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ADDIS ABABA UNIVERSITY SCHOOL OF GRADUATE STUDIES

ADDIS ABABA INSTITUTE OF TECHNOLOGY (AAiT)

Assessment of Water Loss in Water Supply Networks

(A Case of Debre Markos Town)

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 (Hydraulic Engineering)

By: Melaku Abebaw

Advisor Dr.-Ing. Geremew Sahilu

July, 2015

Assessment of water loss in water supply networks: in case of Debre Markos town

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This is to certify that the thesis prepared by Melaku Abebaw, entitled: Assessment of water loss in water supply networks in case of Debre Markos town in partial fulfillment of the requirement for the degree of Master of Science (Hydraulic Engineering) compiles with the regulations of the university and meets the accepted standards with respect to originality and quality.

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Declaration

I, the undersigned person, declare that thesis is my original work and that all sources of material used for this thesis have been duly acknowledged.

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Abstract

Reduction of non-revenue water is one of the major challenges facing many water utilities in Ethiopia in general and Debre Markos water supply system in particular. So, the study focuses on the assessment of water loss in water supply networks in Debre Markos town using statistical analysis, Water audit and water CAD software. A statistical analysis was applied to analyze the current water supply coverage of the entire town. Water audit software was used to analyze water loss components and the efficiency of the system was evaluated using different performance indicators .Water CAD6.5 software was also used to simulation the distribution water supply network. Discussions were made with Local experts' to support the quantitative analysis. From the result of the analysis, it was observed that the total water loss in Debre Markos water supply system is high reaches up to 40% of the system input volume and about 33% of the total system loss is real losses and 7% apparent losses. Besides, the average daily per capita water consumption of the town is 24 liter/person/day. In general, the low water supply coverage of the town was highly influenced by the availability of water. However, the main reasons for the high loss of water in Debre Markos water supply system are the present way of water network management with ad-hoc maintenance and insufficient financial resources of the utility. Thus, it is necessary to identify the losses encountered in the water supply system so as to take remedial actions in reducing the water loss more significantly.

Key words: assessment of water loss, water supply network, water supply coverage, non-revenue water, water audit software, water CAD, performance indicator, Debre Markos.

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Abbreviations and Acronyms

ALC	Active Leakage Control
AWWA	American Water Work Associations
BABE	Burst and Background Estimate
CARL	Current Annual Real Losses
CAS	Census Statistical Abstract
DMA	District Meter Area
ELL	Economic Leakage Level
FAVAD	Fixed Area and Variable Area Discharge
GIS	Geographical Information System
ICF	Infrastructure Correction Factor
ILI	Infrastructure Leakage Index
IWA	International Water Association
LIS	Land Information System
MNF	Minimum Night Flow
NRW	Non-Revenue water
PRV	Pressure Reducing Valve
RL	Real Loss
TLF	Target Loss Factor
TMRL	Target Minimum Real Loss
UARL	Unavoidable Annual Real Loss
UFW	Unaccounted For Water
WDM	Water Demand Management
WHO	World Health Organization

CHAPTER ONE

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 Africa urban areas will be accessed for improved water within 15 years from the years 2000.On the other hand, in Africa largest cities, only 43% inhabitants have house connection water supply services (Wolday, 2005).The main problem that developing countries are faced to provide access to safe water for their citizens is shortage of resources. Moreover, the capacity of the citizens to play 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 rehabilitating the existing aged pipes .Generally, tariffs in developing countries are 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 practical, purposes. Limited institutional capacity is also one of the bottlenecks that hinder cities of developing countries for managing the infrastructure asset in general and water supply in practical. Besides, to low coverage, water loss (physical loss) in urban water supply is accounted to more than 50% the supplies that mainly arise from;

- ✓ Leakage of pipes, joints and valves
- ✓ Over flowing service reservoirs and,
- ✓ Wastage of water thought illegal connection and
- ✓ Unmetered house connections

Although leakage is one the major causes for loss of water in a network 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 continuous and monitoring service, the developing cities have great difficulties even to collect information on the 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 (Woldey, 2005).

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 Water CAD based hydraulic models because of low pressures and pipes without water.

Water CAD 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 Debre Markos town 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 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 et al.,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 Water CAD 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 Water CAD model was use.

1.2 Statement of the problem

Water Loss or Non-Revenue Water (NRW) represents inefficiency in water delivery and measurement operations in transmission and distribution networks and, for some systems, can amount to a sizeable proportion of total water production. The Water Losses for a whole system or for a partial system are calculated as the difference of Systems Input Volume and Authorized Consumption.

Water loss occurs on all the systems, it is only the volume that varies and it reflects the ability of a utility to manage its network.

To understand the reasons why, how and where water is being lost managers have to carry out an appraisal of the physical characteristics of the network and the current operational practice.

In many instances the problem of water loss is caused by poor infrastructure, bad management practice, network characteristics, operational practices, technologies, skills and social and cultural influences.

A high level of real or physical loss reduces the amount of precious water reaching customers, increases the operating costs of the utility and makes capital investments in new resource schemes larger. A high level of apparent or commercial losses reduces the principal revenue stream to the utility. Components of water loss or Non Revenue Water (NRW) are real losses or physical losses. Real (physical) losses are: reported and unreported bursts on pipes, background leakage on pipes and fittings, and Leakage and overflows from service reservoirs. Apparent (commercial) losses are errors on source and production meters, Errors on customer’s meters, unauthorized use i.e. illegal connections and theft.

The volume of water lost through physical leakage depends on the condition of the infrastructure and the leak detection and repair policy of the particular utility. The factors that affect the amount of water lost are pressure in the system, frequency of bursts and their flow rates, length of time the leak runs before it is located and repaired, Level of undetectable small leaks (background losses). The level of apparent losses depends upon: Utility's customer meter change policy and Utility's law enforcement policy for dealing with unauthorized use.

