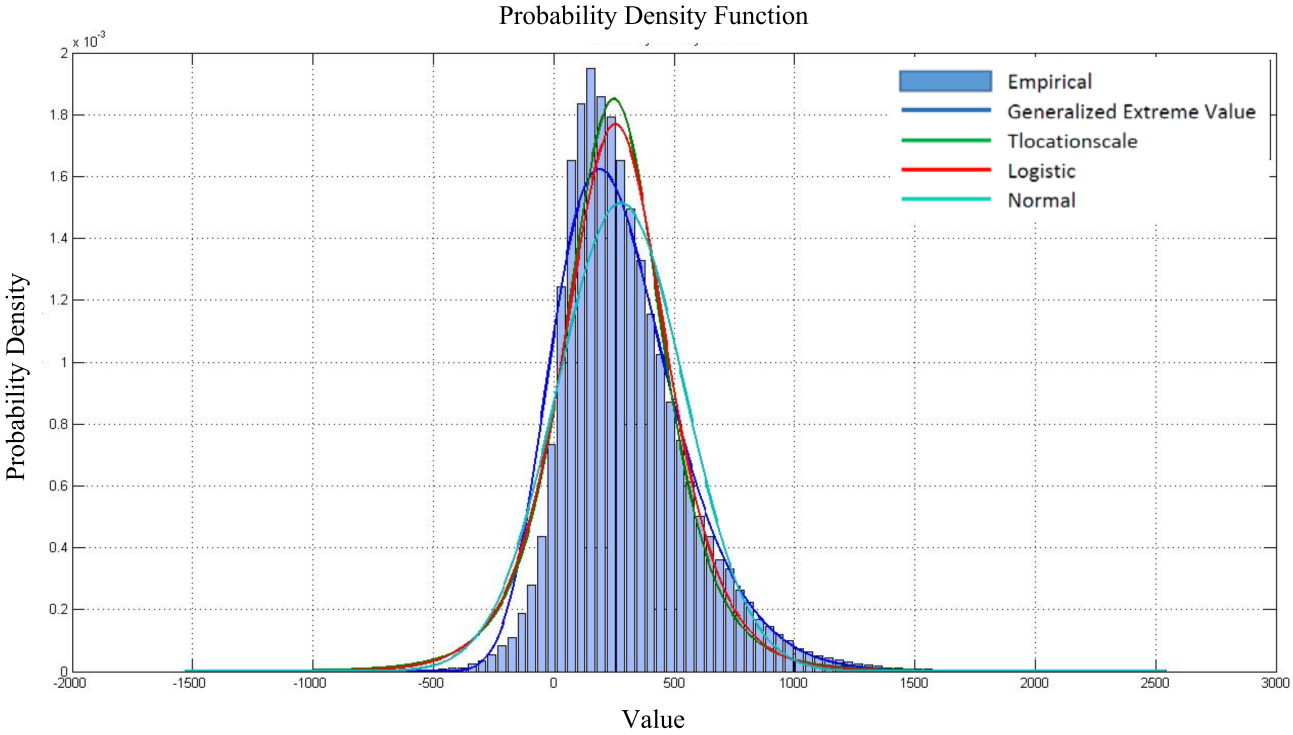


Figure 6-4. UARL Distribution with Cr

The output distributions for the UARL using and without using the corrosivity factor Cr and the 43% decrease in the standard deviation value using the corrosivity factor shows that the corrosion behavior of water networks in soil is closely related to the environmental factors, and it is possible to extract key variables related to corrosion, such as soil corrosivity.

It is evident that by modifying the UARL equation and adding a new factor (Cr) that takes the soil corrosivity into consideration with the main length and the number of connections as shown in equation 6-1, the values of ILI are more reliable and that helps utility companies make better decisions. If the soil corrosivity is known, it can provide useful information for the selection of pipeline paths, the methods of corrosion control in the stage of design, and the maintenance of underground metallic structures.

6.2. Recommendations

Despite growing pressure on water suppliers from drought, water shortages, and other challenges, the North American water industry has been slow to implement reliable and consistent water supply auditing and loss control. AWWA's Technical and Educational Council funded a survey that confirmed that US water loss reporting practices are limited and vary widely. In 2000, an International Water Association task force, with AWWA participation, assembled a water audit methodology as a best management practice (BMP) that is applicable to water suppliers worldwide, providing a framework to tabulate supplier water use and loss.

In addition, effective leakage management methods have been advanced with great success. The use of the international water audit method and water and revenue loss control technologies offer North American water utilities an outstanding water resource recovery opportunity and a great stride toward sustainability.

Recently, water loss has come under greater scrutiny nationwide as water rates continue to increase. Performing an annual water audit is the first step in identifying losses, physical and financial, that are occurring during the process of delivering the commodity of water to customers. Having a reliable water audit is the foundation of proper resource management for drinking water utilities as it shows the quantities of water flow in and out of the distribution system (AWWA Water Loss Control Committee, 2003).

