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Addis Ababa University (Since 1950)

ADDIS ABABA UNIVERSITY SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY (AAiT)

Water Supply Coverage and Water Loss in Distribution System with Modeling

(The Case Study of Addis Ababa)

A Thesis Submitted to the School of Graduate Studies of Addis Ababa University in partial Fulfillment of the Requirement for the Degree of Master Science in Civil Engineering (Water Supply and Environmental Engineering)

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November, 2011

Abstract

Leakage in water distribution pipes is a major problem faced by the water industry. Water utilities often employ traditional audit methods to estimate water lost as leakage. As a result demand for additional water sources and infrastructure is growing. More ever, nearly 37% of the total water production is loss at different level of distribution system before reaching to the consumer. The focus of this study is to evaluate the city's distribution coverage of the water supply and evaluating the total water loss. The water supply coverage at the city level and the total water loss both at the city level and at the sub system level, the collected data was assembled in EPANET and controls were added to best represent the functioning of the water system. Water production that is only for the city and the water consumption as aggregated from individual customer meter reading was to evaluate the total water loss at the city level. Select the pilot area in around Gottera the selection of the area from the branch based on the following criteria. Hydraulically easily desecrate area, 24hr water availability, Customer not more than 1000, and more leakage complain. The sub-system that has isolated networks and production and consumption data were used to evaluate and compare the spatial distribution of water loss. There are several reasons for the high level of water loss in Addis Ababa., and some advisory solutions were briefly proposed for the major effect of the water loss like age of pipe networks, poor maintenance of networks, water scheduling, customer side leakage and illegal connection. The reduction of NRW (Non Revenue Water) by Water Balance Method shows the difference between predicted and actual water losses in water distribution network, The results also shows that after leakage reduction control works took place, the volume of water loss in water distribution network has reduce about 39% of the total production supply to the sub-system.

The distribution system model was then used to evaluate three alternative scenarios to improve system performance. The objective of the first and second scenario was to increase the flow rate at taps of low supply; the third scenario aimed at adding taps to parts of the sub-system without easy access to running water. The first scenario consisted in opening valves to connect subsystems: it increased the flow rate at taps of large supply more so than at taps of low supply. This scenario was not recommended because it would quickly drain parts of the water supply.

Acknowledgments

I would like to thank my advisor, Dr. Yilma Sileshi for his excellent guidance, encouragement and support.

I would also like to express my gratitude to the staff of the Addis Ababa Water and Sewerage Authority (AAWSA) who helps me to get all the necessary data.

I am also grateful to my parents for the constant support and encouragement they have given me right from my childhood.

Finally I want to thank all my friends for their material and moral support.

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List of Abbreviations

AAWSA	*Addis Ababa Water and Sewerage Authority*
CSA	*Central Statistics Agency of Ethiopia*
DMA	*Distric Metering Area*
DCI	*Ducktail Cast Iron*
EPA	*Environmental Protection Authority*
HGL	*High Gradient Level*
IWA	*International Water Associations*
NRW	*Non Revenue Water*
NFPA	*National Fire Protection Association*
UFW	*Unaccounted for Water*
USEPA	*United State Environmental Protection Agency*
WHO	*World Health Organization*

1. Introduction

1.1 Background

According to the Global Water Supply and Sanitation assessment Report, 2000 the percentage of people served with some form of improved water rose from 79% (4.1billion) in 1990 to 82% (4.9billion) in 2000. At the beginning of 2000, one sixth (1.1 billion people) of the world population was without access to improved water supply. The majority of these people live in Asia and Africa, where two out of five African's lack improved water supply. The 2000 (G/C) coverage of water supply for the urban population of Africa and Ethiopia was 85% and 77% respectively. According to the millennium goal targets, the African urban areas will be accessed for improved water with 15 years from the year 2000. On the other hand, in Africa largest cities, only 43% inhabitants have house connection water supply services. The main problem that developing countries are faced to provide access to safe water for their citizen's is shortage of resource. Moreover, the capacity of the citizen's to pay for water that fully recovers the cost is very limited. For this reason many developing cities are faced great difficulty to expand the service and rehabilitation of the exiting aged pipes. Generally The United Nation (UN-HABITAT, 2002) report, tariffs in developing countries is set well below the level needed to cover even operation and maintenance costs. Research has shown that low tariffs are set largely for political, rather than practical, purposes

Limited institution capacity is also one of the bottlenecks that hinder cities of developing countries for managing their infrastructure asset in general and water supply in particular. Besides to low coverage, water losses (physical loss) in urban water supply is accounted to more than 50% of the supplies that mainly arise from

- Leakage of pipes, joints and valves
- Over flowing service reservoir and
- Waste of water through illegal connection and non metered house connections

Although leakage is one of the major causes for loss of water in networks distribution system, the loss of water through illegal connections and non-functioning meters is also contributing a lot; that needs a proper management and monitoring system.

While developed cities have started using on-line continues operation and monitoring service, the developing cities have grate difficult even to collect information on their previously performed operation and maintenance activities that could help them developing a strategy for the future. Many developed countries use water audit procedures to determine the efficiency of the system and to identify the location and magnitude of water losses.

There is also a need for some type of database or information system such as GIS to enable analysis of flows in the networks and provide early warning or indication of leakage. At present, although some cities in developing countries are introduced GIS based information system, many countries are still applying conventional methods for collecting, storing, processing and retrieval of information system, but the good news is that GIS have the ability to use previously collected and stored digital data makes introducing GIS easy and not costly.   Modeling urban networks and intermittent water supply systems is a challenging task because these systems are not fully pressurized pipeline networks but networks with very low pressures, with restricted water supply hours per day, and with thousands of roof tank connections. The alternate emptying and refilling of water pipe lines makes it problematic to apply standard EPANET based hydraulic models because of low pressures and pipes without water.

EPANET source code was adjusted to allow for modeling pressure dependent demands, for dealing with low pressure and "dry pipe" situations. A configurable tool was developed for incorporating roof tanks into the water supply analysis and for better formulation and schematization of the system hydraulics. Cases studies, water distribution model of Goterra site, Addis Ababa are used to illustrate the practical use of this approach.

Hydraulic analysis of flows and pressures in a distribution system has been a standard form of engineering analysis since its development by Dr. Hardy Cross in 1936. Water distribution system computer models have been in use since the middle 1960s and have evolved into sophisticated, user-friendly tools that are capable of simulating large distribution systems (Walski, Chase, and Savic, 2001).

In more recent years, the ability to model water utility and water age has been added to hydraulic models (Clark and Grayman, 1998). There are many commercial models that offer a wide range of capabilities in distribution system modeling EPANET is an open-structured, public domain hydraulic and water quality model developed by USEPA and is used worldwide (Rossman, 2000).

In order to facilitate the examination of required pipe sizes, the standard EPANET model was modified for use in the study project.

1.2 Research Problem

Leakage is often a large source of unaccounted for water and is a result of either lack of maintenance or failure to renew ageing systems. Leakage may also be caused from poor management of presser zones, which result in pipe or pipe joint failure. Although some leakage may go unnoticed for a long time, detection of visible leakage also requires good reporting which includes some level of public participation.

Water shortage and frequent service interruption is not only as a consequence of the shortfall between demand and supply but also as result of unidentified leakage and complicated network systems. Depending on the context of the existing system both or one of the factors may be found as a root cause for the shortfall between demand and supply. According to estimates of the United Nations (UN-HABITAT, 2002), Addis Ababa, which is projected to be the fourth largest city in Africa by 2015, has a current water supply shortfall of 25%. On top of the 25% backlog, as a cause of limited resources that as a result lead to poor maintenance and management of the water supply lines, leakage is observed as one of the main problems of the authority.

1.3 Research Objective

The main objective of the research is to evaluate the supply coverage and explore the water loss in city water supply distribution and suggest a method to better identify and reduce the loss.

Taking the main objective as mentioned above, the following specific objectives are expected to be achieved:

- To identify component of non revenue water (NRW) in water supply
- To introduce District Meter Area (DMA) in NRW reduction and control
- To quantify the benefit of water loss reduction with that of demand satisfaction and economical benefit.
- To test and evaluate loss reduction strategies on a sub-system
- Updating existing network and propose new pipe size
- To evaluate the domestic water supply coverage and distribution
- To evaluate the total loss of water at the city level
- Redesign the sub-system with modeling

1.4 Description of the Study Area

Addis Ababa has been a center of economic, social and political affairs/activities of the country for over 100 years. Addis Ababa at the moment covers 540 square kilometer land area as obtained from city map. The city lies between 2000 and 3000 meters above sea level, enjoying mild and warm temperature climate. Since it is established, the population is increased at alarming rate. According to the Central Statistical Authority of Ethiopia (CSA) 2006 and 2010 population census, this gives a population of 2.793 million and 2.917 million in 2006 and 2010 respectively and average annual growth rate of 2.5% the city is administratively divided into ten sub-cities and 99 woredas. Fig 1.1 depicts the general location for the study areas. Provision of water supply to the people of Addis Ababa is the responsibility of Addis Ababa Water and Sewerage Authority (AAWSA). AAWSA is a public authority and organized into eight branch offices, namely Arada, Addis Ketema, Nifas Silik, Mekenisa, Gulele, Megenagna, Gured Shola and Akaki which provide services to all parts of the city.

Water Supply and Distribution

The city has started getting water supply in 1901. During the years between 1942 and 2010 many water supply projects have been implemented that the construction and upgrading of Legedadi dam and treatment plant, improvement of the distribution, ground water and spring (wells) development of among them (AAWSA, 2004).

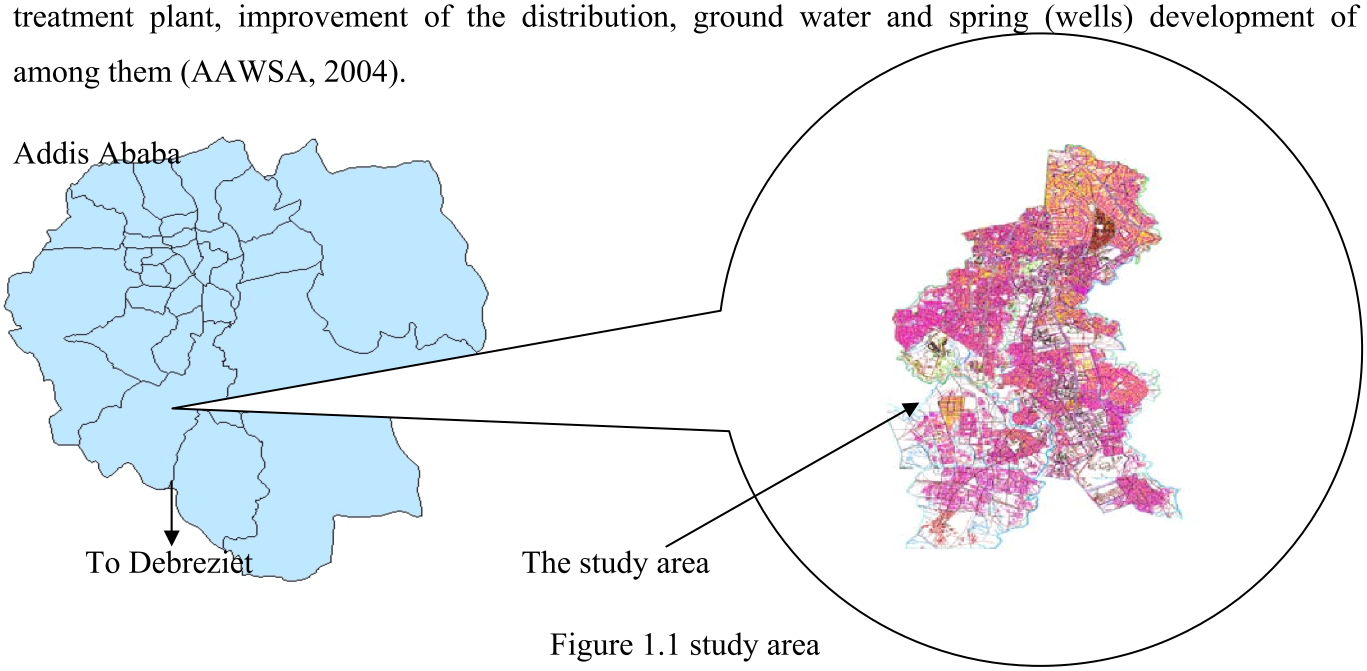

Figure 1.1 study area

Currently, around 220,003 m^3/day of water produced from different sources that among the other are Gefersa and Legedadi treatment plants that design capacity of 30,000 and 150,000 m^3/day respectively. The Gefersa consisting of 400mm steel pipe and the Legedadi line partly comparison of 1400mm (6.8Km) and other a combination of two parallel lines of diameter 900mm and 1200mm (11.5Km) are the main transmission lines that convey water from the treatment plant to the respective reservoirs.

The existing water supply system has 27 pumping situation and 22 balance (storage) reservoirs ranging in capacity from 100 to 20,000m^3 with a total approximate storage capacity of 87,000m^3. At present 324 Km^2 out of 540Km^2 total area of the city (60%) are served with water.

Water demand and Consumption

One of the difficulties faced by the water authority is determining the accurate water demand of the city as the consumption during the past years that should have been used as a base is far below the actual demand due to the shortage of water. Consumption of water for the city is therefore estimated based on the amount supplied rather than the actual demand. For these reason estimate of the future demand by the water authority is found to be uncertain. The current situation as summarized by the water authority is as shown below (AAWSA, 2006)

People having in-house services that are estimated about 4% of the total population use water on average between 80 and 100 liters per capital per day, while the remaining populations with access to safe drinking water (94%) are served by yard connection and use 15 and 30 liters per capital per day.

Non domestic uses excluding industrial and industries water use are about 25 liters per capital per day and 7 liters per capital per day respectively. From the water used by industries about 40% is provided by the water authority while the remaining amount is produced by the industries themselves from deep wells.

The Addis Ababa Water and sewerage Authority (AAWSA) is a public institution in the city in the water sector that is responsible for the supply of potable water and collection, treatment and disposal of water and sludge for the city. The authority is supervised by a board and directly responsible to the city manager. According to the overall structure development plan of the city, 100% of water supply is planned to be ensured by the coming year and water consumption to reach the rate of 140 liters per capital per day during the coming year. The main source of the city is being extracted predominant from three surface water reservoirs (dam) called Legedadi, Gefersa and Dire supported with different wells and springs.

2 Literature Review

2.1 Introduction

Problems in providing satisfactory water supply to the rapidly growing population especially that of the developing countries is increasing from time to time. Water supply system in urban areas are often unable to meet existing demands and are not available to everyone rather some consumers take disproportionate amounts of water and the poor is the first victim develop and expand water supply projects and one of the difficulties among the other is managing and reducing losses of water at all levels of a distribution system. As a result of the overall shortage of water many cities are faced a problem in distributing the available water impartially among the residents. Besides the poor management of existing infrastructure asset increases the level of water losses in water supply. As this research deals with overall coverage of water supply, water loss and modeling of distribution networks' well-maintained water distribution system is a major asset for any city or community.

Continuous monitoring and maintenance of the distribution network is the key step in meeting pressure and flow requirements, and water quality standards. A recent drinking water infrastructure needs survey (USEPA 2001) estimates an investment of 151 billion dollars over a period of 20 years to provide safe drinking water for US customers. Reducing water losses in pipe networks can minimize the maintenance costs and further improve the performance of pipe networks.

Leakage can be defined as unintentional or accidental loss of water from the pipe distribution network (Smith et al, 2000). Leaking pipes are a major concern for water utilities around the globe (Table 1.1) as they constitute a major portion of water losses. One of the primary reasons for leakage in pipes is aged and deteriorated networks. The condition of existing old networks can only worsen and further increase water losses. In the globe alone, 50% of supplied water is lost as leakage in some of the older networks (Jowitt and Xu, 1990). Leakage rates are also related to length of pipes and number of connections. Improper connections can sometimes result in continuous escape of water from the distribution pipes.

Table 2.1 Network leakage rates around the globe.

country	**Leakage Rate %**
Netherland	5
Japan	11
USA	12
France	15
Korea	16
UK	28
India	30

(Source:www3.akwien.at/pdf/uv/university_of_greenwich.pdf).

2.2 Comparing water losses

The amount of water loss differs from country to country, city to city and even from network to another network in the same city. Different countries uses different indicators to evaluate their states in comparison with others and to compare the distribution of water loss from one location to other location of a distribution system in order to take action based on the level of loss. As stated above competition using unaccounted for water (UFW) expressed as percentage has limitation when used for comparison as it highly depends with the volume of water produced.

The traditional performance indicators of water losses are frequently expressed as a percentage of input volume. However, this indicator fails to take account of any of main local influences. Consequently it cannot be an appropriate performance indicator (PI) for comparison (WHO, 2001).

Deplaned upon the consumption per service connection, the same volume of real losses/service connection/day, in percentage terms, is anything from 5% to 30%. Thus developing countries with relatively low consumption, can appear to have high losses when expressed in percentage terms, percentage loses for urban areas in developed countries with high consumption can be equally misleading (Farley and Trow, 2003).

To avoid for the wide diversity of format and definitions related to water loss, many practitioner have identified an urgent need for a common international terminology that among them task forces from the international water association (IWA) recently produced a standard approach for water balance calculation with a definition of all terms involved as indicated in table 1.2 below.

Table 2.2 IWA standard international terminologies

<table>
<tr><td rowspan="9">System Input</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 Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption</td><td>Unbilled Consumption</td><td rowspan="7">NON-REVENUE WATER</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water Loss</td><td rowspan="2">Apparent Loss</td><td>Unauthorized Consumption</td></tr>
<tr><td>Metering Inaccuracies</td></tr>
<tr><td rowspan="3">Real Loss</td><td>Leakage From water mains</td></tr>
<tr><td>Leakage from storage tank</td></tr>
<tr><td>Leakage from service (up to revenue meter)</td></tr>
</table>

(Source, Farley and Stuart)

According to IWA the above terminologies are defined as below:

- System input volume is the annual volume input to the part of the water supply distribution system
- Authorized consumption is the annual volume of metered and/or non-metered water take by registered customers, the water supplier and other who are implicitly or explicitly authorized to do so. It includes water exported, and leak and overflows after the point of customer metering.
- Non-Revenue Water (NRW) is the difference between system input volumes and billed authorized consumption.
- Water losses are the difference between system input volume and authorized consumption, and consist of apparent losses and real losses.

- Apparent losses consist of unauthorized consumption and all types of metering inaccuracies.
- Real losses are the annual volumes lost through all types of leaks, bursts and overflows on mains, service reservoir and service connections, up to the point of customer metering.

2.3 Cause of water losses

Leakage is usually the major component of water loss in developing countries, but this is not always the case in developing or partially developed countries, where illegal connections, meter error, or an accenting error are often more significant (Farly and Trow, 2003) the other component of total water loss are non-physical losses, e.g. Meter under registration, illegal connections and illegal and unknown use (WHO, 2001)

2.4 Pressure and Leakage

In many water network systems, even though the total demand and the total loss of water can be known rather easily, information about the possible influence of local pressure upon demand is sadly lacking that as a result creates difficultly to assess and compare the demand and loss of water in its spatial distribution. Pressure distribution system on the one hand contributes to the shortage of water that as a result causes for unequal distribution of water among residents. To alleviate such problems, some water authorities develop a zoning scheme whereby the complete water distribution network is broken down in to manageable segments that can be easily metered and monitored and analyzed.

The leakage from water distribution system has been shown to be directly proportional to the square root of the distribution system pressure as indicated by the relationship below (Wallingford HR, 2003).

Leak detection techniques that are in use in the water industry involve two major steps.

i. Estimation of leakage rates
ii. Location of leak

2.5 Pressure Management Through Distribution System

Pressure management can be defined as the practice to manage system pressures to an optimum level of service ensuring sufficient and efficient supply to legal uses and consumers, while eliminating or reducing pressure transients and variations, faulty level controls and reducing unnecessary pressures, all of which cause the distribution system to leak and break unnecessarily. There are many different tools that can be used when implementing pressure management, including pump controls, altitude controls and sustaining valves [Lambert et al., 2006]. It was reported that many water utilities introduced pressure management to their water distribution systems. In the most cases, large reductions in a new break frequency can be achieved over a wide range of pressures. In Australia, Canada, German and Italy, ongoing monitoring shows that the reductions in break frequency have been sustained for over five years to date by implementing pressure management procedure [Lamber et al., 2006]. On the other hand, the rapid reduction in new break frequency following pressure management is immediately evident for water loss management. Some of the pressure management benefits reported by many different utilities include:

- Reduction in annual repair costs
- Reduction of the repair backlog, shorter run times for bursts
- Fewer emergency repairs, more planned work
- Reduced inconvenience to customers

Calculations of the economic benefit of pressure management have been based on the predicted reduction in flow rates of existing leaks and the value of the water thus saved. If management of excess pressure can also regularly achieve reduction in numbers of breaks of between 28% and 80% per year [Lamber et al., 2006], the annual savings in repair costs will usually be far greater than the value of the water saved.