Based on DARAL-PMRAN Consultant (2014) study result indicated that the total loss of water in Addis Ababa city has 54,094,795 m^3,out of the total supply to the system 120,088, 391 m^3 and the loss is estimated to be 45.05% of the total sup plied to system. Asmelash (2000) estimated that the total water loss in Axum town has 290,148 m^3 out of the total supply to the system 113,448 m^3 estimated to be 39.1% the system input volume. Although the total loss of water can be easily estimated by comparing billing on water consumption and the total water produced and distribution to the network system, there have been inadequate studies on identifying where and how much water is lost and what are main the cause of water loss in many intermediate towns including Debre-Markos. Thus, this study was contributed to some highlight on the issue of water loss in supply network at Debre-Markos town, Amhara national state, Ethiopia.

1.3 Research Objective

1.3.1 Main Objective

The main objective of this research is to assess water loss in water supply networks in the case of Debre Markos town.

1.3.2 Specific objectives:

- To evaluate the domestic water supply coverage and distribution ,
- To evaluate the total loss of water (unaccounted for water) at the town level,
- To explore the possible causes of water losses and possible solution ,and
- To investigate the pressure distribution system with modeling.

1.3.3 Research Questions

The general and specific objectives of the study would be achieved by way of seeking answers to the following questions.

Specific objectives:	Research Questions
❖ To evaluate the domestic water supply coverage and distribution	✓ How much water do domestic consumers consume? ✓ What is level of water connection?
❖ To evaluate the total loss of water (unaccounted for water) at the town level	✓ How much water is produced and distributed to the network system? ✓ How much water is lost in the entire town while compared with the water produced?
❖ To explore the possible causes of water losses and possible solution	✓ What are the possible causes of water losses in the water distribution system? ✓ What are the possible solutions to reduce the
❖ To investigate the distribution system with modeling	✓ How to analyze the distribution system with mode?

1.4 Thesis Outline

This thesis comprises five chapters, which are organized as follows.

- ✓ Chapter one: Contains general background, the problem statement, the research objective, research question.
- ✓ Chapter two: Discussion literature related to water loss and leakage
- ✓ Chapter three: Discussion about the methodology the data collection and preparation
- ✓ Chapter four: Results and discussion
- ✓ Chapter five: Conclusion and recommendation.

1.5 Existing Water Supply and distribution system

Debre Markos water supply scheme was constructed in 1979 funded by German development bank. The source of water supply for the town is groundwater. In 1979 seven boreholes in

Sentera well field were drilled to the west and southwest about 12 km of from the town. Currently only five of them are giving service and current yield of these wells is 1, 342.80 m3/day.

In 2010 additional ten boreholes were drilled in the southwest of the town and out of these seven boreholes are giving service and current yield of these wells is about 1,440 m3/day.

The existing water supply network includes the Sentera D250/ D200 pipe transmission mains; DN50/150 distribution main and DN 15/32 service connection. An addition new water supply network in the Wotern D250//D500 transmission mains , DN50/450 distribution main and DN 15/32 service connection and the total of four concrete service reservoirs placed throughout the town. The total length of distribution pipelines installed by the town WSS was 57.72 Kilometer up to date. The layout of the existing and new water supply source and distribution network is shown figure1.2.