To be successful in reducing water loss, a municipality must focus on reducing both real losses (e.g. leakage) and apparent losses (e.g. customer meter inaccuracy, unauthorized consumption, and systematic data handling error). Generally speaking, water utilities have had success in reducing real losses by implementing a combination of the following:

- Leak detection program
- Improved leak repair jobs
- Piloting district metered areas
- Pipeline replacement
- Pressure management.

Reductions in apparent losses can be achieved by implementing AMR (automatic meter reading), large-meter right sizing, billing error corrections, and thwarting unauthorized consumption through policy and enforcement.

Implementation of a Water Loss Reduction Program would benefit the water utility companies. The focus of the Program should be on the recoverable real and apparent losses (e.g. metering, billing, leakage) as determined by looking at the cost of the annual losses. Loss reduction actions that can be taken that will yield immediate results are obvious; however, a point of diminishing returns for loss recovery if the Water Utility spends too much money on particular loss pursuits.

More accurate data will yield a more accurate water balance and reliable performance indicators. It is recommended that all water audits be performed in conformance with current AWWA standards and use the AWWA Water Audit Software. This will provide additional transparency between audit periods and assist the Water Utility Companies to evaluate the effectiveness of corrective measures. A list of specific recommendations to improve the accuracy of water audits and reduce non-revenue water is shown in Appendix E.

Appendix A. Survey Questionnaire

Dear ,

My name is Moatassem Ghoniema, I am PhD Candidate in University of Maryland College Park under the supervision of Prof. Ayyub in the Project Management Program.

I am working on my research with a title of "Risk Management of Leakage of Water Distribution Networks", where I am trying to identify the effect of the different causes on the amount of leakage in the water distribution networks. So, we if we can put out hands on the main causes that lead to a high percentages of leakage in different components of the water distribution network, we can perform a preventive action plan rather than reacting to the failure and losses occurring due to the leakage of the water distribution network.

To run this model I need to quantify the Leakage rates in different locations in the US to determine the effect of the different causes, so kindly if you can provide me with the following data for your water utility company:

1) Annual water pumped from filtration plant
2) Annual billed metered Consumption
3) AWWA Water Audit Report (if available)

It will be my pleasure to share the results of my research with you.

Your cooperation with us to advance the knowledge of Leakage of Water Distribution Networks is highly appreciated.

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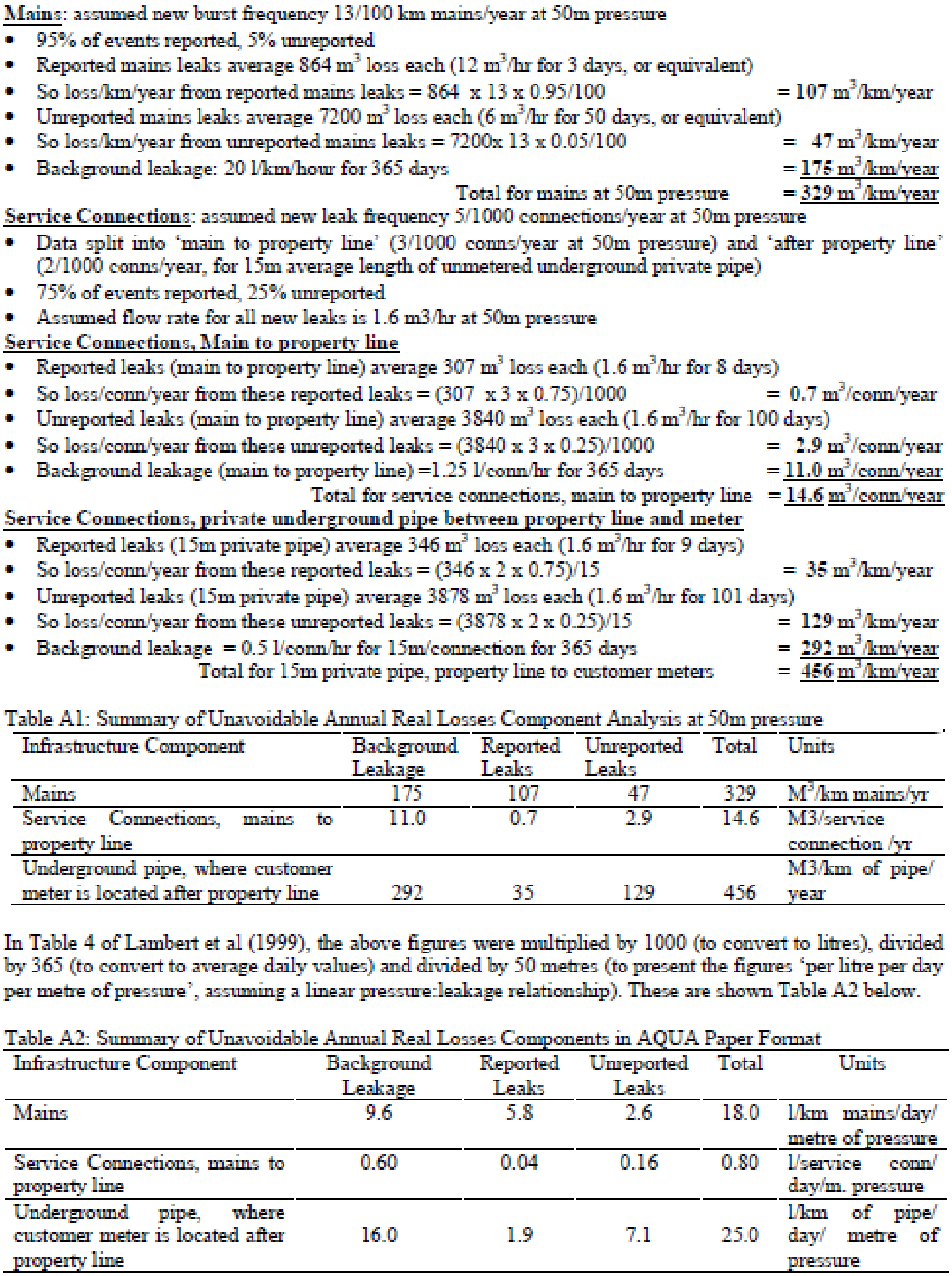
Appendix B. Survey Responses