Replacement of mains and services, the most expensive aspect of water distribution system management, is normally initiated by break frequencies that are considered to be excessive. Most water utilities consider break frequency to be a factor outside their control, and something that can only be remedied by expensive replacement of mains and services. However, if pressure management can reduce break frequencies and extend the working life of parts of the distribution infrastructure by even a few years, the economic benefits would generally be even greater than the short term reduction in repair costs.

2.6 **Leakage Monitoring With District Meter Area (DMA)**

A flow measuring system in a water distribution system should include not only measurement of total flows from source or treatment plants, but also zone and district flows. This allows the engineer to understand and operate the distribution system in smaller areas, and allows more precise demand prediction, leakage management and control to take place. The measurement system must therefore be hierarchical at a number of levels, beginning at production measurement, via zone and district measurement and ending at the customer's meter.

The technique of leakage monitoring is considered to be the major contributor to cost-effective and efficient leakage management. It is a methodology which can be applied to all distribution networks. Even in systems with supply deficiencies leakage monitoring zones can be introduced gradually. One zone at a time is created and leaks detected and repaired, before moving on to create the next zone. This systematic approach gradually improves the hydraulic characteristics of the network and improves supply.

Leakage monitoring requires the installation of flow meters at strategic points throughout the distribution system, each meter recording flows into a discrete district which has a defined and permanent boundary. Such a district is called a district meter area and the concept of design and operation of DMA has been detailed in elsewhere Farley and Trow, 2003.

The design of a leakage monitoring system has two aims:

1. To divide the distribution network into a number of zones or DMAs, each with a defined and permanent boundary, so that night flows into each district can be regularly monitored, enabling the presence of unreported bursts and leakage to be identified and located.
2. To manage pressure in each district or group of districts so that the network is operated at the optimum level of pressure.

It therefore follows that a leakage monitoring system will comprise a number of districts where flow is measured by permanently installed flow meters. In some cases the flow meter installation will incorporate a pressure reducing valve.

2.7 Water Audits

Unaccounted-for water accounts for authorized unmetered use and under-registering metered use in addition to water lost as leakage (Smith et al., 2000). An overall survey of leaks is done as a part of water audit since, leakage is considered as a major component of unaccounted-for water.

A water audit involves comprehensive accounting of the total water pumped at a service station and water utilized at the consumer end. The water supplied at a utility is measured while being pumped into the network, and water consumed is obtained from billing records. Water lost as leakage is estimated from simple mass balance principle which can be stated as the difference between the amount of water produced at the utility and amount of water purchased by consumers.

Flow measurements taken in the distribution network give more precise estimates of leakage rates. A portion of the network is isolated using valves and measurements of flows entering the isolated portion of the network are taken for at least a period of 24 hours. Mass conservation principle is applied to that part of the network to estimate the average amount of leakage rate.

However, such methods give only an approximate estimate of leakage rates.

2.8 Leak Location

The second step in leakage control involves location of leaks across the distribution network. Acoustic equipment combined with correlation methods are generally used to locate leaks. Acoustic methods are based on changes in sound of the escaping water (Smith et al., 2000). Acoustic sound transducers are placed in contact with ground surface to listen to any abnormalities in the underlying pipes. In acoustic equipment accompanied by noise correlates, the acoustic signals from transducers are transmitted to a receiving unit, where the signals are processed automatically. Other leak detection methods employed are infrared thermograph, tracer gas methods, and mechanical drilling of soil.

2.9 System Evaluation and Design

The designing and evaluating of community water supply distribution systems has to consider the amount of water for the commercial interests, governmental property, educational facilities, and all classifications of residential property as presented above in a general relationship to average and maximum daily consumption demand. At any time of the day, the day of the week, or the week of a given year, a structure fire or other fire emergency such as transportation vehicle fires or, in some cases, natural cover fires may erupt. Water is the primary agent of choice to confine, control, and extinguish structural fires. Some new development in fire extinguishing agents may be used for rapid knockdown of a fire, but a well-developed structure fire still requires established needed fire flows from fire hydrants to control and extinguish developing fires. Each community needs to evaluate and design or modify the design of the community water system to meet present-day needs to address future demands based on growth of the built area and population increases, along with the need to meet EPA criteria for water quality,. This will be an ever-increasing demand and challenge for every community water distribution system. Some specific guidelines on consumer consumption requirements and needed fire flows are established by the ISO, which represents in excess of 130 property and casualty underwriters in the United States in developing advisory insurance rates. The following topics address some fundamental information on understanding 1) water system demands, 2) determining design flow, and the very important topic of 3) water storage on a community water system. This should provide community leaders, municipal officials, fire department officials, water supply superintendents, and consulting engineers on water systems, a common knowledge base so that they all can sit at the same table and have a meaningful dialog about the present and future state of a specific water system and even how it may relate to adjacent water supplies in nearby community water systems.

2.9.1 Water system demands

Water demands need to be assessed on the basis of the following considerations:

a. The year and date the water supply system was commissioned or started supplying water through the distribution system. Hopefully, there is a water supply system map that will detail where pipes were laid, the size of the pipes, the pipe material, the location of the valves, the location, size and type of fire hydrants along with the lateral size and valve arrangements.
b. These water maps need to show all extension and changes to the water system covering the same topics as above.

A clear and accurate knowledge of the water system in place is needed before discussing changes to be made to existing systems. If this is to be a new community water system, all of these details should be laid out on a proposed street map for evaluation. If an Engineering Firm uses different criteria, it should conform to the most current publications of the AWWA, the National Fire Protection Association (NFPA), the ISO, the Civil Engineering Handbook, and any special State or county regulations.

There should be no oversight in considering both short-term and long-term goals. The primary objective is to make sure that the community is being serviced adequately. If there are deficiencies in meeting current or future goals because of economic constraints, this needs to be identified for the areas of the community where there may be inadequate flows to meet consumer consumption during peak water demand, so that constraints such as watering lawns and washing cars, can be placed on water usage. If available fire flows do not meet needed fire flows in specific districts of the community, the fire department needs to know these conditions on virtually a real-time basis. The local fire department may need to plan on relaying water from larger supply mains to fire sites using large- diameter hose or the existing water supply may need to be augmented by an alternative water supply using mobile takers from adjacent fire departments under automatic-aid and mutual-aid arrangements. Another alternative is to provide retention ponds in the community to capture runoff. Retention ponds may be outfitted with dry hydrants as a supplementary water supply for fire protection.

2.9.2 Planning water demand changes

Special planning is needed when new water demands will be placed on a specific community water system. This point cannot be overstressed. The amount of construction performed and the amount of construction that realistically can be accomplished to provide adequate service are dependent on when the construction will be needed. Ultimately, final development should be consistent with the utility's ability to provide consumer consumption and fire protection at the same time. Fire protection must not lag behind supplying domestic taps, as often occurs in new residential areas of communities. The planning and installation phase should assure that water supply for fire protection is never interrupted. There have been too many large-loss structure fires because the water system was shut down in the vicinity of the fire site.

The ideal way to develop a water distribution system would be to construct a distribution network of pipe that would adequately serve the short-range and long-range development of the service area. Individual construction projects, developments, subdivisions, and industrial complexes then could be developed without checking for adjustments to ensure that the original design plans remain adequate for all projected consumer consumption and fire protection demand. However, in reality, the best of plans needs to be adaptable to change measures where growth, moves, and demand may decline in older portions of a community. Therefore, the design for the source, through the treatment plant to the distribution system, must provide for growth and change in the delivery demand points. The best water system is the one that is designed with a vision for the future. Existing water systems have to evaluated and redesigned with a future perspective that includes a rehabilitation and/or replacement of existing system components due to the age factor. The maintenance of old infrastructures may be more expensive than the replacement with a better designed system that will meet both the future needs of water quality and distribution system demand.

The bottom line is that a water supply system cannot remain constant. It is the responsibility of elected officials, water supply superintendents and their staff, hydrologist, geologists, professional civil engineers, rural and urban planners as appropriate, fire officials and fire protection engineers, and representatives from the insurance industry to sit at the table and plan water systems for the future with due consideration to all of the regulations and requirements that are being placed on water systems at this time and in the future, especially as programmed by the EPA. The cost to do this is not going to be any small thing, so the financial planning is just as important as the physical planning.
It is now recommended that every 3 to 5 years, as a minimum, existing water distribution systems be evaluated thoroughly for requirements that would be placed on it by development and reconstruction for a 20-year period into the future. A plan then should be developed for meeting those needs. In this way, individual improvements and projects can be evaluated and made to conform, generally, to long-term development and contingency plans for such events as serious system interruptions caused by natural disasters and terrorism attacks without undue additional expense to either the developer or the utility.

A water distribution system is a pipe network which delivers water from single or multiple supply sources to consumers. Typical water supply sources include reservoirs, storage tanks, and external water supply at junction nodes such as groundwater wells. Consumers include both municipal and industrial users. The pipe network consists of pipes, nodes, pumps, control valves, storage tanks, and reservoirs. EPANET views the water distribution system as a network containing nodes and links, where the nodes are connected by links. Figure 2.1 illustrates a node-link representation of a simple water distribution network.

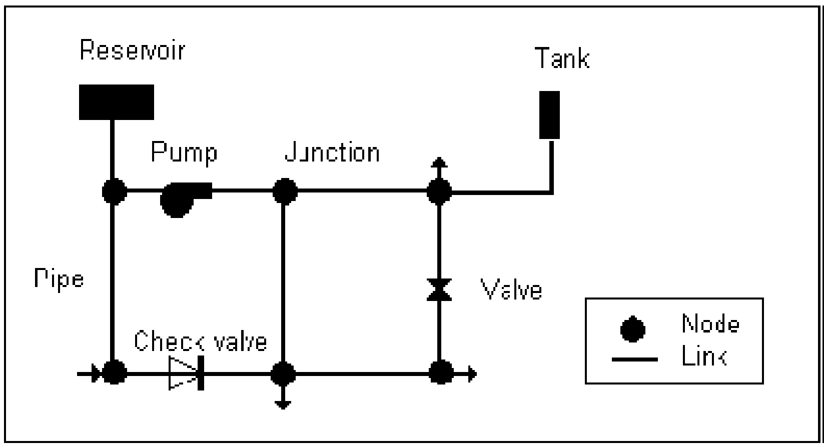

Figure 2.1 Node-link representation of a water distribution network

How the EPANET program models the hydraulic behavior of each of these components is described in the following sections. All flow rates in this discussion will be assumed as liters per second (L/s).

Pipes

Every pipe is connected to two nodes at its ends. In a pipe network system, pipes are the channels used to convey water from one location to another. The physical characteristics of a pipe include the length, inside diameter, roughness coefficient, and minor loss coefficient. The pipe roughness coefficient is associated with the pipe material and age. The minor loss coefficient is due to the fittings along the pipe. When water is conveyed through the pipe, hydraulic energy is lost due to the friction between the moving water and the stationary pipe surface. This friction loss is a major energy loss in pipe flow and is a function of flow rate, pipe length, diameter, and roughness coefficient.

The head lost to friction associated with flow through a pipe can be expressed in a general fashion as:

$h_L = aq^b$ ---2.1

Where

hL = head loss, m

q = flow, L/s

a = a resistance coefficient

b = a flow exponent

EPANET can use any one of three popular forms of the head loss formula shown in Equation 2.1: the Hazen-Williams formula, the Darcy-Weisbach formula, or the Chezy-Manning formula. MIKE NET allows the user to choose the formulation to use.

The Hazen-Williams formula is probably the most popular head loss equation for water distribution systems, the Darcy-Weisbach formula is more applicable to laminar flow and to fluids other than water, and the Chezy-Manning formula is more commonly used for open channel flow. Table 5.1 lists resistance coefficients and flow exponents for each formula. Note that each formula uses a different pipe roughness coefficient, which must be determined empirically. Table 2.3 lists general ranges of these coefficients for different types of new pipe materials. Be aware that a pipe's roughness coefficient can change considerably with age.

While the Darcy-Weisbach relationship for closed-conduit flows is generally recognized as a more accurate mathematical formulation over a wider range of flow than the Hazen-Williams formulation, the field data on E values (required for the Darcy-Weisbach formulation) are not as readily available as are the C values for the pipe wall roughness coefficient (used in the Hazen-Williams formulation).

Table 2.3 Roughness coefficients for new pipe

Material	Hazen-Williams C	Darcy-Weisbach ε, millifeet	Manning's n
Cast Iron	130 - 140	0.85	0.012 - 0.015
Concrete or Concrete Lined	120 - 140	1.0 - 10	0.012 - 0.017
Galvanized Iron	120	0.5	0.015 - 0.017
Plastic	140 - 150	0.005	0.011 - 0.015
Steel	140 - 150	0.15	0.015 - 0.017
Vitrified Clay	110	----	0.013 - 0.015

Valves

Aside from the valves in pipes that are either fully opened or closed (such as check valves), EPANET can also represent valves that control either the pressure or flow at specific points in a network. Such valves are considered as links of negligible length with specified upstream and downstream junction nodes. The types of valves that can be modeled are described below.

Pressure reducing valves (PRV) limit the pressure on their downstream end to not exceed a pre-set value when the upstream pressure is above the setting. If the upstream pressure is below the setting, then flow through the valve is unrestricted. Should the pressure on the downstream end exceed that on the upstream end, the valve closes to prevent reversal of flow.

Pressure sustaining valves (PSV) try to maintain a minimum pressure on their upstream end when the downstream pressure is below that value. If the downstream pressure is above the setting, then flow through the valve is unrestricted. Should the downstream pressure exceed the upstream pressure then the valve closes to prevent reverse flow.

Pressure breaker valves (PBV) force a specified pressure loss to occur across the valve. Flow can be in either direction through the valve.

Flow control valves (FCV) limit the flow through a valve to a specified amount. The program produces a warning message if this flow cannot be maintained without having to add additional head at the valve.

Throttle control valves (TCV) simulate a partially closed valve by adjusting the minor head loss coefficient of the valve. A relationship between the degree to which the valve is closed and the resulting head loss coefficient is usually available from the valve manufacturer.

Node

Nodes are the locations where pipes connect. Two types of nodes exist in a pipe network system, (1) fixed nodes and (2) junction nodes. Fixed nodes are nodes whose HGL are defined. For example, reservoirs and storage tanks are considered fixed nodes, because their HGL are initially defined. Junction nodes are nodes whose HGL are not yet determined and must be computed in the pipe network analysis. Degree of freedom, elevation, and water demand are the three important input parameters for a node (see Figure 2.1). A node's degree of freedom is the number of pipes that connect to that node. In EPANET, a junction node may be connected to more than one pipe, but a fixed node (reservoir) must be connected to exactly one pipe. Therefore, a fixed node's degree of freedom is always one, and a junction node's degree of freedom may be greater than one. The elevation of a node can sometimes be obtained from system maps or drawings. More often, it is approximated using topographic maps. Water demand at a junction node is the summation of all water drawn from or added to the system at that node.

All nodes should have their elevation specified above sea level (i.e., greater than zero) so that the contribution to hydraulic head due to elevation can be computed. Any water consumption or supply rates at nodes that are not storage nodes must be known for the duration of time the network is being analyzed. Storage nodes (i.e., tanks and reservoirs) are special types of nodes where a free water surface exists and the hydraulic head is simply the elevation of water above sea level.

3 Water loss Analysis

3.1 Introduction

The reduction and control of water loss is becoming even more vital in this age of increasing demand and changing weather patterns that bring droughts to a considerable number of locations in the world. Many water utilities have been developing new strategies to reduce losses to an economic and acceptable level in order to preserve valuable water resources. In Addis Ababa, water is supplied by a municipal, Addis Ababa water and sewerage authority, and this is usually the best assurance of an uninterrupted supply of economical and safe water to our people in cities.

The components of water demand are Domestic like residential, Non Domestic like commercial, industrial, institutional and Fountains like public water uses, and unaccounted system losses and leakages. While all components generate revenue to the utility, the unaccounted system loss and leakages are not associated with total cost revenues, and are a source of wasted production costs. With today’s high water production costs and rates, the expense of detecting and mitigating the unaccounted for water and leakages is an attractive option for minimizing operating expenditures. The water utility benefits by:

(a) Saving the production costs of the water,

(b) Increasing revenues through sales of water saved,

(c) Deferring the system expansion and capital expenditures through the capture of lost water,

(d) Reducing increases in utility rates, and thus maintaining better consumer relations.

The annual volume of water loss is an important indicator of water distribution system efficiency, both individual years and as a trend over a period of years. High and increasing water losses are an indicator of ineffective planning and construction, and of low operational and maintenance activities. In Ethiopia cities, the average yearly water loss is as high as 37% of the water volume produced based on Addis Ababa water and sewerage Authority (AAWSA, 1997).

This study was aimed to propose an effective water loss management and water supply coverage in Addis Ababa city. Thus, water loss approach was advised for present study to manage water loss problem in Addis Ababa. In the study, in addition to technical loss, some engineering proposals for the effective control of nontechnical loss and general water economy were suggested.

The water loss analysis is done and compared both at city level and sub-system level. Although the data has been collected for each customer meter or contract of the entire city, as water production was only available at the city level, the water loss analysis has been done focusing at the city and the selected sub-systems level that data is found. Two main line systems that have their own isolated pipes have been selected to analysis and compare the spatial distribution of the water loss in the sub-system.

3.2 City level water loss analysis

The total annual water produced and distributed to the distribution system and the water billed that was aggregated from the individual customer meter readings were used to quantify the total water loss for the city.

Table 3.1Annual Water loss from the year 2005 to 2009

year	Total production	Total Billed data	total loss	% of losses
2005	80,073,546	45,520,623	34,552,923	43.15
2006	82,820,272	51,338,591	31,481,681	38.01
2007	86,278,592	53,272,445	33,006,147	38.26
2008	88,405,625	53,676,740	34,728,885	39.28
2009	92,201,273	58,140,949	34,060,324	36.94

The total annual water loss of the city is decreased from year to year at the end of the year 2009 the annual water produced and distributed to the system within specified year was 92,201,273 cubic meters and annual water loss as derived using the above expression was 34,060,324 cubic meters which account to 36.94% of the total production.

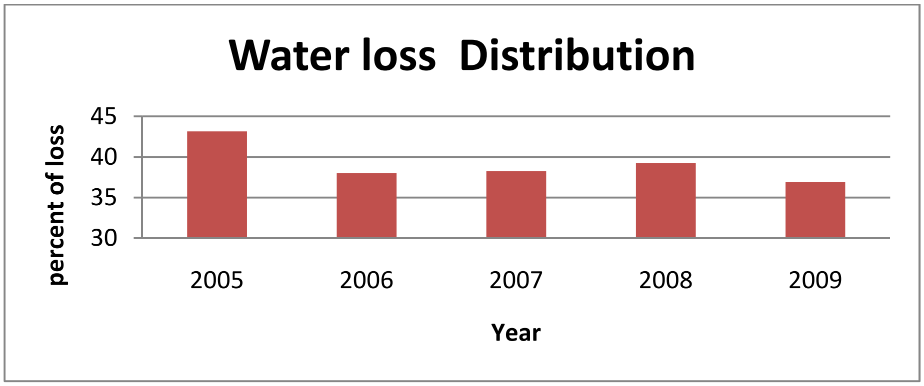

Figure 3.1 Annual water loss of the city

3.2.1 Water loss as per number of connection

Water loss expressed as a percentage could be an appropriate means to show the extent of the loss within a given environment, but it is not a good indicator for comparing the loss from one area to another. According to some literature comparison of water loss between different areas is recommended to be done using the water loss per service connection per day. Taking the total number of connection in the city as 297,500 AAWSA, the water loss per connection for the similar duration was derived as,

Water loss=313.67 liter/connection/day.

3.3 Water loss analysis at sub-systems

The methods used in Non-Revenue water reduction control include the application of District Metering for estimation of water loss. Application DMA at the city level is not practice yet but in most developed and developing city used a lot. This part of a study review a method how the South Africa Water Supply practiced active Leakage control methods (using DMA) in pilot area .