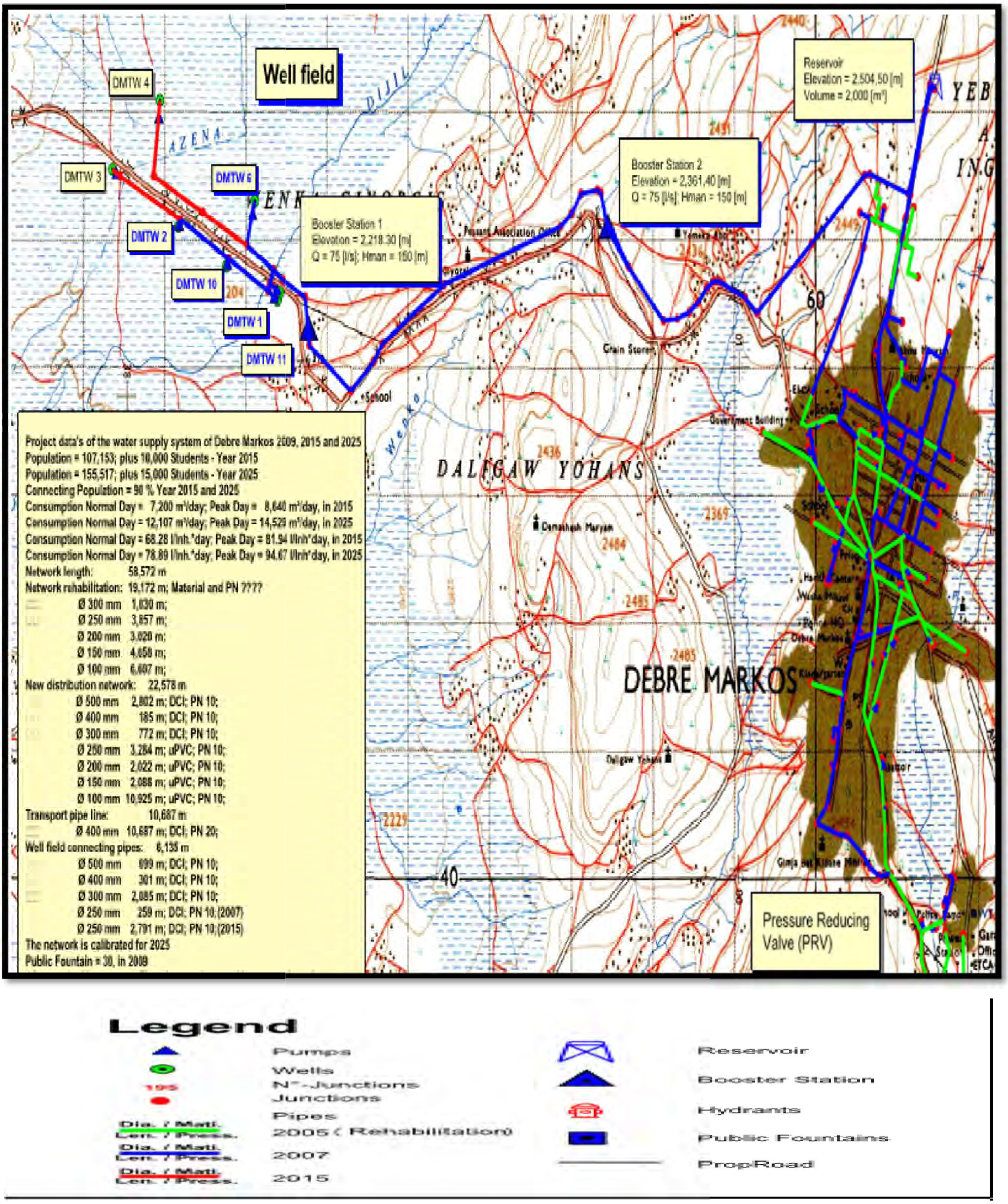


Figure 1.2: Existing and newly water supply network of Debre Markos

1.6 Water Demand and Consumption

One of the difficulties faced by the water service office is determining the accurate water demand if the town as the consumption during the past years that have been used as a base is far below the a actual demand due to shortage of water . Consumption of water for town is therefore estimated based on the amount supplied rather than the actual demand .For these reason estimates of the future demand by the water service office is found to be uncertain. Keeping this in mind, the current situation as summarized by the water service is as below (Debre Markos, 2014):

People having in-house service that are estimated about 25% of the total population use water on average between 40 and 60 litter per capita per day, while the remaining population with access to safe drinking water (75%) are served by yard connection and between 15 and 30 litter /capita/day.

CHAPTER TWO

LITERATURE RVIEW

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 supple projects and one of the difficulties among two 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 asse5t increases the level of water losses in water supply. As this research deals with over all coverage of water supply and water losses in distribution systems, issues related to water loss and leakage like identifying and reducing was reviewed in this chapter.

2.2 Urban Water Demand and Coverage

2.2.1 Urban Water Coverage

Water supply coverage provides a picture of the water supply situation of one specific country or city and helps to compare one country with others and the inter and intra city distribution with in specific country. The percentages of population with or without piped water connection are a relevant indictor to compare the coverage of water supply in urban areas. Although the water supply coverage is better in urban areas while compared with the rural, the actual water supply coverage in cities of developing countries in general and African cities in particular is very low while compared to the demand. According to the Global Water Supply and Sanitation Assessment 2000 Report, the African capital cities are having 43% house connection or yard tap, 21% served by public tap while 31% of the population are un-served (WHO,2000).

A household is considered to have access to improved drinking water if it has sufficient amount of water (20 liters/person/day) for family use, at an affordable price (less than 10% of the total household income), available to household members without being subject to extreme effort (less than one hour) a day for the minimum sufficient quantity), especially to women and children) (UN-Habitat, 2003).

On the other hand a minimum quantity of 25 liters of potable water per person per day provided at a minimum flow rate of not less than 10 liters per minute with the source being available within 200 meters form a household and the supply not interrupted for more than seven days per year (i.e. water should be available 98% of the time) is considered as a basic service for southern African cities' domestic water supply (Wolday, 2005).

2.2.2 Water Demand Management

Water demand is defined as the volume of water requested by users to satisfy their needs. In a simplified way it is often considered equal to water consumption, although conceptually the two terms do not have the same meaning (Walling ford HR.2003). In most developing countries, the theoretical water demand considerably exceeds the actual consumptive water use. Water demand management refers to any socially beneficial action that reduces average or peak water withdrawals or consumption form either surface or ground water, consistent with the protection or enhancement of water quality (Tate, 2000). According to Rothert and Macy (2000), water demand management is the adaptation and implementation of a strategy by a institution to influence the water demand and usage in order to meet any of the following objectives: economic efficiency, social development, social equity (Mwendera et al., 2003).

Urban water demand is classified in to different category that domestic water demands that included in-house-use and out-of -house-use is among the others. In-house-use includes demands for cooking sanitation, house cleaning, laundry and car washing while out-of-house-use includes like garden watering, swimming pools, public stand pipes for public uses and fountains, etc.

Urban water demand is usually quoted in terms of liter per capital per day (1/capita/day). Despite the variation in residential indoor water use from household to household, a typical pattern (referred to as the water use profile) can be developed to provide a reasonable representation of indoor water use, based on the different indoor water use components (kitchen, bathroom, laundry, and toilet) and household occupancy (Mitchell et al, 2000). In many African cities urban water demands are often homogeneous owing to a range of levels of services occurring within the same urban area. Levels of service can vary from household connections to standpipes or to no service at all (Wolday, 2005.)

2.3 Water loss and leakage

Regardless of the magnitude that greatly varies from city to city or from one area to another, water loss is problem experienced in all water distribution systems. The first and foremost cause of water loss is leakage. Water put to inappropriate or excessive uses may also be considered as loss. Water that is unaccounted for because of measurement errors, including inaccurate meters, forgotten users, and unmeasured uses, are also some of the causes for evaluating the waterloos that is usually defined differently by different writers that some of the definitions indicated here under.

2.3.1 Some Definitions of Unaccounted-for water (UFW)

There is no universally applied or accepted definition of unaccounted-for water. In general, Unaccounted-for water (UFW) is the difference between the water supplied to a distribution system and the water that leaves the system through its intended use (Richard G.et al.,2000) UFW may be defined as percentage of the water-produced from the raw water source which is not accounted for (MWAC, 1999). UFW is defined as the difference between water delivered to the distribution system and water sold (Yepes, 1999).