Table B-1. Survey Responses

	States		Survey	Responses	Percentage
1	AL	Alabama	3	0	0.0%
2	AR	Arkansas	1	0	0.0%
3	AZ	Arizona	4	0	0.0%
4	CA	California	25	2	8.0%
5	CO	Colorado	7	2	28.6%
6	DC	District of Columbia	1	0	0.0%
7	DE	Delaware	1	0	0.0%
8	FL	Florida	9	1	11.1%
9	GA	Georgia	6	2	33.3%
10	HI	Hawaii	1	0	0.0%
11	IA	Iowa	3	1	33.3%
12	ID	Idaho	6	0	0.0%
13	IL	Illinois	3	1	33.3%
14	IN	Indiana	4	0	0.0%
15	KS	Kansas	2	1	50.0%
16	KY	Kentucky	4	0	0.0%
17	LA	Louisiana	2	0	0.0%
18	MA	Massachusetts	1	1	100.0%
19	MD	Maryland	5	4	80.0%
20	ME	Maine	3	0	0.0%
21	MI	Michigan	1	1	100.0%
22	MN	Minnesota	6	1	16.7%
23	MS	Mississippi	1	0	0.0%
24	MT	Montana	1	0	0.0%
25	NC	North Carolina	4	3	75.0%
26	NE	Nebraska	3	0	0.0%
27	NH	New Hampshire	3	0	0.0%
28	NJ	New Jersey	8	0	0.0%
29	NM	New Mexico	2	0	0.0%
30	NV	Nevada	7	0	0.0%
31	NY	New York	4	2	50.0%
32	OH	Ohio	9	3	33.3%
33	OK	Oklahoma	4	2	50.0%
34	OR	Oregon	6	1	16.7%
35	PA	Pennsylvania	9	2	22.2%
36	RI	Rhode Island	3	1	33.3%

37	SC	South Carolina	7	0	0.0%
38	SD	South Dakota	4	0	0.0%
39	TN	Tennessee	7	0	0.0%
40	TX	Texas	8	2	25.0%
41	UT	Utah	4	0	0.0%
42	WA	Washington	7	3	42.9%
43	WI	Wisconsin	7	1	14.3%
44	WV	West Virginia	1	0	0.0%
45	WY	Wyoming	5	0	0.0%
Total			212	37	17.5%

Appendix C. Component Analysis to Calculate UARL



Mains: assumed new burst frequency 13/100 km mains/year at 50m pressure

- 95% of events reported, 5% unreported
- Reported mains leaks average 864 m^3 loss each (12 m^3/hr for 3 days, or equivalent)
- So loss/km/year from reported mains leaks = 864 x 13 x 0.95/100 = 107 m^3/km/year
- Unreported mains leaks average 7200 m^3 loss each (6 m^3/hr for 50 days, or equivalent)
- So loss/km/year from unreported mains leaks = 7200x 13 x 0.05/100 = 47 m^3/km/year
- Background leakage: 20 l/km/hour for 365 days = 175 m^3/km/year

Total for mains at 50m pressure = 329 m^3/km/year

Service Connections: assumed new leak frequency 5/1000 connections/year at 50m pressure

- Data split into 'main to property line' (3/1000 conns/year at 50m pressure) and 'after property line' (2/1000 conns/year, for 15m average length of unmetered underground private pipe)
- 75% of events reported, 25% unreported
- Assumed flow rate for all new leaks is 1.6 m3/hr at 50m pressure

Service Connections, Main to property line

- Reported leaks (main to property line) average 307 m^3 loss each (1.6 m^3/hr for 8 days)
- So loss/conn/year from these reported leaks = (307 x 3 x 0.75)/1000 = 0.7 m^3/conn/year
- Unreported leaks (main to property line) average 3840 m^3 loss each (1.6 m^3/hr for 100 days)
- So loss/conn/year from these unreported leaks = (3840 x 3 x 0.25)/1000 = 2.9 m^3/conn/year
- Background leakage (main to property line) =1.25 l/conn/hr for 365 days = 11.0 m^3/conn/year