The investigation carried out in the pilot areas made in different steps as

Step 1: Site identification of the network and its appurtenances

Step 2: Preliminary general zone monitoring

This was made to make sure that the pilot area could be isolated from the remaining network and further check the possibility of subdividing the subzones pilot area units. Thus all boundary valves were closed and their tightness checked. A flow meter was installed at supply point of each subzone for checking the flows after valve closure.

Step 3: Leak Quantifying

In this step flows measured by isolating a certain portion, so that any point in the portion or subzone could only be supplied from one supply lines. The result of this operation could indicate the volume of water lost through the leaks, provided that one knows the normal night consumption and that of big consumer, which should be deducted from the measured flows.

Step 4: Leak Detection and Repair

Here, the leak quantified in step two was detected for its location the leak points within the investigated portion were precisely identified and located. Then the leaks were repaired.

Step 5: Night Flow Monitoring after leak repairs

In order to, again quantify the amount of UFW; night flow monitoring is performed after the identified leaks have been repaired.

This study is focus on calculation of water losses in the pipe network of the area and the amount water saved by Water Balance Method using International Water Association (IWA).

After preparing the schedule immediately select the pilot area. the selected pilot area are around Gottera, this place is found in Kirkos sub-city and under Addis Ababa water and sewerage Authority Nifas silik branch, discuss with the staff member and then arrange different material and man power, finally selected the area from the branch based on the following criteria.

- Hydraulically easily desecrate area
- 24hr water availability
- Customer not more than 1000, and
- More leakage complain

During bulk meter installation firstly checking that if there is any other outlet pipes from the pilot area, and close the inlet valves and investigate each and every neighbors around the boundary which they get water or not. During investigation we get 20 private connections are broken around Gottera then repair the connections and install the bulk meter.

Figure 3.2 Sub-system water distributions Network

From the above network diagram we have two inlets or sources for our pilot area:

- The first one is from wereda 06 which has a diameter of 90mm.
- The second one branch outs from 200mm main line which has a diameter of 65mm.

After knowing the inlet & the outlet water consumption install two bulk meters a diameter of 90mm with HDPE man holes. Because it is important to checking the functionality of the bulk meter monitored per a week.

The total water loss of each sub-system has been evaluated using a similar expression to that of the city level analysis.

The distributed water to the sub-system and that of consumed from is shown in the table 3.2 below and the corresponding total water loss analysis shown in the table.

Table 3.2 Water loss in Sub-system

<table>
<tr><th>It no</th><th>Bulk meter one 3"</th><th>Bulk meter two 2 1/2"</th><th>Total water consumption</th><th>Total flow to the system</th><th>Water loss from the system</th><th>Water loss in %</th></tr>
<tr><td>1st</td><td>13788</td><td>9381</td><td rowspan="2">9690</td><td rowspan="2">14642</td><td rowspan="2">4952</td><td rowspan="2">33.82 %</td></tr>
<tr><td>2nd</td><td rowspan="2">18964</td><td rowspan="2">18847</td></tr>
<tr><td></td><td rowspan="2">21518</td><td rowspan="2">35396</td><td rowspan="2">13878</td><td rowspan="2">39.21 %</td></tr>
<tr><td>3rd</td><td rowspan="2">48726</td><td rowspan="2">24481</td></tr>
<tr><td></td><td rowspan="2">11430</td><td rowspan="2">19123</td><td rowspan="2">7693</td><td rowspan="2">40.23 %</td></tr>
<tr><td>4th</td><td rowspan="2">58085</td><td rowspan="2">34245</td></tr>
<tr><td></td><td rowspan="2">54267</td><td rowspan="2">86054</td><td rowspan="2">31787</td><td rowspan="2">36.94 %</td></tr>
<tr><td>5th</td><td>101264</td><td>77120</td></tr>
</table>

Two secondary lines that have data on production and consumption of water have been selected for analyzing water loss at the sub-system. The total amount of water loss at the sub-system is within three month is 58,310 cubic meter the total water produced and distributed to the sub-system within specified month has been 155,215 cubic meters and which averagely account to 37.54% of the total production has loss from the sub-system.

3.3.1 Water loss as per number of connection

Water loss expressed as a percentage could be an appropriate means to show the extent of the loss within a given environment, but it is not a good indicator for comparing the losses from one area to another. According to some literature comparison of water loss between different areas is recommended to be done using the water loss per service connection per day. Taking the total number of connection in the sub-system as 682 the water loss per connection for the similar duration was derived as,

Water loss=949.98 liter/connection/day.

3.4 Pressure and leakage relationships

If we know the relationship between pressure and leakage they are necessary to conduct the field test data. When we will see the two major sources data became,

- Short tests of pressure: leakage relationships on 20 small sectors Japanese distribution systems (Ogura, 1979), analyzed and presented in the form of a simple Power Law (Leakage L varies with Pressure P).
- Pressure: net night flow relationships from longer duration tests on 18 district metered areas in the UK where all detectable leaks had been located and repaired (Goodwin, 1980),

Leakage rate relationships, in its simplest form this is also a power law. Empirical quadratic and exponential relationship were also used (or rather, misused) in the UK and elsewhere from 1994 to 2003 to analyze test data and predict the effects of pressure management. However, it is now recommended by the Water Losses Task Force (Thornton 2003) and in the UK (UKWIR, 2003), that the most physically meaningful and 'Best Practice' form of equation for representing pressure, leakage rate relationships is a simple power law. There is no international convention for characters used for the exponent and the water Losses task Force uses the alpha-numeric 'N1', resulting in the equations:

L varies with P^{N1} …………… ……………….. (1)

And $L_1/L_0 = (P_1/P_0)^{N1}$ …..………………… ……….. (2)

So, if pressure is reduced from P_0 to P_1, flow rates through existing leaks change from L_0 to L_1, and the extent of the change depends on the exponent N1. When N1 is calculate from the non-dimensional Reynolds Number (Re), Circular holes:

- N1 near 0.5 for metal & PVC pipes for Reynolds Number Re > 4000
- N1 likely to be near 0.5 for polyethylene and AC, and Re >4000
- but N1 can be in range 0.5 to 1.0 for small leaks
- and N1 for corrosion hole clusters may be even higher

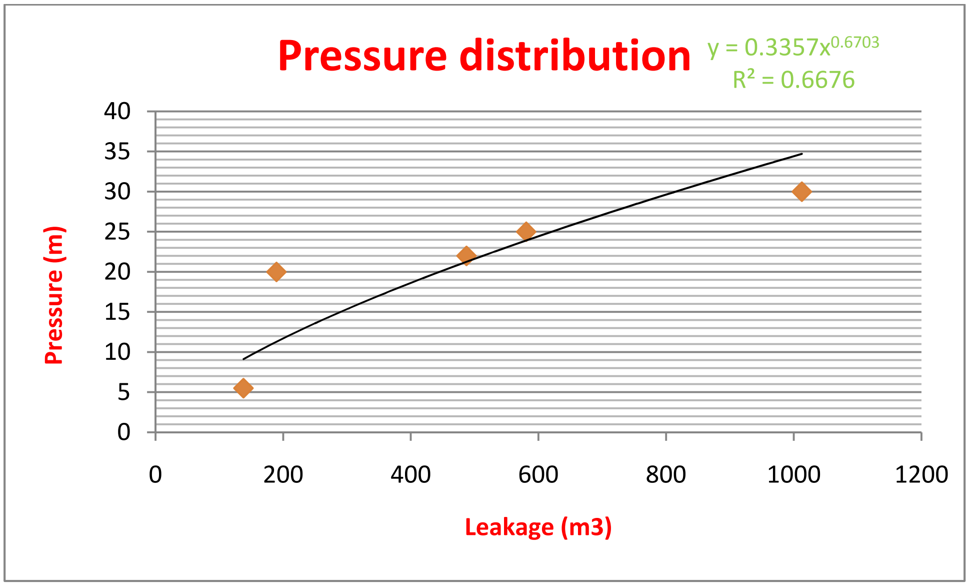

Figure 3.3 Pressure distribution

The Above numerical equation generating from continuous water flow and within the same duration they recorded the pressure and the water loss at the same period, the pressure is varied with time to time during supply the water to the system record the pressure. And then turn off the valve and the Bulk meter and the customer meter immediately start to reading,

The relationship of water loss (leakage) and pressure as you can see from the above graph, the graph is linear function.

Y: - is from the graph represent the pressure (m)

X: - is represent the Leakage (Water loss) (m^3)

From this expression the above graph function is $Y=0.335\ X^{0.670}$ and the factor $R^2= 0.667$ that means $P=0.335\ (L)^{0.670}$ which is the pressure and the water loss (leakage) are directly proportional when the pressure in distribution system is high the water loss is high.

3.4.1 Calculation of Actual Losses by Water Balance Method

The exact value of water losses is calculated by using Water Balance Method. This Water Balance Method requires a total average consumption for each zone before and after leakage reduction works, Mi and Me. The calculation of total average daily consumption is utilized using MS Office application. The bulk meter reading from Baseline 1a (at step 2) is subtracted from the bulk meter reading at Baseline 1b (after seven days of step 2). This gives the volumetric flow through the meter for seven- day period. This figure is divided by seven to give the average daily inflow, Qi. The volume of water used through normal domestic meters (downloaded from meter books) is added together to give total consumption for the corresponding seven-day period. This is again divided by seven to give a daily average consumption figure, Mi. It follows that Qi-Mi will give an accurate volumetric amount of the average daily Non-Revenue Water within the zone. This calculation done the same way after leakage reduction work, And use in the system the above pressure relation and finally we reduce the pressure from the top 35m to 25 m in the pilot area see the following table below summary of actual NRW reduction using water balance method for pilot area.

Table 3.3 Daily Average Inflow Calculations for Pilot Area

Base line Reading1:Before Leakage Reduction work	
Date	Meter reading (m^3)
26-Mar-11	69181
11-Apr-11	86054
Total Inflow=	16873
No of Days=	15
Daily Average Inflow	
Qi=	**1124.9**

Base line Reading1:After Leakage Reduction work	
Date	Meter reading (m^3)
06-July-11	115679.654
21-July-11	128929
Total Inflow=	13249.346
No of Days=	15
Daily Average Inflow	
Qi=	**883.29**

Table 3.4 Summary of Water Balance Method Results

Meter Book	Properties	Before Leakage Reduction work	After Leakage Reduction work
384	42	55.47	66.76
"	38	59.72	60.79
"	37	50.83	56.42
"	30	50.86	58.15
"	32	60.99	48.18
"	32	39.77	63.27
"	34	43	45.76
"	35	60.12	65.82
385	34	64.2	62.12
"	33	65	68
"	42	76.21	76.42
"	33	55.44	60.21
"	46	56.87	54.33
"	42	35.15	54.39
"	35	50.69	37.52
"	42	59.13	59.7
"	47	40	56
"	40	55	62
Total	**674**	**978.45**	**1055.84**

Result before Leakage For pilot Area Reduction Work	(m^3/day)
Average Inflow, Qi	1791.5
Daily Average Consumption, Mi	978.45
NRW before reduction works (Qi-Mi)	813.1
Result After Leakage For pilot Area Reduction work	**(m^3/day)**
Average Inflow, Qe	1549.96
Daily Average Consumption, Me	1055.84
NRW After reduction works (Qe-Me)	494.12
Result before and after Reduction work	**m^3**
Actual NRW Reduction,(Qi-Mi)-(Qe-Me)	318.97

The results of NRW reduction by Water Balance Method and pressure show the difference between predicted and actual of water losses in water distribution network. The results also shows that after leakage reduction control works took place, the volume of water loss in water distribution network has reduce about 39% for pilot area.

3.5 Comparing of water losses

Using the percentage and quantitative figures found from the above tables, comparing is made among the sub-system and the city based on the following approaches, the percentage of water loss, and loss per number of connections.

One of the limitations in comparing the total water losses in the city and that of the sub system was the duration of the data. Both the production and consumption data for the city was from 2005 to 2009 and the sub-system based on the man power averagely we read the meter and the bulk meter is between 15 to 22 day variation based on that the duration of the sub-system is three month from January to March 2011, The percentage of loss and loss per connection is summarized as shown in table 2.6 below

Table 3.5 Comparing of water losses

Network found discretion	Percent loss (%)	loss per connection (liter/connection/day)
City level	36.94	313.67
Sub-system	37.54	949.98

As we can see from the table the loss of the sub-system is high and also the loss per connection is more than double than the city loss per connection,

3.6 Dimensions of Water Loss in Distribution Networks

A safe, reliable and efficient water supply system is essential to any community. In a water supply system including the water supply works, treatment plant, distribution network and storage facilities, the most expensive component is the water distribution network [Kleiner, 1997]. In Addis Ababa, there are 10 districts, and their population is varied between 13,480 and 35,989 based on the bill data of the Authority.

3.7 Major Factors Contributing to High Level of Water Loss in Addis Ababa

There are several reasons for the high level of water loss in Addis Ababa. These factors are given below, and some advisory solutions were briefly proposed in next sections.

3.7.1 Age of pipe network

It is estimated that nearly more than 50% of the pipe network in the city was laid over 25 years ago. The main duties which made more than half a month is checking of each customer (door to door water connection) by sounding rod .In this time get so many invisible & visible leakages both on the private connection & also on the main line.

Totally up to compile this report identify 9135m service line, (682customer connections) from this we get 194 leaked connections, most leakages are easily detectable and the rest are cannot maintain because of its long service age. So we are decided to change by new line.

These lines are including DCI (ducktail cast iron) and carbon steel pipes. The aged pipe is especially in the central part of the city and in densely population areas like in Merkato, Piasa etc. All these materials suffer from degradation over time due to operational measures, environmental conditions and general wear and tear result in increased leakage in the network. It is therefore necessary to replace older mains so that less leakage occurs.

3.7.2 Poor maintenance of networks

In some places like expansion areas including Mekanissa Lebu, Bole Bulubula, CMC, etc., water authorities has performed a maintenance program for distribution system, and in recent years approximately more 50% of network system was replaced in the expansion areas. For partial sub-cities, but in all branches poor maintenance of network and poor man power management for maintenance, it is so difficult to find financial support to renew the water distribution system. Thus, the lack of finance to buy proper materials and poor construction resulted in increased leakage in the system.

3.7.3 Water scheduling

The problem of water scheduling caused by an intermittent supply results in leakage, with a cyclic pressure situation created due to having the supply turned on and off in the corner of the city specially in Nifas silk and Akaki sub-cities, increased levels of leakage are experienced due to stress being inflicted on the pipes causing them to rupture. There is clear irony in this situation as the problem of water scheduling is caused by water shortages. Due to high levels of water loss, a continuous supply is not available resulting in water schedules.

3.7.4 Customer side leakage

Because of the nature of the water storage systems in the country and the generally low rates paid by customers, there is little incentive to conserve water. Consequently, storage tanks and fittings remain unrepaired for long periods there by contributing to significant loss. It is a significant component of water loss and a strain on the delivery of water. And the customer side leakage also more affected the Authority because the water is not used by the customer so the customer is not paid and they complain to the Authority, the office give instruction to the branch and the bill is dismissed, finally the Authority is loss the money and the office are not collect the revenue at the end the month. These types of leakage are mostly happened on the expansion area or in the under constructions area.

3.7.5 Illegal connections

There are a significant number of illegal users of water within distribution system in Addis Ababa city especially in the expansion areas or construction areas. The number of households who do not pay water rates but receive water from its distribution system is not known the Authority. As a consequence, they contribute significantly to apparent losses and revenue loss to the water authority. These connections are often poorly laid just a few inches below the surface and will break easily resulting in real losses taking placed in the form of leakage. Illegal connections are therefore of significant concern of water utilities.

3.8 Water loss Management

This work represents a major step to define the best practice approach for assessing and presenting basic elements of water loss management program in Addis Ababa city, and it will focus on international water loss approach to promote and facilitate the application of water loss recommended methodology of leakage monitoring and pressure management system.

3.9 Leakage Monitoring and control

Leakage management can be classified into two groups including

- passive leakage control and
- Active leakage control.

Passive leakage control is reacting to reported bursts or a drop in pressure, usually reported by customers or noted by the company's own staff while carrying out duties other than leak detection. This method can be justified in areas with plentiful or low cost supplies. Often practiced in less developed supply system where the occurrence of underground leakage is less understood, it is the first step to improvement.

Active leakage control (ALC) is when company staff is deployed to find leaks which have not been reported by customers or other means. The main Active leakage control methods are regular survey and leakage monitoring. Regular survey is a method of starting at one end of the distribution networks and proceeding to the other using one of the following techniques:

- listening for leaks on pipe-work and fittings
- Reading metered flows into temporarily-zoned areas to identify high-volume night flows
- Using clusters of noise loggers (leak localizing)

Leakage monitoring is flow monitoring into zones to measure leakage and to prioritize leak detection activities. This has now become one of the most cost effective activities for leakage management programs.

The most appropriate leakage control strategy will mainly be dictated by the characteristics of the network and local conditions, which may include financial constraints on equipment and other resources. Staffing resources are relevant, as a labor intensive methodology may be suitable if manpower is plentiful and cheap. If the geology of the area allows a high proportion of leaks to appear at the surface, a strategy of regular survey followed by rapid repair may be adequate. If some leaks fail to appear at the surface, then, a more intensive strategy of leakage monitoring is required.

The main factor governing choice, however, is the value of the water, which determines whether a particular methodology is economic for the savings achieved. A low activity method, such as repair of visible leaks only, may be cost-effective in supply areas where water is plentiful and cheap to produce. On the other hand, countries which have a high cost of production and supply, Lather factories, Airport etc. can justify a much higher level of activity, like continual flow monitoring, or even telemetry systems, to warn of a burst or leakage occurring [Farley and Trow, 2003].

3.10 Economical Dimension of Water Loss

In Addis Ababa the water loss rate was varied between 36.94 to 43.15% of the production in the last five years. If it is assumed that unit water production cost is half price of selling, total cost of water loss for Addis Ababa city will be 127,550,769.60 Birr for 2009 year provided that water loss rate is reduced from 43.15% to 36.94%. Average water selling price is obtained 3.80 Birr per cubic meter.

The marginal cost producing one additional unit volume of water as the level of leakage is reduced the cost of water saved is first benefit. Redaction in leakage will produce a similar redaction in future projection of water supply requirements.

3.11 Summary

This study presented the current water loss perspectives of Addis Ababa city. Further, its aim is to offer two IWA approach to control water loss and leakages through water distribution networks. Application of these two methods to distribution systems will result in better knowledge of the components of uncontrolled flow rate, the technical losses (real losses) and nontechnical losses (apparent losses) including demands related to metering errors on the system. For the future, application of these two IWA methods has the potential for reducing water loss within entire of water distribution systems for Addis Ababa city. This paper has attempted to put forward the current situation of water loss in Addis Ababa. Besides, it proposes appropriate solutions for the reduction and control of water loss. It is hoped that it will be a catalyst for increased and enhanced awareness and implementation of water loss solutions in the country. The major cause of these leakage are listed as follows

- internal Corrosion of galvanized pipe
- Shallow layering of service pipelines resulting in damage by vehicles and erosion
- External corrosion of pipes due to aggressive soil conditions
- Unnecessarily high pressure in some area resulting in damage in pipe

The results of NRW reduction by Water Balance Method show the difference between predicted and actual water losses in water distribution network. The results also shows that after leakage reduction control works took place, the volume of water loss in water distribution network has reduce about 39% for pilot area.

4 Water Supply Coverage

4.1 Introduction

Problem in provision of adequate water supply to the rapidly growing urban population are increasing dramatically. Water demand in the domestic sector of developing cities including Addis Ababa increases through time that as a result demand for additional water sources and infrastructure. Financial constrain is one of the major factors for the low water coverage of the water supply but poor management of the existing water supply also has a great impact for the low coverage. Beside to the overall low supply coverage, supply disparity existing among different localities. Therefore evaluating the city distribution of the water supply is important in order to identify the problematic areas and intervene accordingly.

Water supply coverage is usually evaluated based on the quality, quantity, paying capacity of the people, distance, etc. but the intention of this research is not to evaluate all those but related to the quantity of the supply and level of connection that are related to the water loss. In this part of the analysis, the number of domestic connection per family and the average daily per capital consumption is used to analysis the domestic water supply coverage for the city. The level of coverage has been also compared with other cities of developing countries. Beside to the statistical analysis for the city, the distribution of the average daily per capital consumption and connection per family has been evaluated.

4.2 Domestic water supply coverage

The water supply coverage of the city has been evaluated based on the average per capital consumption and level of connection per family. The average per capital consumption has been derived from the yearly consumption of each Weredas that has been aggregated from the individual domestic water meters. Beside to the average per capital water consumption, the distribution number of domestic's connection per family has been also evaluated. Statistical analysis was used to evaluate the supply coverage for the city and supply coverage map has been prepared for the city. Number of population as forecasted to the year 2010 has been used to evaluate the average per capital consumption.