The term Unaccounted-for Water (UFW) refers to an accumulated range of losses that will be experienced by water Utility when comparing the system demand of a hydraulic water network with the quantity of water that is acknowledged as consumed by the water consumers residing within the network (UNEP, 2000).

Although the above definitions seem to have differences, all have in common that they took the water produced and distributed to the system as an input and the water consumed or exported from the distribution system as an output.

From the local context, the UFW has been defined as the water loss calculated as the difference between the amount of treated water produced and supplied and the total amount of water billed and collected. The volume of water consumption due to the inaccuracy of the water meters as well as the lump sum payments made by the customers when their meters cannot be repaired are also taken in to account for the determination of the UFW (AAWSA,1997). On the other hand because of the widely varying interpretation of the term 'Unaccounted-for UFW) worldwide, the IWA* task forces do not recommend use of this terms. If the term UFW is used at all, it should be defined and calculated in the same way as 'non-revenue water' (NRW) (Farley and Trow, 2003).

2.4 Methods of Measuring and Comparing Water Losses

2.4.1 Measuring Water Losses

The unaccounted for water (UFW) expressed as percentage of the total consumption and the minimum night flow (MNF) per connection are the most commonly used methods of measuring losses. UFW is the measure of losses over a period as the difference between the amount of water put in to a system and the metered or estimated quantity of water taken by consumers, while MNF is an indicator of the probable rate of losses at a given time.

Night flow measured in moderately sized sectors (up to around 3000 service connections) are extremely useful for identifying the presence of existing unreported leaks and bursts, and the occurrence of new ones. However, continuous night flows can also be used for assessing annual average real losses (Farley and Trow, 2003).

Unaccounted for water is a useful indicator of probable losses, but it may overestimate them because supply meters tend to under-record consumption. In UK, figures for unaccounted for water tend to be unreliable because the un-metered consumptions have to be estimated and can be 10% in error. Attempts to compare the performance of different undertakings by measuring some uniform figure for domestic consumption can be misleading. Many factors influence unaccounted for water and differ from one undertaking to another , standards of housing, rates of occupancy, age of mains, length of mains per 1000 population served proportion of trade and bulk supplies, ground condition, etc.(Twort A.C.et al., 1994)

The minimum night flow (MNF) per property connection is a better indicator of loss rates on part of a system. However, figures of this type are affected by the characteristics of an area; in dense urban areas there will be more blocks of flats with large storages which may fill at night.

Nevertheless, the MNF is a good direct indicator of the state of parts of a system (Twort A.C.et al., 1994).

On the other hand, Weimer referring to fully metered situations, considers that the annual water balance can initially only be taken as a guide as the calculations are susceptible to errors, analyses show this uncertainty in the calculated annual losses to be +/-46% (Lambert and Wallace, 1993).

Different countries use different methodologies to evaluate the losses like the U.K. leakage practitioners and planners consider leakage almost exclusively in terms of night flow rates, rather than as a calculation of annual losses as in West Germany. Each method has its respective merits. 'Annual losses' are used for retrospective assessment of overall performance and long-term demand forecasting. 'Night flows' are used by practitioners responsible for leakage control and prioritization of leakage control activities. Any conceptual model therefore needs to be able to link night flows with annual losses in a consistent manner (Lambert and Wallace, 1993).

Although percentage figure are rearly meaningfull when comparing different organizations ,they can be used to indicate the extents of reduction of water loss by a single water supplier (WHO,2001).

2.4.1.1 Estimating water loss from discovered leaks

Losses from leaks that are discovered and repaired should be measured to determine the rate of loss and the total volume lost during the life of the leak. Three methods are suggested (from ‘’Leak Detection Productivity‘’) by Douglas S.Greeley (AWWA California Nevada section,1992).

- Use a container of known volume.
- Use a hose and a meter.
- Calculate losses using modified orifice and friction formula.

The first method, sometimes known as the bucket and stop watch method is as simple as its name. Hold a container against the leak for a predetermined time. Time is recorded by a stopwatch. Measures the water captured with a measuring cup or other container of known volume and then converts time and volume to l/min.

The second method requires connecting a hose to the leak and directing the flow through a meter. The third method is the simplest to perform in the field but requires calculation. This method is often helpful for large leaks where the flow is too great to measure and the main must be valve off. It requires that the size and shape of the hole shall be measured and the line pressure will be determined. A pressure gauge or a hand held blade pedometer could be used to determine the pressure of the water coming from the leak or a nearby fire hydrant. This method also uses some assumptions regarding the shape of the hole that may introduce error.

For losses from such items as a pipes or broken taps, Greeley assumes an orifice coefficient of 0.80 and calculates flow in gallons per minute from the formula:

$$Q = \left(\frac{43767}{1400}\right) \times A \times \sqrt{P}$$ ------------ Equation 1 estimation of leak from transmission mains

Where, Q= flow in gallons per minute

A= the cross sectional area of the leak in square inches and P= the pressure in pounds per square inch.

2.4.1.1.1 Calculating Leak Rates for circular holes in distribution mains

A leak loss for circular holes under different pressure is estimated by Douglas S. Greeley formula as:

$Q = (30{,}394) \times A \times \sqrt{P}$ ---------Equation 2 estimation of leaks from circular holes

Where ''A" is the cross sectional area of the leak in square inches and p is the pressure in pounds per square inch.