Total for service connections, main to property line = 14.6 m^3/conn/year

Service Connections, private underground pipe between property line and meter

- Reported leaks (15m private pipe) average 346 m^3 loss each (1.6 m^3/hr for 9 days)
- So loss/conn/year from these reported leaks = (346 x 2 x 0.75)/15 = 35 m^3/km/year
- Unreported leaks (15m private pipe) average 3878 m^3 loss each (1.6 m^3/hr for 101 days)
- So loss/conn/year from these unreported leaks = (3878 x 2 x 0.25)/15 = 129 m^3/km/year
- Background leakage = 0.5 l/conn/hr for 15m/connection for 365 days = 292 m^3/km/year

Total for 15m private pipe, property line to customer meters = 456 m^3/km/year

Table A1: Summary of Unavoidable Annual Real Losses Component Analysis at 50m pressure

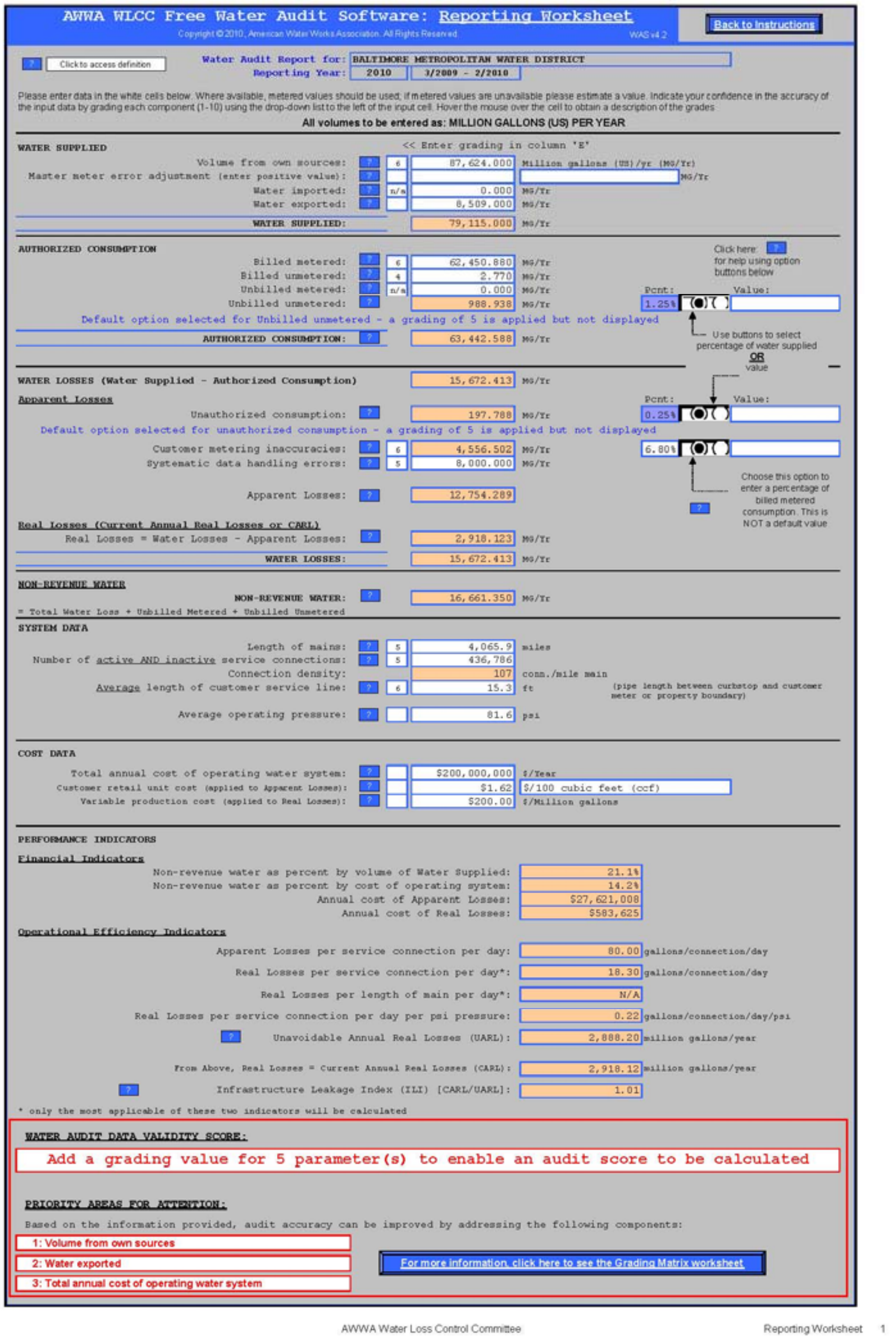
Infrastructure Component	Background Leakage	Reported Leaks	Unreported Leaks	Total	Units
Mains	175	107	47	329	M^3/km mains/yr
Service Connections, mains to property line	11.0	0.7	2.9	14.6	M3/service connection /yr
Underground pipe, where customer meter is located after property line	292	35	129	456	M3/km of pipe/ year

In Table 4 of Lambert et al (1999), the above figures were multiplied by 1000 (to convert to litres), divided by 365 (to convert to average daily values) and divided by 50 metres (to present the figures 'per litre per day per metre of pressure', assuming a linear pressure:leakage relationship). These are shown Table A2 below.