4.3 Level of Domestic Water Supply Coverage

Access to water supply may be evaluated using the amount of water consumed and the level of connection. For evaluating the amount of water consumption, the annual water consumption is converted to average daily per capital consumption using the population data of the city. The number of domestic connection per family has been also used for analyzing the level of connection as elaborated below.

Table 4.1 Domestic Water Supply Coverage

year	Domestic water consumption (m^3)	Total population	consumption m^3/person/year	consumption l/person/day	% of coverage based on the Authority data
2005	45,520,623	2,641,653	17.23	47.21	52.46
2006	51,338,591	2,687,593	19.10	52.33	58.15
2007	53,272,445	2,733,533	19.49	53.39	59.33
2008	53,676,740	2,779,474	19.31	52.90	58.79
2009	58,140,949	2,917,295	19.93	54.60	60.67

4.4 Average Daily per Capital Consumption

The level of water consumed for domestic purpose has been aggregated to all sub-cities of the city so as to analysis the distribution of the water coverage among different localities. Statistical analysis was used to evaluate the distribution of the supply coverage in all weredas of the city while a supply coverage map is prepared of the city. Evaluating the domestic water supply coverage using volume of consumption may not allow realizing the distribution comparison among the weredas. For this reason the annual consumption data has been converted to average daily per capital consumption using the number of population. The average daily per capital consumption of each weredas was derived using the following expressions.

$$\text{Capital consumption (l/person/day)} = \frac{\text{Annual consumption } (m^3) * 1000 l/m^3}{\text{Population number of each weredas} * 365}$$

Table 4.2 Water production and consumption of Addis Ababa city

year	Total production (m^3)	Total Billed data (m^3)	Total population	Water Production l/person/day	consumption l/person/day
2005	80,073,546.00	45,520,623	2,641,653	83.05	47.21
2006	82,820,272.00	51,338,591	2,687,593	84.43	52.33
2007	86,278,592.00	53,272,445	2,733,533	86.47	53.39
2008	88,405,625.00	53,676,740	2,779,474	87.14	52.91
2009	92,201,273.00	58,140,949	2,917,295	86.59	54.60

The distribution of the domestic water coverage has been evaluated using the above statistical tools. The distribution of the production has been first reviewed using the descriptive statics.

Taking the mean production as shown in above the average domestic water coverage of the city is found to be 86.59 l/per/day and the consumption of the city's are 54.60 l/per/day in 2009. The average daily per capital production of the city is low b/c from this production more than 37% are loss before reach to costumer, while the city production compared to other cities even in developing countries like with the southern Africa larger cities. An overview of urban water supply for the southern Africa is shown in the table 4.3.

Table 4.3 African capital cities of water consumption

Country	Largest city	Population of largest city (million)	Water production for the largest city (l/person/day)
Angola	Luanda	4	30
Botswana	Gaborone	0.13	286
Democratic republic of Congo	Kinshasa	5.7	86
Lesotho	Maseru	0.27	81
Maturities	Port Louis	0.15	200
Mozambique	Maputo	0.97	133
Namibia	Windhoek	0.27	
Tanzania	Dar Es salaam	3	150
Zambia	Lusaka	1.21	225
Zimbabwe	Harare	2.38	156

(Source: Wallingford, 2003)

4.5 Evaluating of the distribution of the water supply coverage

As clarified earlier the water supply coverage of the city, both in quantity and level of connection is low while compared to the other cities. In this case section the spatial distribution of the consumption in relation to number of population is discussed. In areas where water supply coverage is sufficient, volume of domestics water consumption is expected to be linear related to the level connection. Areas having better level of connection are expected to consume more water as they can easily get it within their building or compound. A detail demand study in Africa found that average water carried was about 22 l/day per capital over a long distance rising to about 30 l/day per capital where water was obtained from the consumer own stand pipe. Of course distance is not a big problem in urban areas rather than rural areas (ADB, 1993) on the other hand in areas having insufficient supply like Addis Ababa, some areas may have better level of connection but may not necessarily mean they are consuming more volume of water as the possibility of getting the water does not depend only on the location. There are number of places that get low volume of water due to their topographic location. As the city mainly uses gravitational supply system, topography has a great impact on the per capital consumption. A map of average daily per capital consumption of distribution was prepared for the city see in the next page.

Figure 4.1 Water supply coverage of Addis Ababa city

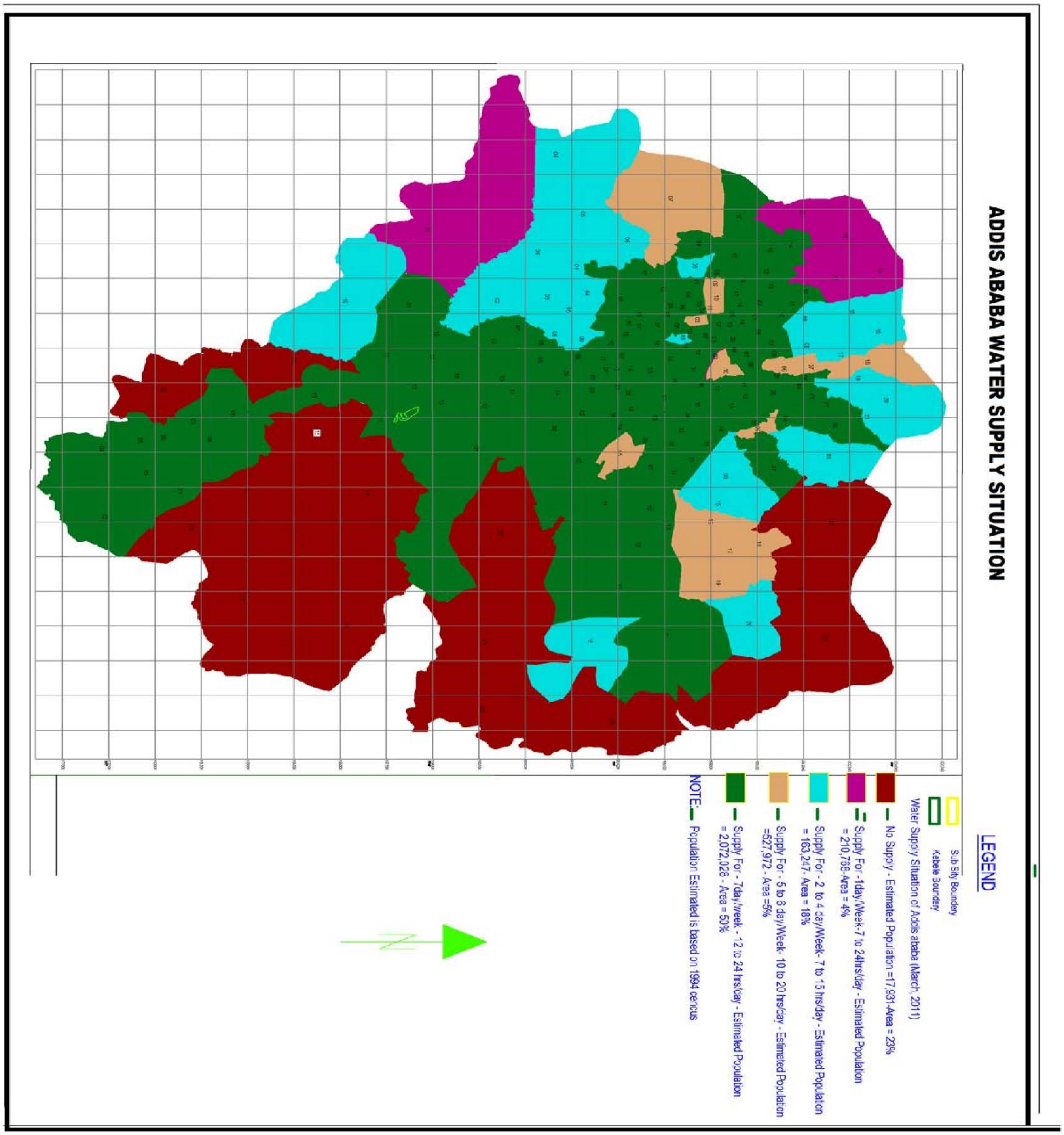

Figure 4.1 Water supply coverage of Addis Ababa city

4.6 Level of connection per family

Level of water connection is an important element on the one hand for evaluating the level of water coverage that will be the focus of this section and on the other hand it has a direct impact on the water loss that will be detail separately.

The total numbers of connection or water meter within the city are about 297,500 that among those, 260,000 are for domestic use. In order to compare the distribution of the water connection among the different sub-cites, the total numbers of connection per weredas are converted to connection per family using the population data of each sub-cites. According to the census of the 2010, average family size of 5.5 is used for calculating the average number of connection per family using the following expression.

$$\text{Connection per famil} = \frac{\mathit{Total}\ \text{number of connection of the city}}{(\mathit{Number\ of\ population\ the\ city}/\mathit{Averagefamilysize})}$$

Similar to the per capital consumption, the distribution of the connection of the connection per family has been evaluated.

Table 4.4 Level of Connection

year	Total Population	Average family size	Total number of connection	level of connection
2005	2,641,653	5.5	215,287	0.45
2006	2,687,593	5.5	234,209	0.48
2007	2,733,533	5.5	257,339	0.52
2008	2,779,474	5.5	279,704	0.55
2009	2,917,295	5.5	272,005	0.51

4.7 Summary

In order to get a realistic average daily per capital consumption of the city of Addis it was necessary to exclude both higher and lower extreme values both for per capital consumption and connection per family. After excluding the outlier the average per capital consumption of the city is found to be 54.60 l/day this average per capital consumption is lower while compared with other developing countries like the southern Africa cities.

5 Distribution System Modeling

5.1 Introduction

As a result of rapid population growth and high water losses from the distribution network, the total water demand of the system in Addis Ababa exceeds available production capacity. To limit total demand and provide an equitable distribution of available water supplies with reduced system pressures are often introduced. The demand for water is not based on the notions of diurnal variations of demand but on the maximum quantity of water that can be collected during supply hours. This will be dependent only on the available pressure heads in the network.

The objective of this modeling was not to predict the exact time at which different users get water but to develop a simplified model, node demand is dependent on the pressure at the junction nodes to reduce the water loss and maximize the flow rate at the tap.

The components and processes of a water distribution system including hydraulic and water quality concepts are described in this section.

a. Definitions

A water distribution system is principally made of links and nodes. Links are pipe sections which can contain valves and bends. The nodes can be categorized as junction nodes, which join pipes and are the points of input or output of flow, and fixed-grade nodes such as tanks and reservoirs with fixed pressure and elevation.

As defined in water distribution system models, reservoirs are nodes that represent infinite sources or sinks of water, such as lakes. Tanks are nodes with fixed storage capacity and varying volumes during distribution. Pumps are devices that impart energy to water thereby increasing its head. Valves limit the pressure or flow at points in the system. These components are illustrated in the figure 2.1

A loop is a sub-component of a distribution system: it consists of an entity made of nodes all connected through links. It is an important component of a model because mass in and out of a loop can be accounted for and used to solve for flows.

b. Hydraulics Modeling

The two fundamental concepts of distribution network hydraulics are conservation of mass and energy. For energy, the Bernoulli equation states that the sum of the elevation, pressure and velocity heads between two points must be constant. Due to losses because of friction during flow through the pipe, this equation does not hold precisely in practice. Frictional head loss is accounted for with head loss factors typically based on the Hazen-Williams, Chezy-Manning or Darcy-Weisbach equations.

Head loss can be described as $h_l = Aq^B$

Where A is the resistance coefficient, B is the flow exponent and q is the flow rate. Table 5.1 illustrates the different formulas used to account for head losses. Notice that each of them contains a friction or roughness coefficient.

Table 5.1 Pipe head loss formulas for full flow

Formula	*Resistance Coefficient (A)*	*Flow Exponent (B)*
Hazen-Williams	$4.727\ C^{-1.852}\ d^{-4.871}\ L$	1.852
Darcy-Weisbach	$0.0252\ f(\varepsilon,d,q)d^{-5}L$	2
Chezy-Manning	$4.66\ n^2\ d^{-5.33}\ L$	2

Notes: C = Hazen-Williams roughness coefficient
ε = Darcy-Weisbach roughness coefficient (ft)
f = friction factor (dependent on ε, d, and q)
n = Manning roughness coefficient
d = pipe diameter (ft)
L = pipe length (ft)
q = flow rate (cfs)

Conservation of mass dictates that flows are equal for pipes in series and they are summed for pipes in parallel. Head losses, on the other hand, are summed for pipes in series and assumed equal at nodes that join pipes in parallel.

Equations can be categorized into loop and node equations. Among loop equations, mass continuity takes the form of

$\Sigma Q_{in} - \Sigma Q_{out} = Q_e$-- *5.1*

Where Qin is the inflow, Qout is the outflow and Qe is the external flow into or out of the system at each node. Energy conservation is written as

$\Delta E = \Sigma hl - \Sigma ep$---*5.2*

Where ΔE is the difference in energy grade, hl is losses considering pipe length, diameter, roughness and minor losses, and ep is the pump head. Node equations expand the mass continuity equations to express discharge in terms of head difference between nodes a and b (Ha-Hb) and resistance of the pipeline (Kab):

$Q_{ab} = [(H_a - H_b)/K_{ab}]^{1/n}$--*5.3*

Where:-

n is selected depending on the head loss equation (Viessman and Hammer, 1998).

The single path adjustment method described by Hardy-Cross is best known for solving loop equations. It consists in making an initial guess of flow rates that satisfy continuity at each node, followed by computing flow correction factors for each loop to satisfy the energy equation. Through iteration, improved solutions are found until the average correction factor is within acceptable limits.

The most widely used algorithm for solving node equations is the single-node adjustment method also described by Hardy-Cross. A grade is assumed for each junction node, followed by computation of a grade adjustment factor to satisfy continuity and through iteration improved solutions are found until a specified convergence criterion is met (Viessman and Hammer, 1998).

5.2 EPANET

EPANET is a computer program that simulates hydraulic and water quality behavior within pressurized pipe networks. It was developed for the Environmental Protection Agency, and presents the great advantage of being available on the internet free of charge. It can model networks of pipes, nodes and reservoirs and tracks the flow of water in each pipe, the pressure at each node. As an illustration,

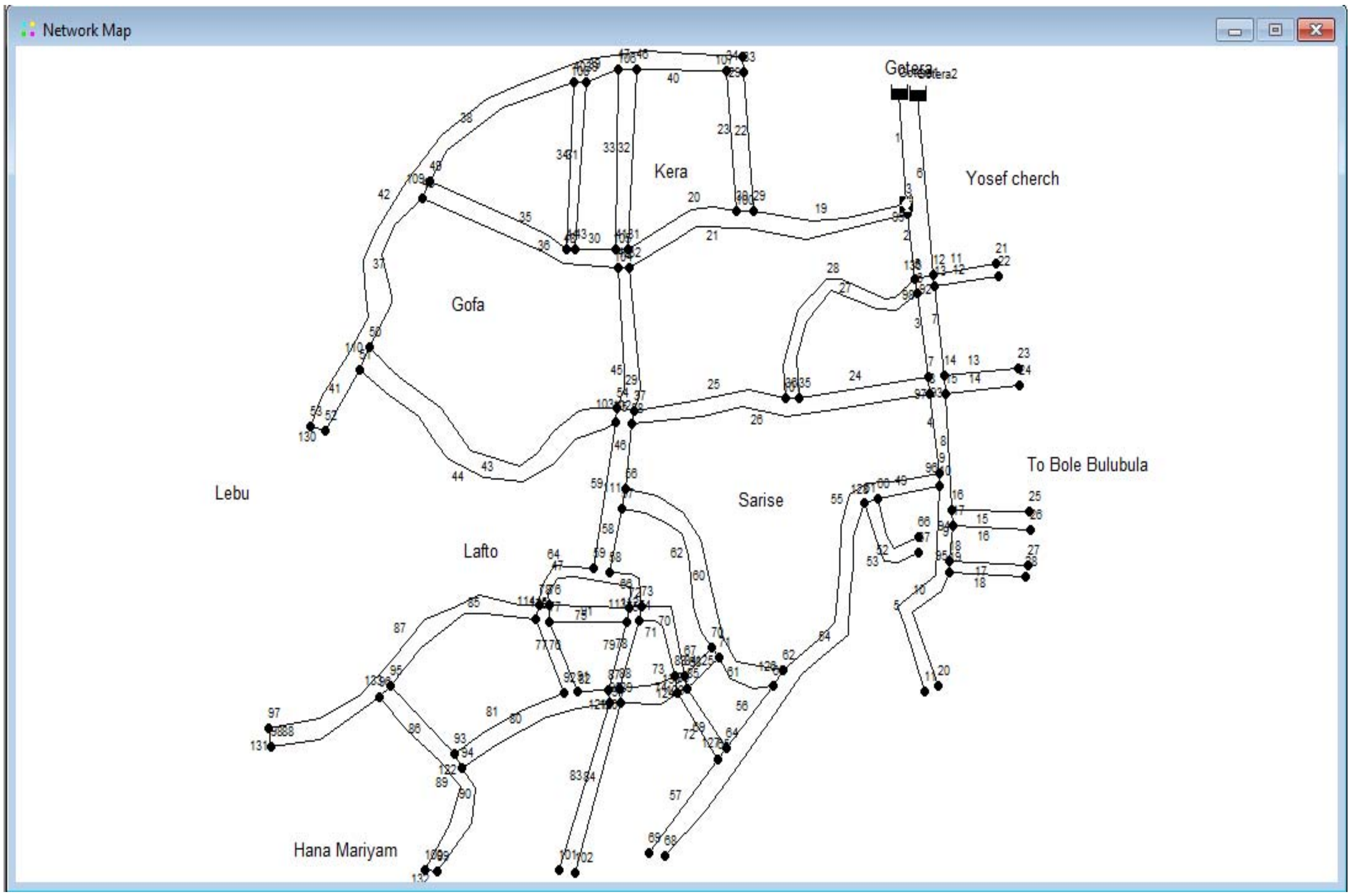

Figure 5.1 Modeling of the sub-system

Some of the key hydraulic capabilities of EPANET include no size limitations on the network, time-varying demand or controls (e.g. opening and closing valves), simulating a pressure-driven node using the concept of emitter coefficient as discussed later, handling multiple head-loss equations, and pump operation control (e.g. based on tank water levels). The program also sends warning messages which point out changes in the system such as negative pressures occurring in the system. No model can perfectly reflect the underlying system but these capabilities enhance the realism of the simulation (Rossman, 2000). An important scenario that often occurs in situations of crisis is that of intermittent flow. EPANET assumes a constantly pressurized system, with instantaneously full pipes at the start of distribution. Hence, relatively significant discrepancies could arise between the model and the actual dynamics of the system and these are discussed in the following section.

5.3 Data Assembly

The following section describes the process of putting together the water distribution model in EPANET from the some raw data collected in the field. The first step in starting the model is to set up some important parameters which define the input values used by the software.

5.3.1 Initial Setup

Throughout the process, International System Units (SI units) have been used. To request the use of these units in EPANET, the user chooses SI flow unit under the hydraulics option. I have selected liters per second for the model, which also defines all other units using the SI system. Hence lengths, pressures, head, elevations are taken in meters and Diameters of pipes are defined as millimeters. The Hazen-Williams equation was chosen for determining head-loss. Further setup of the interface for analysis is discussed in the next section.

5.3.1.1 Tanks

The tanks are further defined in EPANET under the tanks section. Tanks are assumed cylindrical in EPANET: hence their height should be kept but an equivalent diameter needs to be calculated from each tank's volume. After copying the tank's ID into the section, their elevation is inputted. Their initial water level is then taken to be their height (assuming full tanks); such height was taken from the overflow to the outflow pipes, the useful height of the tank, and not from the total height. The minimum level and volume is taken to be zero and the maximum level is the same as the initial one. No volume curve is chosen since all of the tank storage volume varies linearly with height.

5.3.1.2 Pipes

In EPANET, pipes are links connecting two nodes. Inputting pipes can be done using the text or the graphical interface of EPANET. It was recommended using the graphical interface with the GIS map to verify the overall logic of the system by its location. Also, throughout the process of inputting pipes, lengths surveyed were verified against computed lengths to make sure the field lengths were correctly recorded from the GIS plan of Addis Ababa road network. Besides, direct measurements take into account the slope whereas the length feature only provides the planar distance between two points.

As a side note, it should be noted that to write data from an Excel file into an EPANET input data file, it is necessary to first save the Excel file to text file and to then copy and paste the content of the text file into an input file exported from EPANET.