2.4.1.1.2 Leaks Losses for joints and cracks under different pressure

For leaks emitted from joints and cracked service pipes an orifice coefficient of 0.60 is used in the following equation

$Q = (22.796) \times A \times \sqrt{P}$…… Equation 3 estimation of leaks from Cracks and Joints

Where, "A" is the area in square inches and "P" is the pressure in pounds per square inch

2.4.1.1.3 Comparing Water Losses

The amount of water loss differs from country to country, city-to-city and even from net work to another network within one city. Different countries use different indicators to evaluate their status in comparison with other and to compare the distribution system in order to take action based on the level of losses. As stated above comparison using UFW expressed as a percentage has limitation when used for comparison as it highly depends with the volume of the 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 the main local influences. Consequently it cannot be considered to be an appropriate performance indicator (PI) for comparisons (who, 2001). Depending upon the consumption per service connection, the same volume of real losses/services connection/day, in percentage terms, is anything from 44% to 2.4%.Thus countries with relatively low consumption like the developing countries, can appear to have high losses when expressed in percentage terms in contrast percentage losses for urban areas in developed countries with high consumption can be equally misleading (Farley and Trow, 2003).To avoid for the wide diversity of formats and definitions related to water loss ,many practitioners 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 figure 2.1 below.

System Input Volume	Authorized Consumption	Billed Authorized Consumption	Billed Metered Consumption	Revenue Water
			Billed Unmetered Consumption	
		Unbilled Authorized Consumption	Unbilled Metered Consumption	Non-Revenue Water
			Unbilled Unmetered Consumption	
	Water Losses	Apparent Losses	Unauthorized Consumption	
			Metering Inaccuracies and Data Handling Errors	
		Real Losses	Leakage on Transmission and/or Distribution Mains	
			Leakage and Overflows at Utility's Storage Tanks	
			Leakage on Service Connections up to Point of Customer Metering	

Figure 2.1: IWA standard international water balance and terminology

Source, Farley and Stuart, 2003

Water utilities around the world have always established water balances but unfortunately a wide diversity of formats and definition is used, often with in the same country so it was (and still is) virtually impossible to compare UFW, NRW leakage or water losses of different utility. Being aware of the problem of different water balance formats and methods, IWA established standard water balance as shown in table 1 above. According to IWA the above abbreviated terminologies are defined as below:

- ✓ System input volume is the annual volume input to that part of the water supply system
- ✓ Authorized consumption is the annual volume of metered and/or non metered water taken by registered customers, the water supplier and other who are explicitly or implicitly authorized to do so. It includes water exported, and leaks 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 systems in put volume and authorize consumption, and consist of apparent losses and real losses.
- ✓ Apparent losses consist of unauthorized consumption and all types of metering in accuracies
- ✓ Real losses are the annual volumes lost through all types of leaks, bursts and over flows on mains service reservoirs and service connection up to the point customer metering.

Accordingly the quantity in table above can quantified as explained in the following steps (Roland Liemberger, Malcolm Farley, 2005).

Step 1 Determining system input volume

When the entire system input as metered, the calculation of the annual system input should be a street forward task. Ideally, the accuracy of the input methods is verified using portable flow measuring device. If discrepancies between meter readings and the temporary measurements are discovered, the problem has to be investigated and if necessary the recorded quantity as to be adjusted to reflect the real situation. Should there be some unmetered source the annual flow has to be estimated by using any (or a combination) of the following: (i) temporary flow measurements using portable devices, (ii) reservoir drop test or (iii) analysis of pump curves, pressure and average pumping hours.

Step 2 Determining Authorized Consumption

Billed Metered Consumption

The calculation of the annual billed metered consumption goes hand in hand with the detection of possible billing and data handling error, information later on require for the estimation of apparent losses. Consumption of the different consumer categories (e.g. domestic, commercial, industrial) have to be extracted from utility billing system and analyzed. Special attention shall be paid to the group of very large consumers.

Billed Unmetered Consumption

Billed unmetered consumption can be obtained from the utilities billing system in order to analyze the accuracy of the estimate; unmetered domestic customers should be identified and

Monitored for a certain period, for example by measuring a small area with a no- of unmetered customers

Unbilled Metered Consumption

The volume of unbilled metered consumption has to be established similar to that of billed metered consumption.

Unbilled Unmetered Consumption

Unbilled Unmetered consumption traditionally including water used by the utility operational purpose is very of often seriously over estimated. This might be caused by simplifications (a certain % of total system input) or overestimates on purpose to reduce water losses, components of unbilled unmetered consumption shall be identified and individually estimated, for example l:

- Mains flushing: how many times per month? How long? How much water?
- Fire fighting: has there been a big fire? How much water was used?

2.4.3 Quantifying Real and Apparent Losses

Once the volume of NRW is known, it is necessary to break, it down in to real and apparent losses, which is always a difficult task**.**

Step 3 Estimating Apparent Losses

Unauthorized Consumption

It is difficult to provide guide line of how to estimate unauthorized .The estimation of Consumption is always a difficult task and should be done in a transparent, component based way so that the consumptions can later be reviewed.

Customer Metering Inaccuracies and Data Handling Errors

The extent of customer meters inaccuracies namely under or over registration has to be established based on tests of a representative sample of meters. The composition of the sample shall reflect the various brands and age groups of domestic meters based on the results of the accuracy tests, average meter inaccuracy values (as percentage of metered consumption) will be established for different user groups. Data handling errors are sometimes very substantial components of apparent losses.

Step 4 Calculating Real Losses

The Calculation of real losses in its simplest form is now easy Volume of NRW minus volume of apparent losses-and this figure is useful for the start of the analysis in order to get a felling which magnitude of real losses can be expected. However, it always has to be kept in mind that the water balance might have errors and therefore it is important to verity the real loss figure by one of the following two methodologies (i) component analysis and (ii) Bottom –up real loss assessment.

Step 5 Estimating Real Loss Components

Accurate split of real losses into its components will only be possible with a detailed component analysis However; a first estimate can be made using a few basic estimates.

Leakage on transmission and /or distribution mains

Bursts on distribution and especially transmission mains are primarily large events- they are visible reported and normally repaired quickly. By using data from the repair records the number of leaks on mains repaired during the reporting period can be calculated an average flow rate estimated and the total annual volume of leakage form mains calculated as follows:- number of reported bursts x average leak flow rate x average leak during (say 2 days) and then a certain provision for background losses and so far undetected leaks on mains can be added .