Table A2: Summary of Unavoidable Annual Real Losses Components in AQUA Paper Format

Infrastructure Component	Background Leakage	Reported Leaks	Unreported Leaks	Total	Units
Mains	9.6	5.8	2.6	18.0	l/km mains/day/ metre of pressure
Service Connections, mains to property line	0.60	0.04	0.16	0.80	l/service conn/ day/m. pressure
Underground pipe, where customer meter is located after property line	16.0	1.9	7.1	25.0	l/km of pipe/ day/ metre of pressure

Appendix D. AWWA Audits for Case studies



AWWA WLCC Free Water Audit Software: Reporting Worksheet

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WAS v4.2

Back to Instructions

? Click to access definition

Water Audit Report for: BALTIMORE METROPOLITAN WATER DISTRICT
Reporting Year: 2010 3/2009 - 2/2010

Please enter data in the white cells below. Where available, metered values should be used; if metered values are unavailable please estimate a value. Indicate your confidence in the accuracy of the input data by grading each component (1-10) using the drop-down list to the left of the input cell. Hover the mouse over the cell to obtain a description of the grades

All volumes to be entered as: MILLION GALLONS (US) PER YEAR

WATER SUPPLIED << Enter grading in column 'E'

Volume from own sources:	6	87,624.000	Million gallons (US)/yr (MG/Yr)
Master meter error adjustment (enter positive value):			MG/Yr
Water imported:	n/a	0.000	MG/Yr
Water exported:		8,509.000	MG/Yr
WATER SUPPLIED:		79,115.000	MG/Yr

AUTHORIZED CONSUMPTION

Click here: ? for help using option buttons below

				Pcnt:		Value:
Billed metered:	6	62,450.880	MG/Yr			
Billed unmetered:	4	2.770	MG/Yr			
Unbilled metered:	n/a	0.000	MG/Yr			
Unbilled unmetered:		988.938	MG/Yr	1.25%	● ○	

Default option selected for Unbilled unmetered - a grading of 5 is applied but not displayed

AUTHORIZED CONSUMPTION:		63,442.588	MG/Yr

Use buttons to select percentage of water supplied **OR** value

				Pcnt:		Value:
WATER LOSSES (Water Supplied - Authorized Consumption)		15,672.413	MG/Yr			
Apparent Losses						
Unauthorized consumption:		197.788	MG/Yr	0.25%	● ○	

Default option selected for unauthorized consumption - a grading of 5 is applied but not displayed

Customer metering inaccuracies:	6	4,556.502	MG/Yr	6.80%	● ○	
Systematic data handling errors:	5	8,000.000	MG/Yr			
Apparent Losses:		12,754.289				

Choose this option to enter a percentage of billed metered consumption. This is NOT a default value

Real Losses (Current Annual Real Losses or CARL)

Real Losses = Water Losses - Apparent Losses:		2,918.123	MG/Yr
WATER LOSSES:		15,672.413	MG/Yr

NON-REVENUE WATER

NON-REVENUE WATER:		16,661.350	MG/Yr

= Total Water Loss + Unbilled Metered + Unbilled Unmetered

SYSTEM DATA

Length of mains:	5	4,065.9	miles
Number of active AND inactive service connections:	5	436,786	
Connection density:		107	conn./mile main
Average length of customer service line:	6	15.3	ft (pipe length between curbstop and customer meter or property boundary)
Average operating pressure:		81.6	psi

COST DATA

Total annual cost of operating water system:		$200,000,000	$/Year
Customer retail unit cost (applied to Apparent Losses):		$1.62	$/100 cubic feet (ccf)
Variable production cost (applied to Real Losses):		$200.00	$/Million gallons

PERFORMANCE INDICATORS

Financial Indicators

Non-revenue water as percent by volume of Water Supplied:	21.1%	
Non-revenue water as percent by cost of operating system:	14.2%	
Annual cost of Apparent Losses:	$27,621,008	
Annual cost of Real Losses:	$583,625	

Operational Efficiency Indicators

Apparent Losses per service connection per day:	80.00	gallons/connection/day
Real Losses per service connection per day*:	18.30	gallons/connection/day
Real Losses per length of main per day*:	N/A	
Real Losses per service connection per day per psi pressure:	0.22	gallons/connection/day/psi
Unavoidable Annual Real Losses (UARL):	2,888.20	million gallons/year
From Above, Real Losses = Current Annual Real Losses (CARL):	2,918.12	million gallons/year
Infrastructure Leakage Index (ILI) [CARL/UARL]:	1.01	