Pipe length and diameters are then inputted as well as roughness. A roughness of 150 was selected for the polyvinyl chloride (PVC) pipes (Briere, 1999). The pipes are fairly new but considering the stagnation periods due to the intermittent nature of the system, build-up should have made roughness rise to such levels. Minor losses are also included as part of pipes, not at the tee or elbow junctions. These minor losses were ignored and assigned a null value because errors in elevation were assumed to outweigh the minor losses sufficiently that results would not significantly vary by considering such losses.

As to the status of pipes, they were generally left with their default status of open. These were assigned the status of control valve, letting water flow only in the direction of their first to their second node. This setting is important to avoid backwards flow from tanks or reservoirs whose inlets discharge above surface water or for emitter taps which would become sources in the absence of flow.

5.4 Setup

As mentioned earlier, it is important to define the parameters on which the model is based.

5.4.1 Hydraulics and Time Parameters

Under hydraulics options, SI units of measurement are chosen and the Hazen-Williams equation used to find head-loss. The specific gravity of water is set to 0.998, for average temperature of 25 degrees Celsius, and a relative viscosity of 1.The emitter coefficient is 0.52 and the bulk reaction coefficient is 0. The hydraulics accuracy represents the threshold ratio of variation in total flow from one iteration to the next over total flow in the system. If this ratio is under the specified accuracy, iterations for solving the model stop. A lower value for this accuracy would give more precise results. A lower accuracy would also increase the solution time for the model, and possibly prevent the model from converging on a solution. The value of accuracy is often tweaked in the case of non convergence. This represents a reasonable accuracy considering the magnitude of error in the surveyed elevations. It is also preferable to set a large maximum number of trials (10,000 for example) and to select the option of continuing if the model is unbalanced (instead of stopping), since the analysis can be cancelled manually if it takes too long. The quality tolerance is another important parameter. It represents the smallest change in water quality that will cause a new parcel of water to be created. Typically, this accuracy is set at or below the detection limit.

Besides hydraulics setup, time characteristics need to be set up to properly use EPANET. The duration of the simulation is one such time setting. Other parameters include the hydraulic time steps as well as time pattern and reporting time steps. A time pattern can be associated with different parameters such as valve controls and more. The duration of these time increments determine the time resolution for analysis, control setting and reporting. Hydraulics usually vary less throughout the distribution period than quality does and this is why the hydraulics time step is often set to be longer than the water quality time step to reduce computing time. Considering the subtleties of the system with its opening and closing of valves.

Based on the above information the pressure was taken from the above expression, the water loss analysis that means from the leakage and pressure relationship and we use in the model most of the nodes are less pressure from the previous.

5.5 Result

The work discussed so far has led to the construction of a computer model designed to serve as a tool for analysis and improvement of the drinking water supply system in Gotera, Addis Ababa. The following chapter gives a brief overview of the system and evaluates the success of this model in predicting different parameters associated with the water supply. It then provides an analysis of potential improvements of the system.

The skeleton of the system is shown in Figure 5-1. It is made of 136 pipes and 102 junctions. The different subsystems within the overall distribution system are shown. Reservoir Gotera 1 and Reservoir Gotera 2 are sub-components of the sub-system since reservoir can supply water to system.

EPANET can show pressures, demand, and water quality at different nodes as well as flows, velocities and head loss in pipes throughout the distribution period. Such results can be exported to tables and graphics or visualized on the graphical interface as illustrated in Figure. Direction of flow is shown by arrows on Figure 4.3. The figure shows large flows (red arrows) going into the system which then re-distributes water to taps. To the very left of the illustration, flow is coming in the opposite direction but a valve blocks it from entering the pipes supplied by the Gotera Reservoirs Many other analysis tools are available in EPANET, such as drawing contour plots of the region based on a parameter of choice, or time series plots of specific nodes.

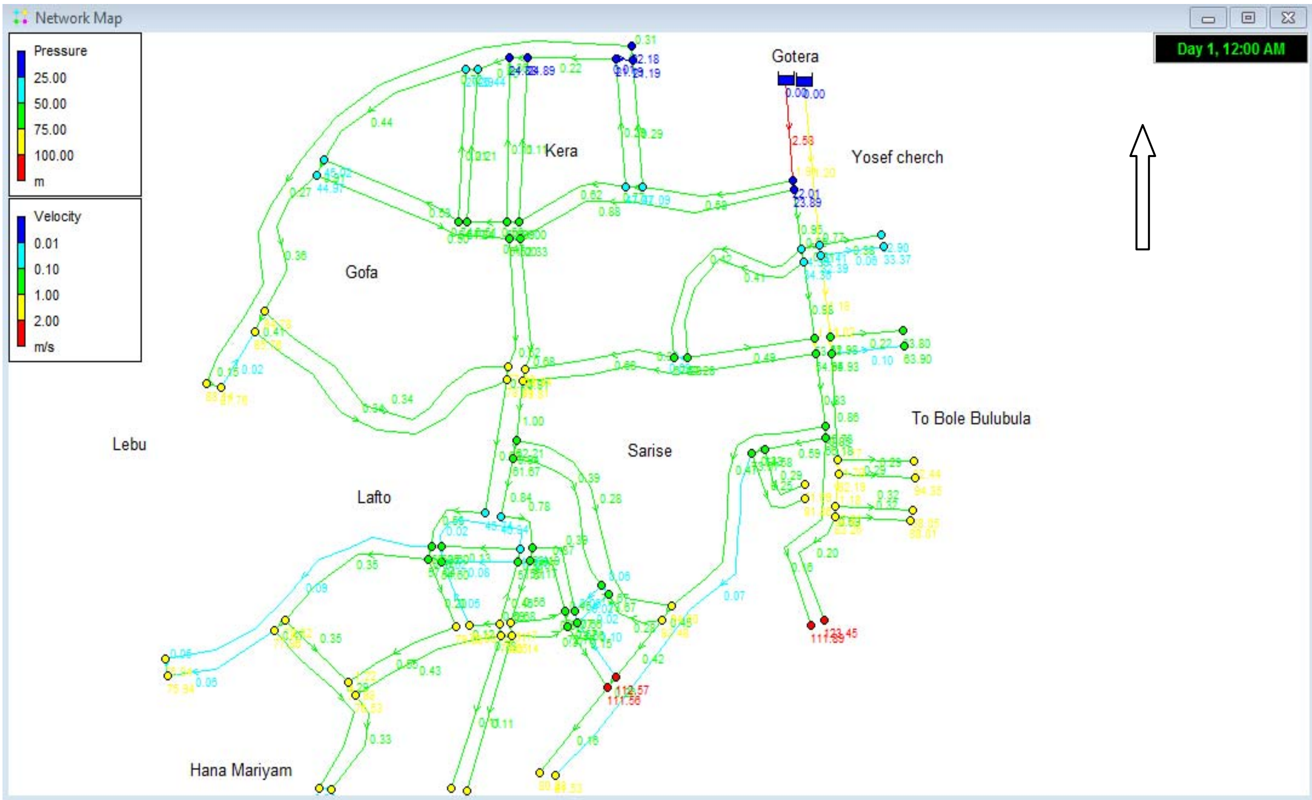

Figure 5.2 Illustration of predicted pressure and flows by the EPANET graphical interface

5.5.1 Pressure

Figure 5.2 is an illustration of the pressure distribution throughout the sub-system. The orientation of the sub-system is the same as in Figure 5.1 and all subsequent figures of the sub-system will follow the same orientation. North is upward of the image while the road is adjacent to the right edge and the left edge in the image. The pressure distribution is extrapolated by EPANET to areas without pipes but these areas should not distract the viewer. It should be noted that the pressure distribution is not an illustration of pressures at taps only; it includes the pressure at all different nodes in the system.

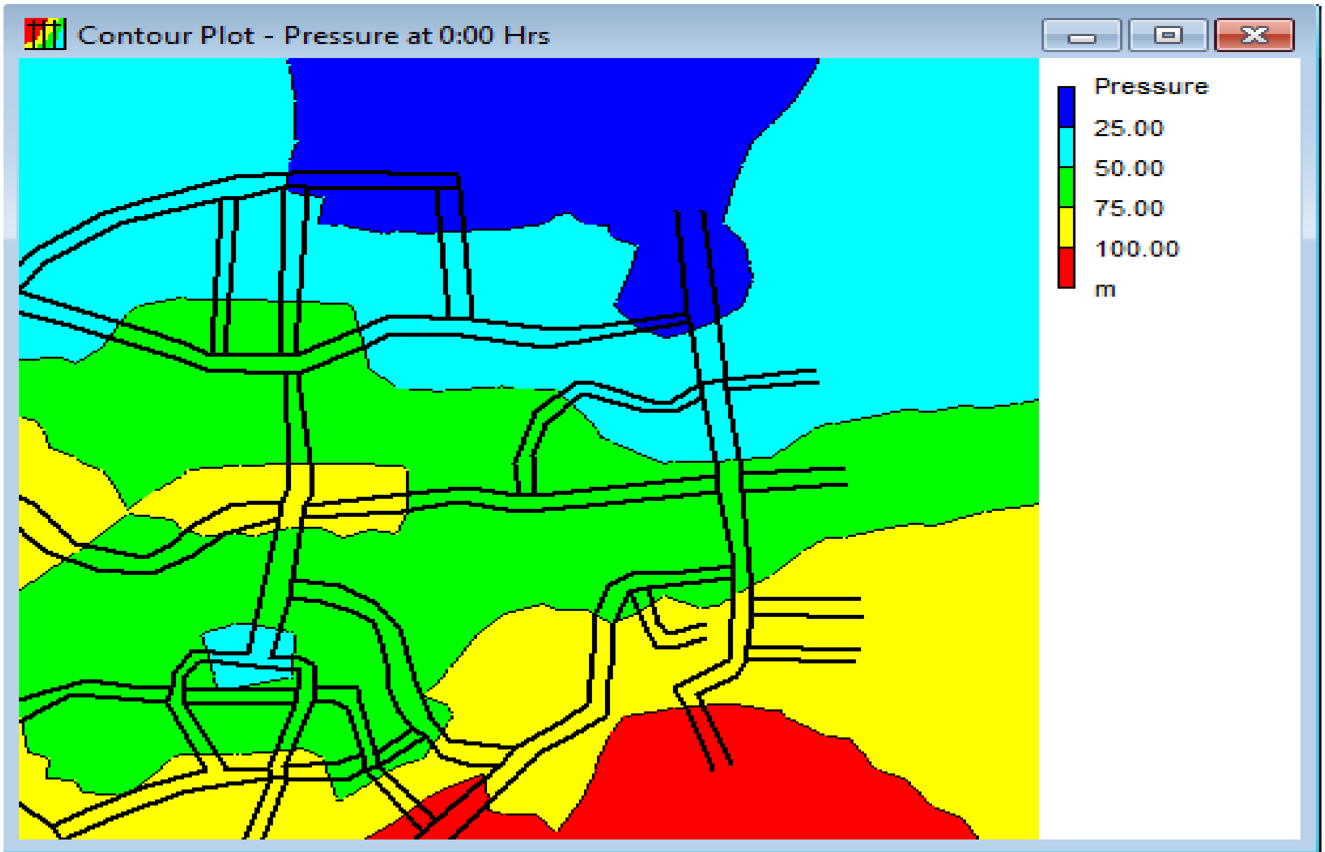

Fig 5.3 Geographical distribution of pressure in sub-system

There are extremes of low and high pressure throughout the system mainly due to the topography of the area and the elevation of the distribution reservoirs. Most of the Gotera reservoir 1 is subsystem in particular is marked by low pressure in the pipes bordering the road. Most of the Gotera reservoir 2 is subsystem could make use of higher pressures as well. On the other hand, the two reservoirs are reservoir 1 and reservoir 2 subsystems are marked by high pressures even though they also possess some low pressure areas. The objective is to convey some of the high pressure to low pressure areas.

Figure 5.3 provides a geographical analysis of the pressure distribution in the sub-system whereas Figure 5.4 provides more quantitative information through a frequency plot of the pressure at nodes throughout the sub system, About 50 percent of the sub-system is marked by pressure between 50 to 75 meters, each tap added to the pipe system in the sub-system will produce about 140 liters per day for each domestic costumer and for non domestic and fountain based on consumer. The pressure is only important as it creates the flow to provide water to the general sub-system population. Figure 5.4 shows the geographical distribution of pressure around Gotera.

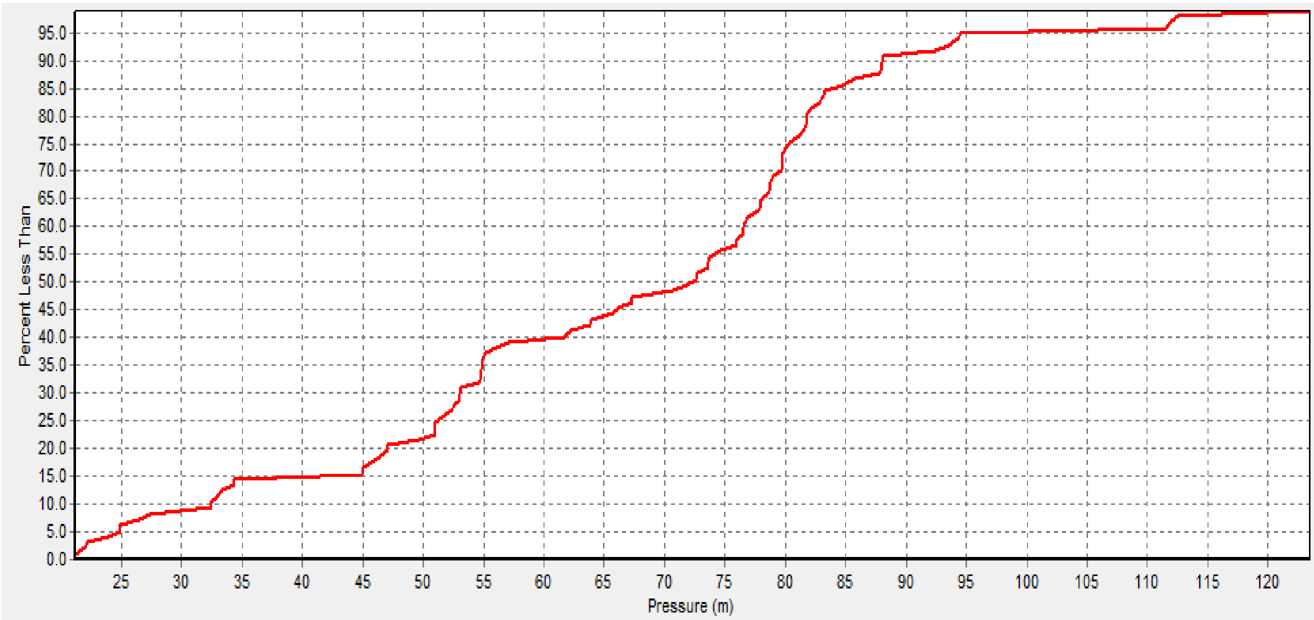

Figure 5.4 Numerical distribution of pressure in sub-system

Comparing pressure (Figure 5.3) and flow distribution (Figure 5.5) shows that elevated flows are located in more concentrated areas than pressure; these areas are centered on the public taps. However, some extremes of the pipe system do not end in a concentrated demand area: this is either due to a flow less than 5 liters per second, or simply because these taps have an emitter coefficient of zero because they were used minimally (e.g. latrines). These are usually nonpublic taps such as those for Non domestic, schools or latrine and private taps or drums. Unfortunately, there are some high flow public taps not represented on the figure: some taps are calculated to have low or no flow.

As far as distribution of water, it seems that the northern part of the sub-system has better access to water than the southern part. This might be because the northern part is more densely populated but people also live in the southern region and they should be provided drinkable water at a reasonable distance.

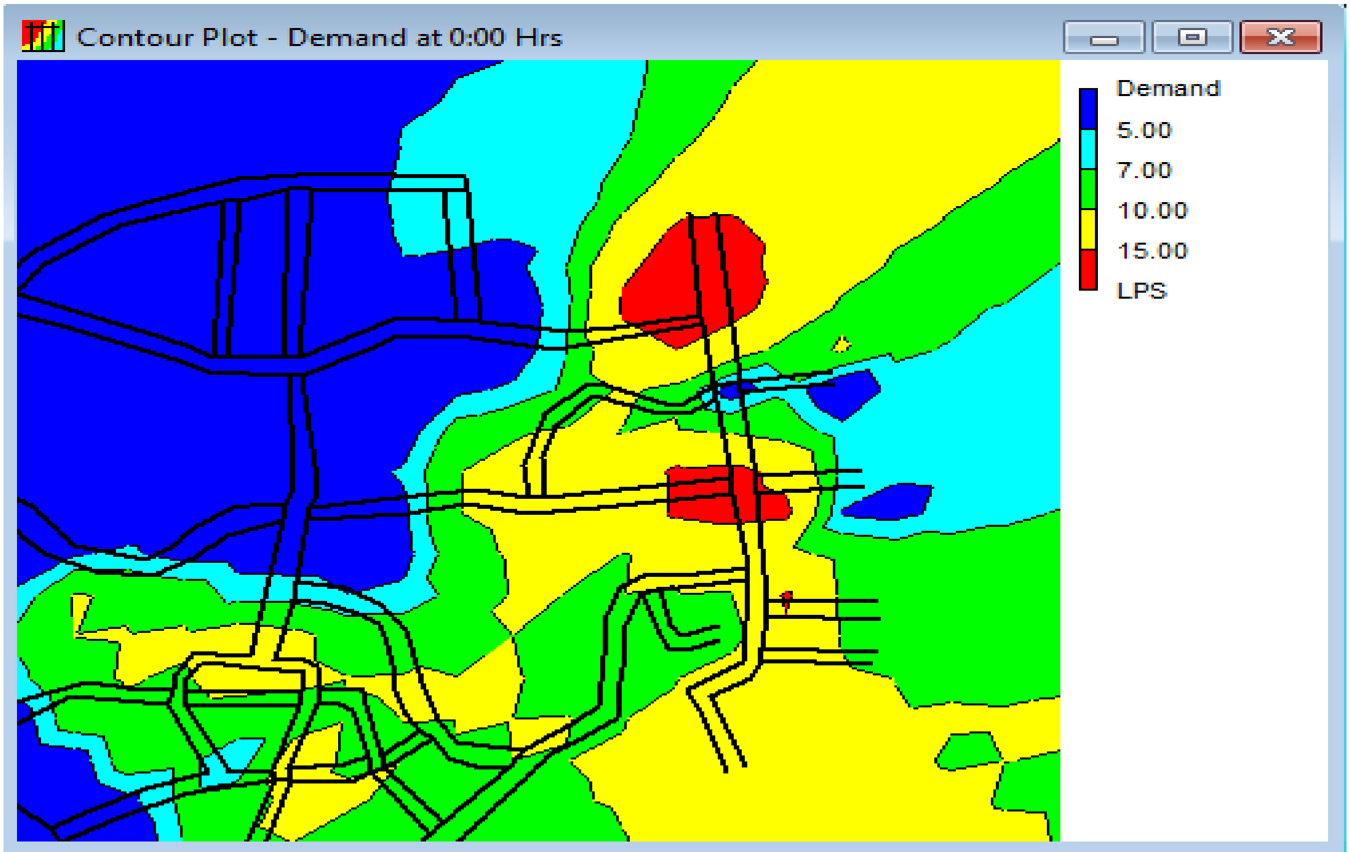

Figure 5.5 Geographical distribution of flow in sub-system

5.5.2 Demand

As far as distribution of water, it seems that the southern part of the sub-system has better access to water than the northern part. This might be because the northern part is more densely populated but people also live in the southern region and they should be provided drinkable water at a reasonable distance.

Figure 5.6 provides a more quantitative analysis of the demand throughout the sub-system. The distribution of flows in Figure 5.6 only takes into account taps with non-zero flows. This is close to the median value of 140 liters per day flow from a single tap which was calculated from the pressure distribution. The median flow from Figure 5.6 is a bit higher because the system includes many tap stands with more than one valve. Tap stands with more than one tap have a reduced pressure at each tap but can generally discharge more than a single valve tap stand because of the non-linear relationship between flows and pressure.