Leakage and Overflows at Utility's Storage Tanks

Leakage and Overflows at Storage Tanks are usually known and can be quantified.

Leakage on Service Connections up to Point of Customer Metering

By deducting mains leakage and storage tank leakage from the total volume of real losses, the approximate quantity of service connection leakage can be calculated, this volume of leakage includes reported and repaired service connection leaks as well as hidden (so far unknown) leaks and background losses from service connections.

2.4.4 Detailed Quantification of Real Loss Components

Step 1 the Figure form the Top down Water Balance

Although real loss assessment can be done without an annual water balance the total volume of real losses is useful for the start of the analysis in order to get a feeling which magnitude of real losses can be expected.

Step 2 Component Analysis

The key data required for a real loss component analysis of a water distribution system are:

- Total length of pipe network and number of service connections
- Average service connection length between curb-stop and customer meter
- Total number of distribution mains repairs per year (reported and unreported)
- Total number of service connection repairs per year (reported and unreported)
- Average system pressure across the entire network
- Estimates of the time periods for awareness, location and repair duration
- Estimates of utility storage tank leaks and overflows

Most of this data is readily available in well organized water utilities: however the determination of the average pressure across the network is often difficult to estimate.

Calculation of Average Pressure

As the average pressure is a key parameter in any real loss analysis it is certainly worth undertaking some detailed work to obtain a good estimate of the average pressure. Pressures should be calculated as 24 hours averages values.

Calculation of Background Losses

The first of the real loss components calculated are the background losses. Background losses are individual events (small leaks and weeps) that will continue to flow, with flow rates too low to be detected by an active leakage control campaign either unless detected by chance or utile they gradually worsen to the point that they can be detected. Table 2 provides for unavoidable background leakage rates per PSI of pressure at an ICF1 (infrastructure correction Factor) of 1 .

Table 2.1: Unavoidable Background Leakages

Infrastructure Component	Background leakage at ICF=1.0	Units
Mains	9.6	Liters per km of mains per day per meter of pressure
Service connection main to property boundary	0.6	Liters per service connection per day per meter of pressure
Service connection-property boundary to customer meter	16.0	Liters per km of service connection per day per meter of pressure

Source: IWA water loss task force, 2003

Unfortunately the ICF is a mostly unknown factor, without carrying our detailed measurements, it is impossible to know the ICF. In such cases working with the default values of one will means that there is a good chance that the background losses are underestimated and consequently the recoverable losses are overestimated. Using a higher ICF (of say 5) might easily lead to an overestimation of the background losses that will cause an underestimation of the true excess loss reeducation potential. Thus, it is recommended to work with the ICF=1 background leakage values unless better data is available.

Calculation of Losses form Reported and unreported Burst

At this Point two definitions have to be introduced

Reported Bursts are those events that are brought to the attention of the water utility conditions, manifests itself at the surface will normally be reported to the water utility.

Unreported Bursts are those that are located by leak detection teams as part of their normal everyday active leakage control duties. After collecting the annual numbers of reported bursts on mains and service connections, flow rates and durations have to be established, unless the utility has investigated average leak flow rates, it is recommended to use the figures form table 2.2 below (Lambert, A and Lalonde,2005).

Table 2.2: Average Leakage Rate

Location of Burst	Flow rate for reported bursts (1/hour/m Pressure)	Flow rate for unreported bursts (1/hour/m pressure)
Mains	240	120
Service connection	32	32

Source: IWA water loss task force, 2003

The leak duration can be split in three elements time needed for (i) awareness, (ii) location and (iii) repair and estimates will have to be made for each of them.

Awareness duration: the awareness duration for reported bursts is generally very short, probably not more than 24 hours. The situation is quite different in respect to unreported bursts, which by definition are detected by active leakage control methods, the awareness time will depends on the active leakage control policy, if for example regular sounding is used and the system is surveyed once a year, the average awareness time will be 183 days.

Location duration: - the location of a reported leak will in general not take much time since it is visible and a quick check with a ground microphone will be sufficient to verify the leak location, the location duration also depends on the active leakage control policy used.

Repair Duration: - depends on the utility's repair policy and capacity. Often leaks on mains are repaired within 24 hours but small leaks on service connections within 7 days.

Calculation of losses from leaking and overflowing storage tanks

This component has to be dealt with on a case-by -case. Plant operators will normally know if there are problems with overflowing storage tanks, old underground storage tanks may leak, and if this is suspected than level, drop tests could be undertaken. Calculation of excess losses once all the components mentioned above are quantified, the excess losses can be calculated.

(Real losses) - (known real loss components) = (excess losses)

In thesis case this equation results in a negative value for excess losses, the assumptions for the real loss components analysis have to be checked and if necessary corrected.

Step 3 Bottom up Real Loss Assessment

24 Hours Zone Measurement

Assuming that no district meter areas (DMA) are established, areas of the distribution network have to be selected which can be temporarily isolated and supplied from one to two inflow points only, suitable areas shall be selected in various parts of the distribution system with the objective of obtaining a representative sample of the system. In these area 24-hours inflow measurements will be carried out with portable flow measurement device, these flow measurements shall always be done along with pressure measurements where pressures are recorded at the zone inlet point(s) at the average pressure point and at the critical pressure point, all relevant data on the zone shall be collected, such as (i) length of mains, (ii) number of service connections, (iii) number of household properties and (iv) number and types of non-household properties.

Night flow Analysis

The minimum night flow (MNF) in urban situations normally occurs during the early morning period usually between around 02:00 and 04:00 hours, the estimation of the real loss components at minimum night flow is carried out by subtraction an assessed amount of legitimate nigh consumption for each of the customers connected to the mains in the zone being studied. The result obtained from subtracting these legitimate night uses form the minimum night flow consists predominantly of real losses from the distribution network. The daily level of real losses obtained from the minimum night flow analysis can be determined by applying the FAVAD principles (lambert,2001) and simulating leakage over the full 24 hours period.