* only the most applicable of these two indicators will be calculated

WATER AUDIT DATA VALIDITY SCORE:

Add a grading value for 5 parameter(s) to enable an audit score to be calculated

PRIORITY AREAS FOR ATTENTION:

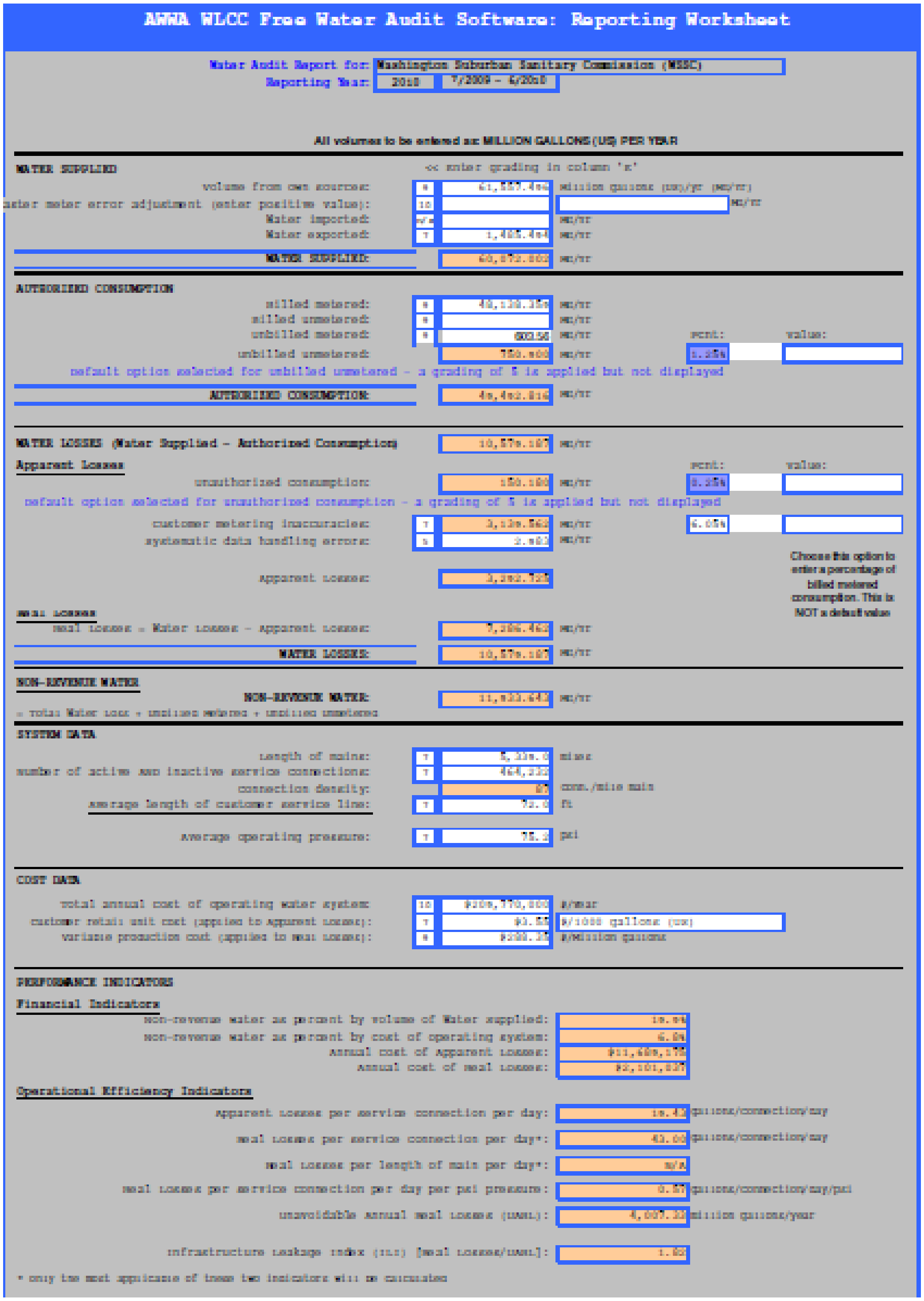
Based on the information provided, audit accuracy can be improved by addressing the following components:

1: Volume from own sources
2: Water exported
3: Total annual cost of operating water system

For more information, click here to see the Grading Matrix worksheet

AWWA Water Loss Control Committee Reporting Worksheet 1

Figure D-1. Baltimore Metropolitan Water District Water Audit



AWWA WLCC Free Water Audit Software: Reporting Worksheet

Water Audit Report for: Washington Suburban Sanitary Commission (WSSC)
Reporting Year: 2010 7/2009 - 6/2010

All volumes to be entered as: MILLION GALLONS (US) PER YEAR

WATER SUPPLIED <<enter grading in column 'E'
volume from own sources: 9 61,557.466 million gallons (US)/yr (MG/Yr)
Master meter error adjustment (enter positive value): 10 MG/Yr
Water imported: n/a MG/Yr
Water exported: 7 1,485.464 MG/Yr
WATER SUPPLIED: 60,072.002 MG/Yr