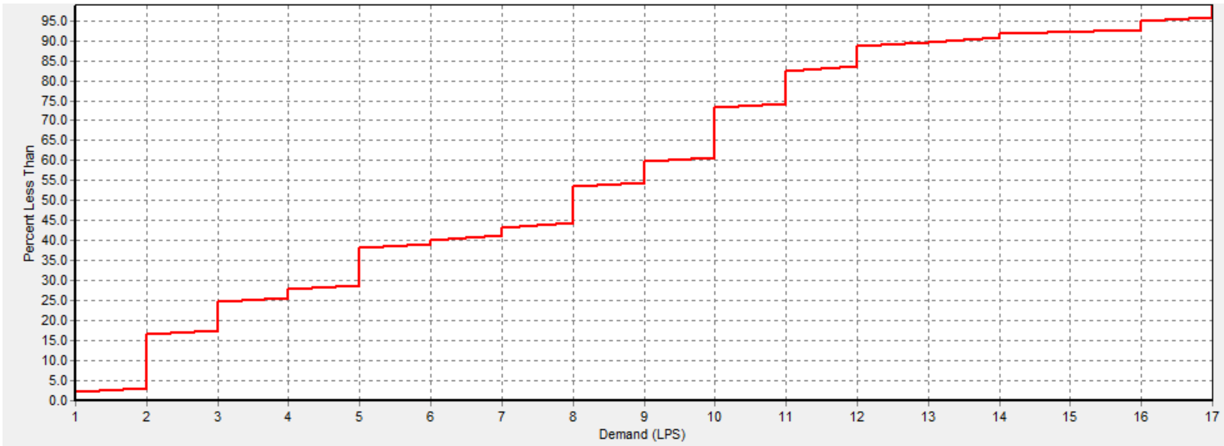

Figure 5.6 Numerical distribution of flow in sub-system

The lower flow portions of this distribution represent the tap stands of concern with flows that need to be increased. Currently, about 15 percent of the taps provide flows of 21600 liters per day, 34 percent are at flows of less than 14400 liters per day, 40 percent at 7200 liters per days and 11 percent 25200 liters per day.

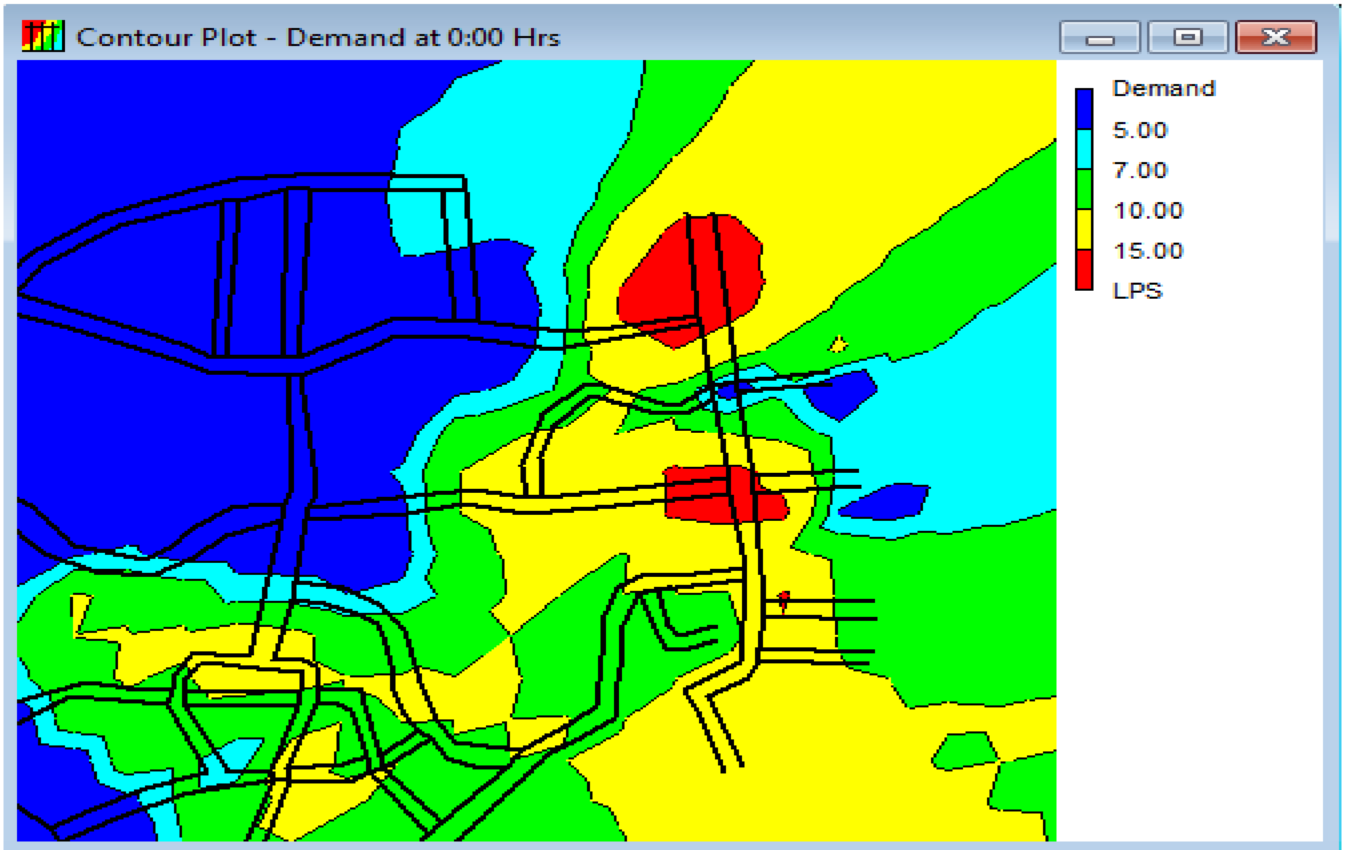

Figure 5.7 Geographical distribution of demand in sub-system

5.6 Summary

The purpose of this work was to develop a model that would represent the water distribution system in Gotera site. It would serve as an analysis tool to increase understanding of the complexities of the system and to plan improvements.

The distribution system model was then used to evaluate three alternative scenarios to improve system performance. The objective of the first and second scenario was to increase the flow rate at taps of low supply; the third scenario aimed at adding taps to parts of the sub-system without easy access to running water. The first scenario consisted in opening valves to connect subsystems: it increased the flow rate at taps of large supply more so than at taps of low supply. This scenario was not recommended because it would quickly drain parts of the water supply. The second scenario consisted of adding connecting pipes between subsystems of high pressure and those of low pressure. It was recommended because it would increase the flow rate of low- and medium-supply taps.

6 CONCLUSION AND RECOMENDATIION

Both the average water supply coverage and the city distribution were evaluated based on the daily per capital consumption and level of connection using the population data of the city. The average water supply coverage of the city is found to be 86.59 liter/person/day. This average per capital consumption is lower compare with other developing cities like the southern of Africa.

Despite the low water coverage of the city, the total water loss is found to be high, the total water loss was computed by subtracting the consumption (bill data) from the water supplied is 34,060,524 m^3 that means 39% of the production water in 2009 at the city level,

In the sub-system after collect the meter reading both the customer and Bulk meter for five times the total loss has 58310 m^3, out of the total supply to the system 155215 m^3 and the loss is 37.56 % of the total supplied to the sub-system. The approaches unaccounted for water expressed as a percentage and loss as per connection. The total water loss express as percentage is an important tool than water loss as per number of connection. In the city the loss of water as per number of connection, taking the total number of connection in the city as 297,500 the water loss per connection for the similar duration was derived as; Water loss is 313.67 liter/connection/day.
From the water loss analysis of the sub-system, higher water loss has been found in around Gotera, Meskel flower.

The other issue addressed in the analysis was the major factor contributing to high levels of water loss in Addis Ababa is

1) Age of pipe network
2) Poor maintenance of networks
3) Water scheduling
4) Customer side leakage
5) Illegal connections

This paper has attempted to put forward the current situation of water loss in Addis Ababa. Besides, it proposes appropriate solutions for the reduction and control of water loss. It is hoped that it will be a catalyst for increased and enhanced awareness and implementation of water loss solutions in the country.

The water supply coverage of the city, supply 7 days a week for 12-24 hours/day 50% of the total Addis Ababa area and its consumed full nearly to 2,072,028 population out of the total population of the Addis Ababa city, supply 5-6 days a week 10 to 20 hours/day is only 5% of the total area and consumed estimated population 527,972 people, supply for 2 to 4 days a week for 15 hour/day are 18% and estimated population are 163,247 people, supply for 1 day a week for 7 to 24 hours/day are 4% from the total area of Addis Ababa served for 210,768 people and 23% of the total area is no supply and estimated population 17,931 are without service.

The purpose of this work was to develop a model that would represent the water distribution system in Gotera site. It would serve as an analysis tool to increase understanding of the complexities of the system and to plan improvements.

The distribution system model was then used to evaluate three alternative scenarios to improve system performance. The objective of the first and second scenario was to increase the flow rate at taps of low supply; the third scenario aimed at adding taps to parts of the sub-system without easy access to running water. The first scenario consisted in opening valves to connect subsystems: it increased the flow rate at taps of large supply more so than at taps of low supply. This scenario was not recommended because it would quickly drain parts of the water supply. The second scenario consisted of adding connecting pipes between subsystems of high pressure and those of low pressure. It was recommended because it would increase the flow rate of low- and medium-supply taps. For the third scenario, these new taps were successful in providing water to these areas without significantly affecting the rest of the system. An additional recommendation for increasing the water supply in the sub-system was found from analyzing reservoirs:

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Appendix A: Input parameter

Annual production of Addis Ababa water supply in (m^3)

year	Legedadi	Gefersa	Akaki well	other Well	Total production
1994	45844675	9374150			55,218,825
1995	47186039	9565064		1189126	57,940,229
1996	45930020	9466714		958423	56,355,157
1997	47591408	8668216		2184111	58,443,735
1998	43737558	8371040		1758190	53,866,788
1999	53134069	8297900	312933	2261461	64,006,363
2000					65,793,897
2001	52246317	7917757	387620	2476480	63,028,174
2002	52716452	8244936	5424830	2418970	68,805,188
2003	50973399	7972714	6590432	2001084	67,537,629
2004	53591559	8607688	12023963	1596948	75,820,158
2005	57081868	8315611	12939921	1736146	80,073,546
2006	59514199	8138448	13363547	1804078	82,820,272
2007	60038338	8467056	14977954	2795244	86,278,592
2008	60144865	7456619	16995893	3808248	88,405,625
2009	60475493	8132038	16734281	6859461	92,201,273
2010	60321250	10569909	18324278	9391896	98,607,333

Annual Billed data of Addis Ababa water Supply in (m^3)

year	Domestic	Non Domestic	Total Billed data
2006	21,982,984	23,537,639	45,520,623
2007	23,587,590	27,751,001	51,338,591
2008	24,827,437	28,445,008	53,272,445
2009	26,230,408	27,446,332	53,676,740
2010	29,910,592	28,230,357	58,140,949

Appendix B: EPANET Input Parameters for Current System

```
Page 1                                        Thu Jun 23 12:05:12 2011

**********************************************************************
*                            E P A N E T                             *
*                    Hydraulic and Water Quality                     *
*                    Analysis for Pipe Networks                      *
*                           Version 2.0                              *
[ ********************************************************************** ]
```

```
[JUNCTIONS]
;ID              Elev            Demand          Pattern
 3               2282            17                              ;
 4               2280            17                              ;
 5               2268            8                               ;
 6               2268            12                              ;
 7               2247            16                              ;
 8               2245            17                              ;
 9               2233            11                              ;
 10              2232.5          7                               ;
 11              2186.4          11                              ;
 12              2270            11                              ;
 13              2270            1                               ;
 14              2246            13                              ;
 15              2244            17                              ;
 16              2214            16                              ;
 17              2213.5          16                              ;
 18              2209.5          14                              ;
 19              2209            14                              ;
 20              2168.4          10                              ;
 21              2269            12                              ;
 22              2269            2                               ;
 23              2235            7                               ;
 24              2235            3                               ;
 25              2203            9                               ;
 26              2201            9                               ;
 27              2204            10                              ;
 28              2204            10                              ;
 29              2256            3                               ;
 30              2256            5                               ;
 31              2251            2                               ;
 32              2248            5                               ;
 33              2281            5                               ;
 34              2281            7                               ;
 35              2234            10                              ;
 36              2234            11                              ;
 37              2220            4                               ;
 38              2219            3                               ;
```

39	2275	2	;
40	2274	2	;
41	2251	2	;
42	2248	3	;
43	2247	1	;
44	2247	5	;
45	2280	5	;
46	2277	1	;
47	2277	2	;
48	2255.5	3	;
49	2255.5	4	;
50	2215	2	;
51	2214	3	;
52	2212	2	;
53	2212	2	;
54	2220	5	;
55	2219	3	;
56	2236	4	;
57	2236.5	6	;
58	2251	12	;
59	2251	12	;
60	2222	10	;
61	2223	10	;
62	2202	10	;
63	2203	10	;
64	2183	8	;
65	2184	8	;
66	2214	9	;
67	2214	8	;
68	2215	5	;
69	2215	5	;
70	2222	10	;
71	2222	10	;
72	2244	11	;
73	2244	10	;
74	2242	10	;
75	2242	8	;
76	2241	9	;
77	2239	8	;
78	2240	11	;
79	2239	10	;
83	2224	9	;
84	2225	11	;
85	2223	9	;
86	2223	11	;
87	2215	8	;
88	2216	12	;
89	2214	11	;
90	2214	12	;
91	2217	5	;

```
 92              2217            8                                               ;
 93              2219            2                                               ;
 94              2219            2                                               ;
 95              2218            3                                               ;
 96              2218            2                                               ;
 97              2220            2                                               ;
 98              2220            2                                               ;
 99              2214            5                                               ;
 100             2214            6                                               ;
 101             2217            8                                               ;
 102             2217            8                                               ;

[RESERVOIRS]
;ID              Head            Pattern
 Gotera1         2310.5          1               ;
 Gotera2         2310.5          1               ;

[TANKS]
;ID              Elevation       InitLevel       MinLevel        MaxLevel
      Diameter        MinVol          VolCurve

[PIPES]
;ID              Node1           Node2           Length          Diameter
      Roughness       MinorLoss       Status
 1               3               Gotera1         370
      100             0               Open       ;
 2               5               4               544             500
      100             0               Open       ;
 3               7               6               617             400
      100             0               Open       ;
 4               9               8               441             400
      100             0               Open       ;
 5               11              10              2215            300
      100             0               Open       ;
 6               12              Gotera2         1918            500
      100             0               Open       ;
 7               14              13              640             400
      100             0               Open       ;
 8               16              15              1062            400
      100             0               Open       ;
 9               18              17              366             250
      100             0               Open       ;
 10              19              20              1151            250
      100             0               Open       ;
 11              12              21              344             200
      100             0               Open       ;
 12              13              22              344             200
      100             0               Open       ;
 13              14              23              334             200
      100             0               Open       ;
```

14	15	24	334	200	100	0	Open	;
15	16	25	397	200	100	0	Open	;
16	17	26	397	200	100	0	Open	;
17	18	27	244	200	100	0	Open	;
18	19	28	244	200	100	0	Open	;
19	3	29	816.5	500	100	0	Open	;
20	30	31	654	400	100	0	Open	;
21	32	4	1500	500	100	0	Open	;
22	29	33	1278	250	100	0	Open	;
23	30	34	1278	250	100	0	Open	;
24	7	35	860	300	100	0	Open	;
25	36	37	914	300	100	0	Open	;
26	38	8	1871	300	100	0	Open	;
27	6	35	1435	400	100	0	Open	;
28	5	36	1435	400	100	0	Open	;
29	37	32	1000	500	100	0	Open	;
30	41	43	136	400	100	0	Open	;
31	43	39	1050	250	100	0	Open	;
32	31	46	1053	250	100	0	Open	;
33	41	47	1053	250	100	0	Open	;
34	44	40	1050	250	100	0	Open	;
35	44	49	747	300	100	0	Open	;
36	42	48	967	300	100	0	Open	;
37	50	48	820	300	100	0	Open	;
38	49	40	729	300	100	0	Open	;

39	39	47	209	200	100	0	Open	;
40	46	34	550	200	100	0	Open	;
41	52	51	277	200	100	0	Open	;
42	53	45	1918.5	150	100	0	Open	;
43	50	54	1635	200	100	0	Open	;
44	51	55	1635	200	100	0	Open	;
45	54	42	1000	400	100	0	Open	;
46	38	56	199	500	100	0	Open	;
49	10	60	1000	300	100	0	Open	;
52	60	66	1019	200	100	0	Open	;
53	61	67	1019	200	100	0	Open	;
54	61	68	1000	300	100	0	Open	;
55	9	62	1659	300	100	0	Open	;
56	63	64	522	200	100	0	Open	;
57	65	69	615	200	100	0	Open	;
58	57	58	376	500	100	0	Open	;
59	55	59	648	400	100	0	Open	;
60	56	62	2085.6	200	100	0	Open	;
61	63	71	1000	200	100	0	Open	;
62	57	70	1628	200	100	0	Open	;
64	59	78	573	400	100	0	Open	;
66	58	73	79	500	100	0	Open	;
67	70	84	1000	150	100	0	Open	;
68	71	85	1000	150	100	0	Open	;
69	85	64	686	200	100	0	Open	;

70	73	84	1000	200	100	0	Open ;
71	74	83	1000	200	100	0	Open ;
72	86	65	686	200	100	0	Open ;
73	83	88	1000	200	100	0	Open ;
74	86	89	1000	200	100	0	Open ;
75	76	72	22	300	100	0	Open ;
76	91	77	26	300	100	0	Open ;
77	92	79	26	400	100	0	Open ;
78	88	74	04	500	100	0	Open ;
79	87	75	04	300	100	0	Open ;
80	94	90	1000	200	100	0	Open ;
81	93	92	50	200	100	0	Open ;
82	91	87	130	300	100	0	Open ;
83	90	101	1129	300	100	0	Open ;
84	89	102	1129	300	100	0	Open ;
85	79	95	14	250	100	0	Open ;
86	95	93	80	150	100	0	Open ;
87	97	78	1013	200	100	0	Open ;
88	98	96	299	150	100	0	Open ;
89	96	100	1718	150	100	0	Open ;
90	94	99	1069	150	100	0	Open ;
91	77	75	122	200	100	0	Open ;
92	12	13	12	500	100	0	Open ;
93	14	15	12	400	100	0	Open ;
94	16	17	12	300	100	0	Open ;

95	18	19	12	250	100	0	Open	;
96	9	10	12	300	100	0	Open	;
97	7	8	12	400	100	0	Open	;
98	5	6	12	500	100	0	Open	;
99	3	4	12	500	100	0	Open	;
100	29	30	12	400	100	0	Open	;
101	35	36	12	300	100	0	Open	;
102	37	38	12	500	100	0	Open	;
103	54	55	12	400	100	0	Open	;
104	32	42	1000	300	100	0	Open	;
105	31	41	12	400	100	0	Open	;
106	46	47	12	200	100	0	Open	;
107	33	34	12	200	100	0	Open	;
108	39	40	12	200	100	0	Open	;
109	49	48	12	300	100	0	Open	;
110	50	51	12	200	100	0	Open	;
111	56	57	16	500	100	0	Open	;
112	73	74	12	500	100	0	Open	;
113	72	75	12	300	100	0	Open	;
114	78	79	16	400	100	0	Open	;
115	76	77	16	300	100	0	Open	;
120	88	89	12	400	100	0	Open	;
121	87	90	12	300	100	0	Open	;
122	93	94	20	150	100	0	Open	;
123	84	85	12	200	100	0	Open	;

```
 124             83               86               12               200
      100              0                Open       ;
 125             70               71               12               200
      100              0                Open       ;
 126             62               63               12               300
      100              0                Open       ;
 127             64               65               20               200
      100              0                Open       ;
 128             61               60               12               300
      100              0                Open       ;
 129             45               33               12               200
      100              0                Open       ;
 130             53               52               1000             150
      100              0                Open       ;
 131             97               98               20               150
      100              0                Open       ;
 132             100              99               16               150
      100              0                Open       ;
 133             95               96               20               150
      100              0                Open       ;
 134             90               89               1000             300
      100              0                Open       ;
 135             86               85               20               150
      100              0                Open       ;
 136             12               5                50               500
      100              0                Open       ;
 47              76               72               1021             300
      100              0                Open       ;
 48              44               43               20               300
      100              0                Open       ;


[PUMPS]
;ID              Node1            Node2            Parameters

[VALVES]
;ID              Node1            Node2            Diameter         Type
      Setting          MinorLoss

[TAGS]

[DEMANDS]
;Junction        Demand           Pattern          Category

[STATUS]
;ID              Status/Setting
```

```
[PATTERNS]
;ID                Multipliers
;
 1                 1.0                1.3                1.1                0.6

[CURVES]
;ID                X-Value            Y-Value
;PUMP:
 1                 600                150
;PUMP:
 2                 600                150

[CONTROLS]

[RULES]

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

[EMITTERS]
;Junction          Coefficient

[QUALITY]
;Node              InitQual

[SOURCES]
;Node              Type               Quality            Pattern

[REACTIONS]
;Type        Pipe/Tank          Coefficient


[REACTIONS]
 Order Bulk                   1
 Order Tank                   1
 Order Wall                   1
 Global Bulk                  0
 Global Wall                  0
 Limiting Potential           0
 Roughness Correlation        0

[MIXING]
;Tank              Model

[TIMES]
 Duration               24
 Hydraulic Timestep     1:00
```

```
 Quality Timestep       0
 Pattern Timestep       6
 Pattern Start          0:00
 Report Timestep        1:00
 Report Start           0:00
 Start ClockTime        12 am
 Statistic              None

[REPORT]
 Status                 Yes
 Summary                No
 Page                   0

[OPTIONS]
 Units                  LPS
 Headloss               H-W
 Specific Gravity       1
 Viscosity              1
 Trials                 40
 Accuracy               0.001
 Unbalanced             Continue 10
 Pattern                1
 Demand Multiplier      1.0
 Emitter Exponent       0.5
 Quality                None mg/L
 Diffusivity            1
 Tolerance              0.01

[COORDINATES]
;Node              X-Coord           Y-Coord
 3                 473760.4152       991186.7318
 4                 473771.4983       991144.1432
 5                 473853.9587       990621.9472
 6                 473860.0981       990583.3769
 7                 473923.0241       989988.1607
 8                 473918.4170       989939.0778
 9                 473877.3169       989501.2041
 10                473872.3019       989447.7747
 11                473975.6822       987554.8294
 12                473925.4164       990645.5578
 13                473928.1036       990600.8252
 14                473995.5183       989969.5800
 15                473991.5401       989925.8610
 16                474087.4015       988917.9532
 17                474110.3342       988867.9623
 18                474255.7018       988551.0760
 19                474276.0278       988505.3875
 20                474032.8436       987589.3414
 21                474208.3963       990641.3530
 22                474224.0640       990612.1404
```