2.4.5 Calculating Real Loss Performance Indicators

Since high level of water losses, both real and apparent is a very important efficiency issue, one would assume that accurate performance indicators are used for benchmarking, international performance comparison, or target setting. However, unfortunately this with widely not the case with the exception of the UK water industry, water losses are still quoted as percentage of system input, the serious problems with this indicator were highlighted in many conferences around the world, most recently at the IWA leakage conference in Cyprus (Liemberger, 2002). The new and most advanced real loss indicators (recommended by the IWA and the AWWA) are the ILI, the infrastructure leakage index. The ILI is a measure of how well a distribution network is managed for the control of real losses, at eh current operating pressure. It is the ration of current annual volume or real losses (CARL) to unavoidable annual real losses (UARL).

ILI = CARL /UARL

Being a ratio, the ILI has no units and thus facilitates comparison between countries that use different measurement units (U.S. metric or imperial). Nevertheless what are unavoidable losses and how are they calculated? Leakage management practitioner around the world are well aware that real losses will always exist- even in new and well managed systems, it is just a question of how high these unavoidable losses will be. The complex initial components of the UARL formula were converted to a user friendly pressure dependent format for practical use.

UARL (liters/day) =18xLm+0.8xNC+25xLP) x P

Where Lm= Mains length (km) NC= number of service connection Lp = total length of private pie, property boundary to customer meter (km), P = average pressure (m).

2.5 Causes of water losses

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

2.5.1 Leaks in Water Distribution Systems

Leakage is often a large source of unaccounted for water (UFW) and is a result of either lack of maintenance or failure to renew ageing systems. Leakage may also be caused for poor management of pressure zones, which result in pipe or pipe-join failure. Although some leakage may go unnoticed for a long time, detection of visible leakage also requires good reporting which also needs a strong public participation. Although leakages after water meter has its own contribution to the overall wastage of water, it is not considered as of the total unaccounted for water, as it would be paid for. It is important to distinguish between total water losses (sometimes called unaccounted for water (UFW) and leakage. Total water loss describes the difference between the amount of water produced and the amount which is billed or consumed.

Leakage is one of the components of total water lost in a network, and comprises the physical losses from pipes, joints and fittings and also from over flowing service reservoirs (WHO, 2001).

The amount of leakage from a reticulation system varies from location to location, due to differences in construction methods, age, and condition. The condition of the reticulation system is affected by soil movement, corrosive conditions, pipe material, workmanship, age, supply pressure, number of joints and connections, and the occurrence of bursts/cracks result from overburden loading or water hammer (Heeps,1977),(Mitchell et al., 2000). Leakage reduction as a whole is a complex task which requires coordinated actions in different areas of the water network management such as direct detection and repair of existing leaks, pipe rehabilitation program, pressure control system, etc. and many companies use a mixture of these. Many cities have separated the network into 'leakage districts', and have installed water flow and pressure meters to monitor each district. The registered data are checked and necessary actions taken. Data on bursts and leaks are collected and evaluated to estimate the future need of rehabilitation. During the last 10-20 years, several cities have started to use computer-based water network records. These databases contain information on network properties, such as pipe material, construction year and diameter and failure information (where, when, failure description, etc.).

By simple analyses of these data or by employing more complex statistical methods, information is collected to show differences in failure rate for different pipe properties (Hadzilacos and Kalles, 2000).

2.5.1.1 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 difficulty to assess and compare the demand and loss of water in its spatial distribution. Pressure distribution system on the one hand contributes to the increase of leakage, when it is more, and on the other hand when it is low 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 systems has been shown to be directly proportional to the square root of the distribution system pressure as indicated by the relationship (Wallingford HR, 2003).

$$L \alpha \sqrt{P} \text{……………………Equation 1 leakage and pressure relation}$$

Where: L=leakage in distribution system

P= distribution system pressure

Burst rates are also a function of pressure. The strength of the relationship, and the quantification of it, is not as well understood as the relationship between flow rate and pressure. However, there is still considerable evidence to show that burst frequency is very sensitive to pressure. Evidence shows that the rate of increase of bursts is more than linearly proportional to pressure. Indeed it has even been suggested that there could be a cubic relationship, i.e. burst frequency proportional to pressure cubed (Farley and Trow, 2003).

Pressure variation in distribution network is caused, among others, by changes of demand of users. The demand usually reaches a peak in the morning when people are at home and preparing their meal and its second peak in the evening.

If one compares daily diagram for total demand of the whole system with corresponding data captured at the level of (relatively small) demand management areas one will discover that the first has much smaller amplitude in comparison with the later. The minimum night flow (MNF) is relatively higher and the morning/evening peaks are less prominent (Obradovie, 2000). Frequent starts and stops of pumps, closure and openings of control valves that induce water hammer are also some of the causes to be mentioned for pipe breakage and water loss through leakage. The position of reservoirs also has a great impact on the pressure distribution.

'Distribution Losses" is the sum of losses from four different parts of the distribution system; trunk mains, service reservoirs, distribution mains and communication pipes. The combination of these assets in individual companies and supply areas are widely variable, as are the variations of pressure which are known to significantly affect leakage (Lambert and Wallace, 1993). The elevation at which it is desirable to position a service reservoir depends upon both the distance of the reservoir from the distribution area and the elevation of the highest buildings to be supplied.

If the distribution area varies widely in elevation it may be necessary to use two or more service reservoirs at different levels, so that the lower areas do not receive an unduly high pressure. Generally, 45 to 75 meters static pressure is that which best suits the domestic distribution systems. Pressure below 45 meters will be likely to cause trouble in supplying extensive distribution areas; pressure above 90 meters, tend to result in excessive leakage losses (Twort A.C et al., 1994).