AUTHORIZED CONSUMPTION
billed metered: 8 48,138.356 MG/Yr
billed unmetered: 8 MG/Yr
unbilled metered: 8 603.56 MG/Yr Pcnt: Value:
unbilled unmetered: 750.900 MG/Yr 1.25%
default option selected for unbilled unmetered - a grading of 5 is applied but not displayed
AUTHORIZED CONSUMPTION: 49,492.816 MG/Yr

WATER LOSSES (Water Supplied - Authorized Consumption) 10,579.187 MG/Yr
Apparent Losses Pcnt: Value:
unauthorized consumption: 150.180 MG/Yr 0.25%
default option selected for unauthorized consumption - a grading of 5 is applied but not displayed
customer metering inaccuracies: 7 3,139.562 MG/Yr 6.05%
systematic data handling errors: 5 2.982 MG/Yr
Apparent Losses: 3,292.725

Choose this option to enter a percentage of billed metered consumption. This is NOT a default value

Real Losses
Real Losses = Water Losses - Apparent Losses: 7,286.462 MG/Yr
WATER LOSSES: 10,579.187 MG/Yr

NON-REVENUE WATER
NON-REVENUE WATER: 11,933.647 MG/Yr
= Total Water Loss + Unbilled Metered + Unbilled Unmetered

SYSTEM DATA
Length of mains: 7 5,338.0 miles
Number of active AND inactive service connections: 7 464,232
Connection density: 87 conn./mile main
Average length of customer service line: 7 72.0 ft
Average operating pressure: 7 75.2 psi

COST DATA
Total annual cost of operating water system: 10 $208,770,000 $/Year
Customer retail unit cost (applied to Apparent Losses): 7 $3.55 $/1000 gallons (US)
Variable production cost (applied to Real Losses): 8 $288.35 $/Million gallons

PERFORMANCE INDICATORS
Financial Indicators
Non-revenue water as percent by volume of Water Supplied: 19.9%
Non-revenue water as percent by cost of operating system: 6.8%
Annual cost of Apparent Losses: $11,689,175
Annual cost of Real Losses: $2,101,037

Operational Efficiency Indicators
Apparent Losses per service connection per day: 19.43 gallons/connection/day
Real Losses per service connection per day*: 43.00 gallons/connection/day
Real Losses per length of main per day*: N/A
Real Losses per service connection per day per psi pressure: 0.57 gallons/connection/day/psi
Unavoidable Annual Real Losses (UARL): 4,007.32 million gallons/year
Infrastructure Leakage Index (ILI) [Real Losses/UARL]: 1.82