23	474255.3961	989940.6229
24	474256.5714	989915.4331
25	474456.8681	989015.4114
26	474460.8573	988978.3299
27	474468.5374	988576.3890
28	474466.8661	988544.3217
29	472984.2145	991159.8521
30	472934.0920	991166.9163
31	472324.4392	990985.9263
32	472324.2805	990916.8809
33	473149.0706	992122.7974
34	473149.4270	992093.2403
35	473063.5261	990016.8779
36	473032.6010	990028.2336
37	472196.1995	990296.5821
38	472142.7430	990271.5719
39	472086.9696	992017.7228
40	472027.4154	992004.0265
41	472244.4099	990980.9170
42	472253.3429	990919.5794
43	472128.1043	990973.6370
44	472066.2967	990979.2435
45	473102.8221	992088.6642
46	472331.0323	992012.8818
47	472276.3131	992018.4327
48	471478.3906	991453.7421
49	471515.5935	991475.8085
50	470898.4771	990882.4783
51	470807.1179	990843.7590
52	470584.0849	990719.2209
53	470499.5491	990805.4215
54	472140.9516	990350.1670
55	472107.4193	990305.6152
56	471996.5092	990157.8008
57	471947.9398	990122.1370
58	471755.9988	989824.1905
59	471726.5649	989822.4438
60	473589.3984	989347.4441
61	473569.8925	989340.8144
62	472836.5417	988387.6548
63	472795.2882	988360.9527
64	472344.4219	988069.2290
65	472266.3935	988034.8763
66	474066.3259	988713.5456
67	474079.1112	988686.9078
68	471935.5397	987564.9096
69	471890.7588	987595.6522
70	472616.9213	988763.3749
71	472633.5818	988720.6512
72	471842.5691	989459.4542

```
 73               471872.7787      989459.1663
 74               471875.5976      989409.8280
 75               471830.7776      989413.9526
 76               471521.0029      989465.9412
 77               471535.0376      989411.6366
 78               471479.0738      989461.7982
 79               471489.7105      989408.0489
 83               472202.8634      988781.3070
 84               472263.7658      988775.3988
 85               472268.3771      988729.7608
 86               472207.7058      988735.6406
 87               471790.0743      988816.6738
 88               471824.8940      988813.0160
 89               471821.6321      988770.7238
 90               471781.9065      988776.9619
 91               471670.8917      988817.7770
 92               471636.2403      988812.0835
 93               471210.7376      988715.7531
 94               471230.0915      988666.2381
 95               470970.1231      988998.8695
 96               470925.3593      988974.2654
 97               470663.2260      989040.3549
 98               470661.6395      988969.6345
 99               470832.8488      987826.4570
 100              470805.3164      987855.5815
 101              471719.1454      987668.4645
 102              471746.5045      987663.5874
 Gotera1          473480.3398      992663.1828
 Gotera2          473528.8992      992642.3299


[LABELS]
;X-Coord            Y-Coord            Label & Anchor Node
 473111.8660      992740.9300      "Gotera "
 471647.0453      989576.2975      "Lafto"
 471547.0741      990701.8104      "Gofa"
 473081.4307      989415.7305      "Sarise"
 474128.5864      990189.3122      "To Bole Bulubula"
 471454.2186      988135.6260      "Hana Mariyam"
 471384.2518      991826.3025      "Kera"
 472966.2721      990699.6878      "Yosef cherch"
 470913.3542      988537.2468      "Lebu"

[BACKDROP]
 DIMENSIONS       0.00             0.00             10000.00         10000.00
 UNITS            Meters
 FILE
 OFFSET           0.00             0.00
```

[END]

Appendix C: EPANET Report for the First Scenarios

```
*                    Hydraulic and Water Quality                    *
*                    Analysis for Pipe Networks                     *
*                          Version 2.0                              *
**********************************************************************

Input File: shime kiya.net


Link - Node Table:
```

Link ID	Start Node	End Node	Length m	Diameter mm
2	5	4	544	500
3	7	6	617	400
4	9	8	441	400
5	11	10	2215	300
6	12	Gotera2	1918	500
7	14	13	640	400
8	16	15	1062	400
9	18	17	366	250
10	19	20	1151	250
11	12	21	344	200
12	13	22	344	200
13	14	23	334	200
14	15	24	334	200
15	16	25	397	200
16	17	26	397	200
17	18	27	244	200
18	19	28	244	200
19	3	29	816.5	500
20	30	31	654	400
21	32	4	1500	500
22	29	33	1278	250
23	30	34	1278	250
24	7	35	860	300
25	36	37	914	300
26	38	8	1871	300
27	6	35	1435	400
28	5	36	1435	400
29	37	32	1000	500
30	41	43	136	400
31	43	39	1050	250
32	31	46	1053	250
33	41	47	1053	250
34	44	40	1050	250
35	44	49	747	300
36	42	48	967	300
37	50	48	820	300
38	49	40	729	300

Link - Node Table: (continued)

Link ID	Start Node	End Node	Length m	Diameter mm
39	39	47	209	200
40	46	34	550	200
41	52	51	277	200
42	53	45	1918.5	150
43	50	54	1635	200
44	51	55	1635	200
45	54	42	1000	400
46	38	56	199	500
49	10	60	1000	300
52	60	66	1019	200
53	61	67	1019	200
54	61	68	1000	300
55	9	62	1659	300
56	63	64	522	200
57	65	69	615	200
58	57	58	376	500
59	55	59	648	400
60	56	62	2085.6	200
61	63	71	1000	200
62	57	70	1628	200
64	59	78	573	400
66	58	73	79	500
67	70	84	1000	150
68	71	85	1000	150
69	85	64	686	200
70	73	84	1000	200
71	74	83	1000	200
72	86	65	686	200
73	83	88	1000	200
74	86	89	1000	200
75	76	72	22	300
76	91	77	26	300
77	92	79	26	400
78	88	74	04	500
79	87	75	04	300
80	94	90	1000	200
81	93	92	50	200
82	91	87	130	300
83	90	101	1129	300
84	89	102	1129	300
85	79	95	14	250
86	95	93	80	150
87	97	78	1013	200
88	98	96	299	150
89	96	100	1718	150
90	94	99	1069	150

Page 3
Link - Node Table: (continued)

Link ID	Start Node	End Node	Length m	Diameter mm	
92	12	13	12	500	
93	14	15	12	400	
94	16	17	12	300	
95	18	19	12	250	
96	9	10	12	300	
97	7	8	12	400	
98	5	6	12	500	
99	3	4	12	500	
100	29	30	12	400	
101	35	36	12	300	
102	37	38	12	500	
103	54	55	12	400	
104	32	42	1000	300	
105	31	41	12	400	
106	46	47	12	200	
107	33	34	12	200	
108	39	40	12	200	
109	49	48	12	300	
110	50	51	12	200	
111	56	57	16	500	
112	73	74	12	500	
113	72	75	12	300	
114	78	79	16	400	
115	76	77	16	300	
120	88	89	12	400	
121	87	90	12	300	
122	93	94	20	150	
123	84	85	12	200	
124	83	86	12	200	
125	70	71	12	200	
126	62	63	12	300	
127	64	65	20	200	
128	61	60	12	300	
129	45	33	12	200	
130	53	52	1000	150	
131	97	98	20	150	
132	100	99	16	150	
133	95	96	20	150	
134	90	89	1000	300	
135	86	85	20	150	
136	12	5	50	500	
47	76	72	1021	300	
48	44	43	20	300	
1	Gotera1	1	250	12	
V1	1	3	#N/A	500	Valve

Node Results at 7:00 Hrs:

Node ID	Demand LPS	Head m	Pressure m	Quality
3	22.10	2885.84	603.84	0.00
4	22.10	2885.86	605.86	0.00
5	10.40	2890.98	622.98	0.00
6	15.60	2890.91	622.91	0.00
7	20.80	2885.90	638.90	0.00
8	22.10	2885.78	640.78	0.00

Node ID	Demand LPS	Head m	Pressure m	Quality
9	14.30	2883.52	650.52	0.00
10	9.10	2883.44	650.94	0.00
11	14.30	2882.81	696.41	0.00
12	14.30	2892.75	622.75	0.00
13	1.30	2892.71	622.71	0.00
14	16.90	2887.18	641.18	0.00
15	22.10	2887.10	643.10	0.00
16	20.80	2881.98	667.98	0.00
17	20.80	2881.83	668.33	0.00
18	18.20	2876.32	666.82	0.00
19	18.20	2876.26	667.26	0.00
20	13.00	2875.59	707.19	0.00
21	15.60	2891.92	622.92	0.00
22	2.60	2892.68	623.68	0.00
23	9.10	2886.88	651.88	0.00
24	3.90	2887.04	652.04	0.00
25	11.70	2881.41	678.41	0.00
26	11.70	2881.27	680.27	0.00
27	13.00	2875.90	671.90	0.00
28	13.00	2875.84	671.84	0.00
29	3.90	2884.53	628.53	0.00
30	6.50	2884.49	628.49	0.00
31	2.60	2883.01	632.01	0.00
32	6.50	2882.07	634.07	0.00
33	6.50	2883.20	602.20	0.00
34	9.10	2883.20	602.20	0.00
35	13.00	2888.36	654.36	0.00
36	14.30	2888.36	654.36	0.00
37	5.20	2880.81	660.81	0.00
38	3.90	2880.77	661.77	0.00
39	2.60	2882.25	607.25	0.00
40	2.60	2882.17	608.17	0.00
41	2.60	2882.99	631.99	0.00
42	3.90	2879.24	631.24	0.00
43	1.30	2882.78	635.78	0.00
44	6.50	2882.70	635.70	0.00
45	6.50	2883.18	603.18	0.00
46	1.30	2882.84	605.84	0.00
47	2.60	2882.82	605.82	0.00
48	3.90	2880.97	625.47	0.00
49	5.20	2881.04	625.54	0.00
50	2.60	2879.99	664.99	0.00
51	3.90	2879.96	665.96	0.00
52	2.60	2879.96	667.96	0.00
53	2.60	2880.41	668.41	0.00
54	6.50	2877.37	657.37	0.00
55	3.90	2877.34	658.34	0.00

Node ID	Demand LPS	Head m	Pressure m	Quality	
56	5.20	2879.84	643.84	0.00	
57	7.80	2879.77	643.27	0.00	
58	15.60	2878.45	627.45	0.00	
59	15.60	2875.71	624.71	0.00	
60	13.00	2880.04	658.04	0.00	
61	13.00	2880.02	657.02	0.00	
62	13.00	2878.26	676.26	0.00	
63	13.00	2878.23	675.23	0.00	
64	10.40	2876.09	693.09	0.00	
65	10.40	2876.06	692.06	0.00	
66	11.70	2878.60	664.60	0.00	
67	10.40	2878.86	664.86	0.00	
68	6.50	2879.96	664.96	0.00	
69	6.50	2875.76	660.76	0.00	
70	13.00	2876.24	654.24	0.00	
71	13.00	2876.24	654.24	0.00	
72	14.30	2871.86	627.86	0.00	
73	13.00	2878.20	634.20	0.00	
74	13.00	2878.18	636.18	0.00	
75	10.40	2871.87	629.87	0.00	
76	11.70	2871.86	630.86	0.00	
77	10.40	2871.86	632.86	0.00	
78	14.30	2874.66	634.66	0.00	
79	13.00	2874.64	635.64	0.00	
83	11.70	2876.25	652.25	0.00	
84	14.30	2876.13	651.13	0.00	
85	11.70	2876.13	653.12	0.00	
86	14.30	2876.22	653.22	0.00	
87	10.40	2871.88	656.88	0.00	
88	15.60	2878.17	662.17	0.00	
89	14.30	2878.13	664.13	0.00	
90	15.60	2871.94	657.94	0.00	
91	6.50	2871.86	654.86	0.00	
92	10.40	2874.64	657.64	0.00	
93	2.60	2874.44	655.44	0.00	
94	2.60	2873.99	654.99	0.00	
95	3.90	2874.62	656.62	0.00	
96	2.60	2874.53	656.53	0.00	
97	2.60	2874.49	654.49	0.00	
98	2.60	2874.49	654.49	0.00	
99	6.50	2871.13	657.13	0.00	
100	7.80	2871.13	657.13	0.00	
101	10.40	2871.76	654.76	0.00	
102	10.40	2877.95	660.95	0.00	
1	22.10	-950239.30	-952521.30	0.00	
Gotera1	-21.21	3003.65	693.15	0.00	Reservoir
Gotera2	-965.49	3003.65	693.15	0.00	Reservoir

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
2	362.28	1.85	9.41	Open
3	-185.96	1.48	8.12	Open
4	-144.83	1.15	5.11	Open
5	-14.30	0.20	0.28	Open
6	-965.49	4.92	57.82	Open
7	-192.40	1.53	8.64	Open
8	-140.40	1.12	4.82	Open
9	-75.40	1.54	15.05	Open
10	13.00	0.26	0.58	Open
11	15.60	0.50	2.41	Open
12	2.60	0.08	0.09	Open
13	9.10	0.29	0.89	Open
14	3.90	0.12	0.19	Open
15	11.70	0.37	1.42	Open
16	11.70	0.37	1.42	Open
17	13.00	0.41	1.72	Open
18	13.00	0.41	1.72	Open
19	139.05	0.71	1.60	Open
20	93.30	0.74	2.26	Open
21	-178.15	0.91	2.53	Open
22	17.83	0.36	1.04	Open
23	17.53	0.36	1.01	Open
24	-49.68	0.70	2.86	Open
25	88.06	1.25	8.25	Open
26	-47.91	0.68	2.67	Open
27	81.85	0.65	1.78	Open
28	83.19	0.66	1.83	Open
29	-122.21	0.62	1.26	Open
30	75.16	0.60	1.51	Open
31	12.11	0.25	0.51	Open
32	6.49	0.13	0.16	Open
33	6.45	0.13	0.16	Open
34	12.00	0.24	0.50	Open
35	43.25	0.61	2.21	Open
36	-38.65	0.55	1.80	Open
37	-31.15	0.44	1.20	Open
38	-35.65	0.50	1.55	Open
39	-16.74	0.53	2.75	Open
40	-7.70	0.25	0.65	Open
41	0.36	0.01	0.00	Open
42	-5.56	0.31	1.45	Open
43	12.51	0.40	1.60	Open
44	12.50	0.40	1.60	Open
45	-84.19	0.67	1.87	Open
46	249.08	1.27	4.70	Open
49	54.60	0.77	3.41	Open

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
52	11.70	0.37	1.42	Open
53	10.40	0.33	1.14	Open
54	6.50	0.09	0.07	Open
55	52.53	0.74	3.17	Open
56	20.79	0.66	4.11	Open
57	6.50	0.21	0.48	Open
58	213.02	1.08	3.52	Open
59	98.79	0.79	2.52	Open
60	8.33	0.27	0.75	Open
61	14.07	0.45	1.99	Open
62	14.73	0.47	2.17	Open
64	83.19	0.66	1.83	Open
66	197.42	1.01	3.06	Open
67	1.40	0.08	0.11	Open
68	1.40	0.08	0.11	Open
69	1.99	0.06	0.05	Open
70	14.40	0.46	2.08	Open
71	13.80	0.44	1.92	Open
72	4.52	0.14	0.24	Open
73	-13.77	0.44	1.92	Open
74	-13.74	0.44	1.91	Open
75	-12.33	0.17	0.22	Open
76	4.88	0.07	0.03	Open
77	-30.56	0.24	0.29	Open
78	-143.22	0.73	1.64	Open
79	41.92	0.59	2.08	Open
80	14.29	0.45	2.05	Open
81	-20.16	0.64	3.88	Open
82	-11.38	0.16	0.19	Open
83	10.40	0.15	0.16	Open
84	10.40	0.15	0.16	Open
85	21.61	0.44	1.49	Open
86	7.06	0.40	2.25	Open
87	-3.72	0.12	0.17	Open
88	-1.48	0.08	0.13	Open
89	6.57	0.37	1.97	Open
90	7.73	0.44	2.67	Open
91	-3.34	0.11	0.14	Open
92	196.30	1.00	3.03	Open
93	166.40	1.32	6.60	Open
94	107.90	1.53	12.03	Open
95	44.20	0.90	5.61	Open
96	78.00	1.10	6.60	Open
97	214.84	1.71	10.59	Open
98	283.41	1.44	5.98	Open
99	-162.04	0.83	2.13	Open
100	117.32	0.93	3.45	Open

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
101	19.17	0.27	0.50	Open
102	205.07	1.04	3.30	Open
103	90.19	0.72	2.13	Open
104	49.44	0.70	2.83	Open
105	84.21	0.67	1.89	Open
106	12.89	0.41	1.71	Open
107	-0.73	0.02	0.02	Open
108	26.25	0.84	6.33	Open
109	73.70	1.04	5.93	Open
110	16.04	0.51	2.53	Open
111	235.55	1.20	4.24	Open
112	170.02	0.87	2.33	Open
113	-28.18	0.40	0.99	Open
114	65.17	0.52	1.17	Open
115	2.18	0.03	0.00	Open
120	113.85	0.91	3.27	Open
121	-63.70	0.90	4.54	Open
122	24.62	1.39	22.80	Open
123	1.50	0.05	0.02	Open
124	15.88	0.51	2.48	Open
125	0.33	0.01	0.00	Open
126	47.86	0.68	2.65	Open
127	12.38	0.39	1.56	Open
128	-29.90	0.42	1.12	Open
129	-12.06	0.38	1.49	Open
130	2.96	0.17	0.45	Open
131	1.12	0.06	0.07	Open
132	-1.23	0.07	0.09	Open
133	10.65	0.60	4.82	Open
134	-75.41	1.07	6.19	Open
135	10.79	0.61	4.96	Open
136	739.29	3.77	35.27	Open
47	-1.55	0.02	0.00	Open
48	-61.74	0.87	4.29	Open
1	21.21	187.58	3812972.00	Open
V1	0.00	0.00	0.00	Closed Valve

Appendix D: EPANET Report for the Second Scenarios

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Page 1                                                 6/24/2011 10:43:46 AM
**********************************************************************
*                             E P A N E T                            *
*                     Hydraulic and Water Quality                    *
*                     Analysis for Pipe Networks                     *
*                            Version 2.0                             *
**********************************************************************

Node Results at 0:00 Hrs:
----------------------------------------------------------------------
Node                      Demand      Head  Pressure   Quality
ID                           LPS         m         m
----------------------------------------------------------------------
3                          17.00   2304.01     22.01      0.00
4                          17.00   2303.89     23.89      0.00
5                           8.00   2302.39     34.39      0.00
6                          12.00   2302.36     34.36      0.00
7                          16.00   2300.01     53.01      0.00
8                          17.00   2299.95     54.95      0.00
9                          11.00   2298.73     65.73      0.00
10                          7.00   2298.68     66.18      0.00
11                         11.00   2298.29    111.89      0.00
12                         11.00   2302.41     32.41      0.00
13                          1.00   2302.39     32.39      0.00
14                         13.00   2298.98     52.98      0.00
15                         17.00   2298.93     54.93      0.00
16                         16.00   2295.78     81.78      0.00
17                         16.00   2295.69     82.19      0.00
18                         14.00   2292.31     82.81      0.00
19                         14.00   2292.26     83.26      0.00
20                         10.00   2291.85    123.45      0.00
21                         12.00   2301.90     32.90      0.00
22                          2.00   2302.37     33.37      0.00
23                          7.00   2298.80     63.80      0.00
24                          3.00   2298.90     63.90      0.00
25                          9.00   2295.44     92.44      0.00
26                          9.00   2295.35     94.35      0.00
27                         10.00   2292.05     88.05      0.00
28                         10.00   2292.01     88.01      0.00
29                          3.00   2303.09     47.09      0.00
30                          5.00   2303.07     47.07      0.00
31                          2.00   2302.00     51.00      0.00
32                          5.00   2300.33     52.33      0.00
33                          5.00   2302.19     21.19      0.00
34                          7.00   2302.19     21.19      0.00
35                         10.00   2301.28     67.28      0.00
36                         11.00   2301.28     67.28      0.00
37                          4.00   2298.84     78.84      0.00
38                          3.00   2298.81     79.81      0.00
39                          2.00   2301.44     26.44      0.00
40                          2.00   2301.39     27.39      0.00
41                          2.00   2301.99     50.99      0.00
42                          3.00   2299.00     51.00      0.00
43                          1.00   2301.84     54.84      0.00
44                          5.00   2301.77     54.77      0.00
45                          5.00   2302.18     22.18      0.00
```

46	1.00	2301.89	24.89	0.00
47	2.00	2301.88	24.88	0.00
48	3.00	2300.47	44.97	0.00
49	4.00	2300.52	45.02	0.00

Node ID	Demand LPS	Head m	Pressure m	Quality
50	2.00	2299.78	84.78	0.00
51	3.00	2299.76	85.76	0.00
52	2.00	2299.76	87.76	0.00
53	2.00	2300.14	88.14	0.00
54	5.00	2297.83	77.83	0.00
55	3.00	2297.81	78.81	0.00
56	4.00	2298.21	62.21	0.00
57	6.00	2298.17	61.67	0.00
58	12.00	2297.34	46.34	0.00
59	12.00	2296.74	45.74	0.00
60	10.00	2296.58	74.58	0.00
61	10.00	2296.57	73.57	0.00
62	10.00	2296.50	94.50	0.00
63	10.00	2296.48	93.48	0.00
64	8.00	2295.57	112.57	0.00
65	8.00	2295.56	111.56	0.00
66	9.00	2295.69	81.69	0.00
67	8.00	2295.86	81.86	0.00
68	5.00	2296.53	81.53	0.00
69	5.00	2295.38	80.38	0.00
70	10.00	2295.67	73.67	0.00
71	10.00	2295.67	73.67	0.00
72	11.00	2293.60	49.60	0.00
73	10.00	2297.19	53.19	0.00
74	10.00	2297.17	55.17	0.00
75	8.00	2293.61	51.61	0.00
76	9.00	2293.60	52.60	0.00
77	8.00	2293.60	54.60	0.00
78	11.00	2296.05	56.05	0.00
79	10.00	2296.04	57.04	0.00
83	9.00	2295.77	71.77	0.00
84	11.00	2295.66	70.66	0.00
85	9.00	2295.66	72.66	0.00
86	11.00	2295.75	72.75	0.00
87	8.00	2293.61	78.61	0.00
88	12.00	2297.17	81.17	0.00
89	11.00	2297.14	83.14	0.00
90	12.00	2293.65	79.65	0.00
91	5.00	2293.60	76.60	0.00
92	8.00	2296.03	79.03	0.00
93	2.00	2295.88	76.88	0.00
94	2.00	2295.53	76.53	0.00
95	3.00	2296.02	78.02	0.00
96	2.00	2295.96	77.96	0.00
97	2.00	2295.94	75.94	0.00
98	2.00	2295.94	75.94	0.00
99	5.00	2293.82	79.82	0.00

100	6.00	2293.82	79.82	0.00	
101	8.00	2293.54	76.54	0.00	
102	8.00	2297.03	80.03	0.00	
Gotera1	-507.11	2310.50	0.00	0.00	Reservoir
Gotera2	-234.89	2310.50	0.00	0.00	Reservoir

Link Results at 0:00 Hrs:

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
1	-507.11	2.58	17.55	Open
2	-186.46	0.95	2.75	Open
3	-123.66	0.98	3.81	Open
4	-104.02	0.83	2.77	Open
5	-11.00	0.16	0.18	Open
6	-234.89	1.20	4.22	Open
7	-148.00	1.18	5.32	Open
8	-108.00	0.86	2.97	Open
9	-58.00	1.18	9.26	Open
10	10.00	0.20	0.36	Open
11	12.00	0.38	1.48	Open
12	2.00	0.06	0.05	Open
13	7.00	0.22	0.55	Open
14	3.00	0.10	0.11	Open
15	9.00	0.29	0.87	Open
16	9.00	0.29	0.87	Open
17	10.00	0.32	1.06	Open
18	10.00	0.32	1.06	Open
19	114.68	0.58	1.12	Open
20	78.00	0.62	1.62	Open
21	-171.97	0.88	2.37	Open
22	14.47	0.29	0.71	Open
23	14.21	0.29	0.68	Open
24	-34.85	0.49	1.48	Open
25	47.84	0.68	2.67	Open
26	-21.49	0.30	0.61	Open
27	51.41	0.41	0.75	Open
28	52.28	0.42	0.77	Open
29	-134.01	0.68	1.49	Open
30	63.61	0.51	1.11	Open
31	10.26	0.21	0.37	Open

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
37	-25.63	0.36	0.84	Open
38	-30.81	0.44	1.18	Open
39	-14.37	0.46	2.07	Open
40	-6.98	0.22	0.54	Open
41	0.70	0.02	0.01	Open
42	-4.70	0.27	1.06	Open
43	10.67	0.34	1.19	Open
44	10.66	0.34	1.19	Open
45	-65.31	0.52	1.17	Open
46	196.34	1.00	3.03	Open
49	42.00	0.59	2.10	Open
52	9.00	0.29	0.87	Open
53	8.00	0.25	0.70	Open
54	5.00	0.07	0.04	Open
55	33.02	0.47	1.34	Open
56	13.08	0.42	1.74	Open
57	5.00	0.16	0.29	Open
58	165.41	0.84	2.20	Open
59	78.64	0.63	1.65	Open
60	8.72	0.28	0.82	Open
61	8.66	0.28	0.81	Open
62	12.21	0.39	1.53	Open
64	66.64	0.53	1.21	Open
66	153.41	0.78	1.92	Open
67	0.44	0.02	0.01	Open
68	0.44	0.02	0.01	Open
69	3.13	0.10	0.12	Open
70	12.19	0.39	1.53	Open
71	11.63	0.37	1.40	Open
72	4.79	0.15	0.27	Open
73	-11.61	0.37	1.40	Open
74	-11.62	0.37	1.40	Open
75	-9.05	0.13	0.12	Open
76	3.70	0.05	0.02	Open
77	-25.38	0.20	0.21	Open
78	-109.59	0.56	1.04	Open
79	32.30	0.46	1.30	Open
80	13.64	0.43	1.88	Open
81	-17.38	0.55	2.95	Open
82	-8.70	0.12	0.11	Open
83	8.00	0.11	0.10	Open
84	8.00	0.11	0.10	Open
85	17.37	0.35	1.00	Open
86	6.13	0.35	1.73	Open
87	-2.89	0.09	0.11	Open
88	-1.11	0.06	0.07	Open
89	5.13	0.29	1.25	Open

Link Results at 0:00 Hrs: (continued)

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
90	5.87	0.33	1.60	Open
91	-2.54	0.08	0.08	Open
92	151.00	0.77	1.86	Open
93	128.00	1.02	4.06	Open
94	83.00	1.17	7.40	Open
95	34.00	0.69	3.45	Open
96	60.00	0.85	4.06	Open
97	142.51	1.13	4.96	Open
98	187.07	0.95	2.77	Open
99	375.43	1.91	10.06	Open
100	97.21	0.77	2.44	Open
101	6.56	0.09	0.06	Open
102	177.85	0.91	2.52	Open
103	70.98	0.56	1.36	Open
104	32.96	0.47	1.34	Open
105	70.80	0.56	1.36	Open
106	11.18	0.36	1.30	Open
107	-0.23	0.01	0.00	Open
108	22.63	0.72	4.80	Open
109	63.98	0.91	4.58	Open
110	12.96	0.41	1.71	Open
111	183.62	0.94	2.67	Open
112	131.22	0.67	1.43	Open
113	-21.77	0.31	0.62	Open
114	52.75	0.42	0.78	Open
115	1.77	0.02	0.00	Open
120	85.98	0.68	1.95	Open
121	-49.00	0.69	2.79	Open
122	21.51	1.22	17.75	Open
123	1.63	0.05	0.04	Open
124	14.24	0.45	2.03	Open
125	1.77	0.06	0.05	Open
126	31.74	0.45	1.25	Open
127	8.21	0.26	0.74	Open
128	-23.00	0.33	0.69	Open
129	-9.70	0.31	1.00	Open
130	2.70	0.15	0.38	Open
131	0.89	0.05	0.04	Open
132	-0.87	0.05	0.05	Open
133	8.24	0.47	3.01	Open
134	-55.36	0.78	3.49	Open
135	10.06	0.57	4.35	Open
136	60.89	0.31	0.35	Open
47	-1.71	0.02	0.00	Open
48	-52.35	0.74	3.15	Open

Appendix E: EPANET Report for the Third Scenarios

Node ID	Demand LPS	Head m	Pressure m	Quality
3	17.00	2238.05	-43.95	0.00
4	17.00	2238.07	-41.93	0.00
5	8.00	2241.22	-26.78	0.00
6	12.00	2241.17	-26.83	0.00
7	16.00	2238.09	-8.91	0.00
8	17.00	2238.01	-6.99	0.00
9	11.00	2236.63	3.63	0.00
10	7.00	2236.58	4.08	0.00
11	11.00	2236.19	49.79	0.00
12	11.00	2242.30	-27.70	0.00
13	1.00	2242.28	-27.72	0.00
14	13.00	2238.87	-7.13	0.00
15	17.00	2238.83	-5.17	0.00
16	16.00	2235.67	21.67	0.00
17	16.00	2235.59	22.09	0.00
18	14.00	2232.20	22.70	0.00
19	14.00	2232.16	23.16	0.00
20	10.00	2231.75	63.35	0.00
21	12.00	2241.79	-27.21	0.00
22	2.00	2242.26	-26.74	0.00
23	7.00	2238.69	3.69	0.00
24	3.00	2238.79	3.79	0.00
25	9.00	2235.33	32.33	0.00
26	9.00	2235.24	34.24	0.00
27	10.00	2231.94	27.94	0.00
28	10.00	2231.90	27.90	0.00
29	3.00	2237.25	-18.75	0.00
30	5.00	2237.22	-18.78	0.00
31	2.00	2236.31	-14.69	0.00
32	5.00	2235.74	-12.26	0.00
33	5.00	2236.43	-44.57	0.00
34	7.00	2236.43	-44.57	0.00
35	10.00	2239.60	5.60	0.00
36	11.00	2239.60	5.60	0.00
37	4.00	2234.96	14.96	0.00
38	3.00	2234.94	15.94	0.00
39	2.00	2235.84	-39.16	0.00
40	2.00	2235.80	-38.20	0.00
41	2.00	2236.30	-14.70	0.00
42	3.00	2233.99	-14.01	0.00
43	1.00	2236.17	-10.83	0.00
44	5.00	2236.12	-10.88	0.00
45	5.00	2236.42	-43.58	0.00
46	1.00	2236.21	-40.79	0.00
47	2.00	2236.20	-40.80	0.00
48	3.00	2235.06	-20.44	0.00

Node ID	Demand LPS	Head m	Pressure m	Quality
49	4.00	2235.10	-20.40	0.00
50	2.00	2234.45	19.45	0.00
51	3.00	2234.43	20.43	0.00
52	2.00	2234.43	22.43	0.00
53	2.00	2234.71	22.71	0.00
54	5.00	2232.84	12.84	0.00
55	3.00	2232.83	13.83	0.00
56	4.00	2234.36	-1.64	0.00
57	6.00	2234.32	-2.18	0.00
58	12.00	2233.51	-17.49	0.00
59	12.00	2231.82	-19.18	0.00
60	10.00	2234.48	12.48	0.00
61	10.00	2234.48	11.48	0.00
62	10.00	2233.39	31.39	0.00
63	10.00	2233.37	30.37	0.00
64	8.00	2232.05	49.05	0.00
65	8.00	2232.04	48.04	0.00
66	9.00	2233.60	19.60	0.00
67	8.00	2233.76	19.76	0.00
68	5.00	2234.43	19.43	0.00
69	5.00	2231.86	16.86	0.00
70	10.00	2232.15	10.15	0.00
71	10.00	2232.15	10.15	0.00
72	11.00	2229.45	-14.55	0.00
73	10.00	2233.36	-10.64	0.00
74	10.00	2233.34	-8.66	0.00
75	8.00	2229.46	-12.54	0.00
76	9.00	2229.45	-11.55	0.00
77	8.00	2229.45	-9.55	0.00
78	11.00	2231.18	-8.82	0.00
79	10.00	2231.17	-7.83	0.00
83	9.00	2232.16	8.16	0.00
84	11.00	2232.08	7.08	0.00
85	9.00	2232.08	9.08	0.00
86	11.00	2232.14	9.14	0.00
87	8.00	2229.47	14.47	0.00
88	12.00	2233.34	17.34	0.00
89	11.00	2233.31	19.31	0.00
90	12.00	2229.50	15.50	0.00
91	5.00	2229.45	12.45	0.00
92	8.00	2231.16	14.16	0.00
93	2.00	2231.04	12.04	0.00
94	2.00	2230.76	11.76	0.00
95	3.00	2231.15	13.15	0.00
96	2.00	2231.09	13.09	0.00
97	2.00	2231.07	11.07	0.00
98	2.00	2231.07	11.07	0.00

ID	LPS	m	m		
99	5.00	2229.01	15.01	0.00	
100	6.00	2229.01	15.01	0.00	
101	8.00	2229.39	12.39	0.00	
102	8.00	2233.20	16.20	0.00	
1	17.00	-592649.50	-594931.50	0.00	
Gotera1	-16.45	2310.50	0.00	0.00	Reservoir
Gotera2	-742.55	2310.50	0.00	0.00	Reservoir

Link Results at 1:00 Hrs:

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
2	278.57	1.42	5.79	Open
3	-143.04	1.14	4.99	Open
4	-111.40	0.89	3.14	Open
5	-11.00	0.16	0.18	Open
6	-742.55	3.78	35.56	Open
7	-148.00	1.18	5.32	Open
8	-108.00	0.86	2.97	Open
9	-58.00	1.18	9.26	Open
10	10.00	0.20	0.36	Open
11	12.00	0.38	1.48	Open
12	2.00	0.06	0.05	Open
13	7.00	0.22	0.55	Open
14	3.00	0.10	0.11	Open
15	9.00	0.29	0.87	Open
16	9.00	0.29	0.87	Open
17	10.00	0.32	1.06	Open
18	10.00	0.32	1.06	Open
19	106.97	0.54	0.98	Open
20	71.77	0.57	1.39	Open
21	-137.05	0.70	1.56	Open
22	13.72	0.28	0.64	Open
23	13.48	0.27	0.62	Open
24	-38.22	0.54	1.76	Open
25	67.73	0.96	5.08	Open
26	-36.85	0.52	1.64	Open
27	62.96	0.50	1.09	Open
28	63.99	0.51	1.13	Open
29	-94.03	0.48	0.77	Open
30	57.82	0.46	0.93	Open
31	9.32	0.19	0.31	Open
32	4.99	0.10	0.10	Open
33	4.96	0.10	0.10	Open
34	9.23	0.19	0.31	Open
35	33.27	0.47	1.36	Open

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
36	-29.73	0.42	1.11	Open
37	-23.96	0.34	0.74	Open
38	-27.42	0.39	0.95	Open
39	-12.87	0.41	1.69	Open
40	-5.92	0.19	0.40	Open
41	0.27	0.01	0.00	Open
42	-4.27	0.24	0.89	Open
43	9.62	0.31	0.99	Open
44	9.61	0.31	0.98	Open
45	-64.76	0.52	1.15	Open
46	191.60	0.98	2.89	Open
49	42.00	0.59	2.10	Open
52	9.00	0.29	0.87	Open
53	8.00	0.25	0.70	Open
54	5.00	0.07	0.04	Open
55	40.40	0.57	1.95	Open
56	15.99	0.51	2.52	Open
57	5.00	0.16	0.29	Open
58	163.86	0.83	2.17	Open
59	75.99	0.60	1.55	Open
60	6.41	0.20	0.46	Open
61	10.82	0.34	1.23	Open
62	11.33	0.36	1.33	Open
64	63.99	0.51	1.13	Open
66	151.86	0.77	1.88	Open
67	1.08	0.06	0.07	Open
68	1.08	0.06	0.07	Open
69	1.53	0.05	0.03	Open
70	11.07	0.35	1.28	Open
71	10.62	0.34	1.18	Open
72	3.48	0.11	0.15	Open
73	-10.60	0.34	1.18	Open
74	-10.57	0.34	1.17	Open
75	-9.43	0.13	0.13	Open
76	3.75	0.05	0.03	Open
77	-23.51	0.19	0.18	Open
78	-110.17	0.56	1.04	Open
79	32.25	0.46	1.30	Open
80	10.99	0.35	1.26	Open
81	-15.51	0.49	2.38	Open
82	-8.75	0.12	0.11	Open
83	8.00	0.11	0.10	Open
84	8.00	0.11	0.10	Open
85	16.62	0.34	0.91	Open
86	5.43	0.31	1.39	Open
87	-2.86	0.09	0.10	Open
88	-1.14	0.06	0.08	Open

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
89	5.05	0.29	1.21	Open
90	5.95	0.34	1.64	Open
91	-2.56	0.08	0.09	Open
92	151.00	0.77	1.86	Open
93	128.00	1.02	4.07	Open
94	83.00	1.17	7.39	Open
95	34.00	0.69	3.44	Open
96	60.00	0.85	4.06	Open
97	165.25	1.32	6.52	Open
98	217.99	1.11	3.68	Open
99	-124.52	0.63	1.30	Open
100	90.25	0.72	2.12	Open
101	14.74	0.21	0.30	Open
102	157.75	0.80	2.01	Open
103	69.38	0.55	1.30	Open
104	38.03	0.54	1.74	Open
105	64.78	0.52	1.15	Open
106	9.92	0.32	1.04	Open
107	-0.56	0.02	0.01	Open
108	20.19	0.64	3.89	Open
109	56.69	0.80	3.65	Open
110	12.34	0.39	1.56	Open
111	181.19	0.92	2.60	Open
112	130.79	0.67	1.43	Open
113	-21.69	0.31	0.62	Open
114	50.13	0.40	0.72	Open
115	1.69	0.02	0.01	Open
120	87.58	0.70	2.01	Open
121	-49.00	0.69	2.78	Open
122	18.94	1.07	14.03	Open
123	1.15	0.04	0.02	Open
124	12.21	0.39	1.53	Open
125	0.25	0.01	0.00	Open
126	36.81	0.52	1.65	Open
127	9.52	0.30	0.97	Open
128	-23.00	0.33	0.68	Open
129	-9.27	0.30	0.92	Open
130	2.27	0.13	0.28	Open
131	0.86	0.05	0.04	Open
132	-0.95	0.05	0.06	Open
133	8.19	0.46	2.97	Open
134	-58.01	0.82	3.81	Open
135	8.30	0.47	3.05	Open
136	568.55	2.90	21.68	Open
47	-1.26	0.02	0.00	Open
48	-47.50	0.67	2.63	Open
1	16.45	145.432379840.00		Open

Link Results at 1:00 Hrs: (continued)

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
V1	0.00	0.00	0.00	Closed Valve