* only the most applicable of these two indicators will be calculated

Figure D-2. WSSC Water Audit

Appendix E. Typical Soil Resistivity Map for the US

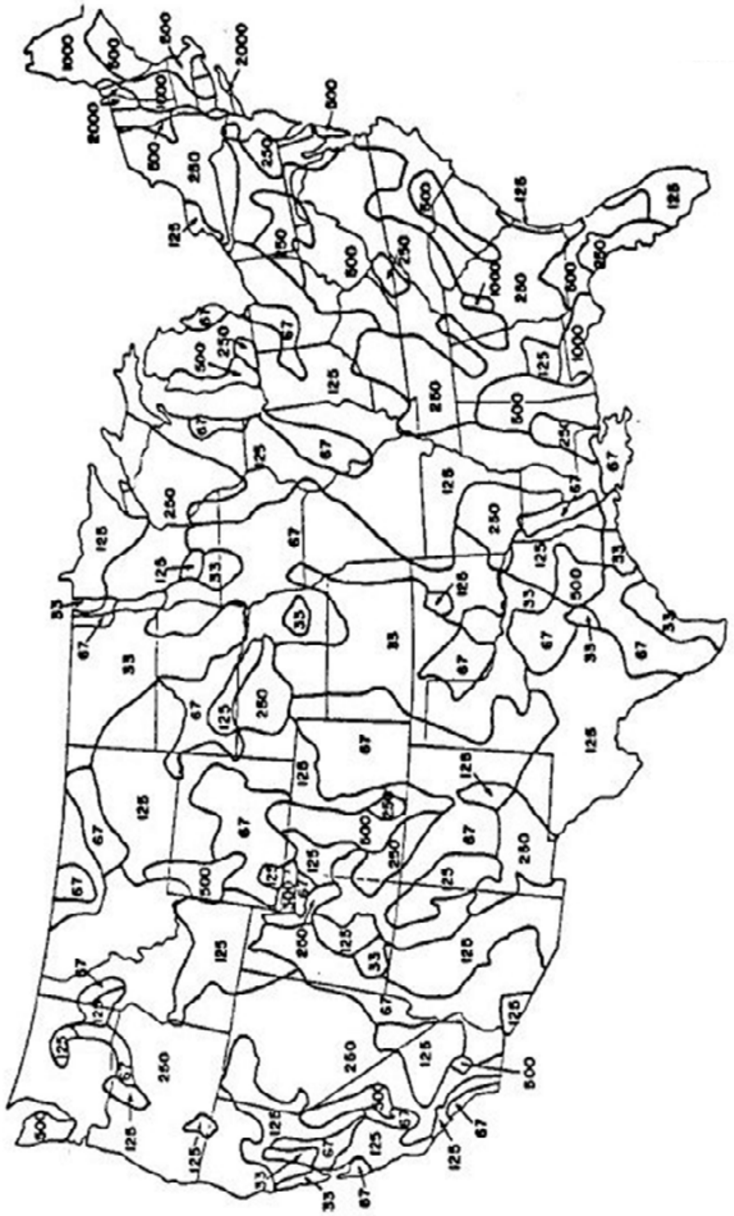


Figure E-1. Typical Soil Resistivity Map for the US in Ohm-meters

Appendix F. Water Loss Recommendations

The following is a list of specific recommendations to improve the accuracy of water audits and reduce non-revenue water (KCI 2011):

F.1. System Data Management and Reporting

1. Continue to update the water GIS.
2. Understand the programming used to generate reports from the billing system OR as the City moves forward with an AMI/AMR program, replace the existing billing system with one that will support the new metering system.
3. Consideration should be given to improving the reliability of the overall SCADA system. Data loggers installed on all production meters can be used to assess the reliability of the data reported. If the hand logs are to remain the basis of the Annual Water Production Report, data should be recorded more often than every 2 hours and account for changes in pumping at any given time period and should include the flow for each pump and an aggregate total for each pump station.
4. Data loggers may be installed on calibrated meters, which will provide an accurate local read. This method requires manually downloading data from the data loggers on regular intervals (approximately every 90 days). (Note: Data loggers have been installed on some of the master meters.

5. Revise the format of the Annual Water Production Report produced by the Water Analyzer Office to follow the water audit methodology suggested by AWWA (M36 Manual). The key outstanding elements include quantifying water losses (apparent and real) and unbilled unmetered consumption. In order to follow the approximate

format of the Annual Water Production Report, the City may choose to do a water audit for each service level. The Annual Water Audit published by the City of Philadelphia is a good model to use as a guide.

F.2. Metering

1. Correct the meter setting of large meters (meters ≥ 3”) to include meter test ports, and inlet and outlet valves. A bypass should also be provided for critical customers that cannot be without water for even a short duration5. Large meters need to have settings that will allow the meter to be tested while maintaining service to the customer.
2. Perform actual reads on all large meters. It was evident that regular meter reading is not occurring on many of the large meters that were visited by the auditors because the vaults were flooded and also required confined space entry (tripod and harness). Without regular meter readings on these accounts, the billing cannot be accurate.
3. Expand the large meter testing and replacement/overhaul program. This will help reduce the amount of non-revenue water (i.e. the amount of water supplied to the customer that the water utility is not getting revenue from). Large meter testing schedules should be based on consumption as well as potential revenue recovery for each meter type and size. Large meters located below grade would then be serviced more often, insuring revenues would remain more representative of what is being used. Regular large meter testing should be a part of the overall plan to reduce revenue loss and apparent water loss.

4. Budget approximately 2.5% of the annual gross revenue from customer metering for annual meter testing. Meter repairs will be covered by the recovered revenue from the meters that fail the tests.
5. Master meters should be tested in accordance with AWWA recommended time frames, typically annually or every six months. Calibration, repairs, and/or routine maintenance should be performed according to a regular schedule.
6. Replace jumper pipes with meters.
7. Implement a Large Meter Inspection Program independent of regular large meter reading to identify large-meter billing discrepancies.

F.3. Leak Detection and Repair

1. Conduct regular, systematic leak surveys and detection throughout all pressure zones.
2. Train and develop Standard Operating Procedures for Maintenance Division to improve responsiveness and effectiveness in repairing leaks.
3. Prepare On-Call Water Infrastructure Leak Repair contracts to assist City in expediting repairs during high periods of service requests.
4. Perform a pressure management study and/or consider a reduction of water pressure. Since leakage is a function of pressure, reducing pressure overall could reduce the amount of leakage.

F.4. Customer Billing

1. Implement the AMR/AMI initiative for automatic monthly meter reading. There would be a huge revenue increase if billing occurred on a monthly cycle. Larger meters located in vaults requiring confined space entry for reading could then be read

remotely without entry. Meters that malfunction can then be quickly identified and attended to.

2. Consider a review of rate codes for a possible revision of the coding system.
3. Continue to inspect large accounts to identify sources of apparent losses such as improper rate codes. Correct billing errors accordingly.
4. Formally inactive abandoned accounts that have been reactivated must be placed back in active meter reading routes. Currently these active accounts are not being billed.
5. Audit accounts that have consistently zero consumption to verify the meter is working properly.
6. Institute a system wide program to located buried meter vaults.
7. Reduce the number of estimated reads.

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