The critical points which would first run dry if pressure is reduced are usually areas located at the highest elevation and excessive pressures can be reduced by adjusting the speed of pumps in areas supplied by pumping, installing a pressure reduction valves (PRV) and dividing the system in to different pressure zones. Pressure control valves are sometimes installed in outlet mains from service reservoirs in order to reduce the pressure to low lying zones, or to limit increases of pressure at night to reduce leakage. Pressure reducing valves (PRVs) throttle automatically to prevent the downstream hydraulic grade from exceeding set value, and are used in situations where high downstream pressures could cause damage (Walskiet al., 2003).

2.5.1.2 Ages of Pipes and Leakage

Although there are no scientifically based criteria for defining the useful life for water mains, there has been a growing concern that many older urban water distributions are deteriorating that as a result massive rehabilitation will be required to replace mains older than some predetermined number of years in age or "useful life".

Pipe age and material are important factors contributing to the burst probability of pipes that as a result cause lots of water loss. However, as this information is mostly not available especially for aged pipes, it is usually estimated using the history of the urban development. Reports from undertakings collected by the WRC, and evidence from elsewhere suggest that leakage rates from Mains are of the order of 100 to 2 00 l/hr per km for newer mains and 150 to 3001/hr per km for older mains. Assuming an average of 100 connections per km these figures would represent 1.0 to 3.0 1/hr per connection (Twort A.C. et al., 1994).

Leakage is frequently the largest component of UFW and includes distribution losses from supply pipes, distribution and trunk mains, services up to the meter, and tanks. The amount of leakage varies from system to system, but there is a general correlation between the age of a system and the amount of UFW. Newer systems may have as little as 5 percent leakages, while older systems may have 40 percent leakage or higher (Walski et al., 2003).

Although age is considered as an indicator for predicting the break rate of mains, some studies have shown that it is not the major determinant factor for main water break rates. Poor design, deterioration of pipe material and unanticipated load condition will also result in pipe breakage.

2.5.1.3 Effects of Corrosions on leakage

Corrosion is the problem that is created as water supply pipelines are in continuous contact with soil surrounding it and the water moving through it. The water itself or the surrounding soil may cause problems that will affect the performance and life of the distribution pipes in the system. The majority of the main breaks occur at locations where the pipe wall has been weakened due to corrosion of metal pipes. Corrosion of the external surfaces of cast-iron or steel pipes can, under some conditions, be a significant problem. Therefore, ductile-iron or steel pipelines placed in aggressive soils must be protected by coatings with corrosive resistant materials. The characteristics of the soil in which a pipe is placed affect the rates of corrosion.

Recent estimates indicate that the cost of water main breaks in Canada is about $80 million per year. One reason that this cost is so high is that most water mains in Canada are made from either cast or ductile iron. As these pipes age, they are weakened by corrosion, causing an increased number of breaks (IRC, 1996).

Designing against corrosion, selection of appropriate materials and usage of protective coating and lining during installation can help for the prevention of corrosion but not limited: Some soils exist in non-corrosive soils too. Soil conditions are responsible for the exterior corrosion of metal structures under or in contact with the ground (Wolday, 2005).

2.5.2 Meter error and water loss

Under registration of customer meters is also one of the causes of water loss. Like the ages of pipes, ages of meters also has an impact to the increase of water loss. Customer meter errors include errors due to accounting procedure and errors due to under or over registration of the meters. Many countries especially developing countries are experienced losses of water due to under registration of meter that many of them put meter replacement policies to alleviate the problem.

The selection of customer meter types and classes may be limited by water quality considerations, as well as technical and economic considerations, economic replacement policies for residential. Meters based on selective testing programs in the National Reports generally indicate changeover periods between 5 and 10 years. Where customers are served by way of roof tanks, the probability of customer meter under-registration is increased, because of the tendency for a greater part of the consumption to pass through the meter at rates less than the Q minimum specified for the meter (Lambert, 2003).

The cities of Africa appear to use meters for 78% of domestic consumption and the yearly meter replacement is about 8.8%. Considering that meters typically under read as they age, it is likely that considerable proportion of unaccounted for water is experienced by metering errors (WHO, 2000).

Domestic water meters tend to under register for two reasons, i) malfunctioning due to deterioration with use, and ii) inability to measure low flows accurately. Much larger under registration can occur when maintenance of meters is poor (Twort A.C. et al., 1994).

An under registering meters and any meter stoppage could be noted immediately if meter readers are alert to compare readings of on specific meter with its past readings, but in reality this situation doesn't happen.

2.6 Consequences of water loss and leakage

The primary consequence of leaks in a distribution system is financial. Reduction in water losses enables water utilities to use existing facilities efficiently, alleviate shortage of water supply, improving the supply capacity to consumers and the reduction of operational expenditures that are related to power and chemical costs. Reduction of water losses extends the service life of existing water supply components that as a result to meet the present as well as the future needs of residents without construction of any new water facilities. Beside to low revenue generation as a result of under-recording of faulty meters, or totally uncharged due to illegal connections and unregistered consumption leakage also greatly contributes to loss of revenue.

The operation and maintenance costs including price of energy, chemicals and other items that are constantly rising will also be aggravated by the increase of water loss due to leakage. Beside to directly affected production and management costs, leaks have great consequence on the quality of services. The water that escapes from leaks may also cause a damage of structures such as sinking of roads and other properties. When the leak becomes more serious or a pipe bursts, service may be interrupted totally that many people will be severely affected.

2.7 Leakage monitoring and control

The losses of water are inevitable in the process of supplying thousands of customers spread over a large area started from reservoirs at the treatment plants, through a complex network to the individual customer. Leakage monitoring and control in pipe reticulation systems is critical in ensuring the efficient performance of the system. Pipe systems are commonly used for distributing water to areas of consumption. If pipes are worn out, large volumes of treated water may be lost through leakage as a result of high pressures of flow. Leakage control is possibly one of the most difficult tasks for water engineers. Even in developed countries, about 15-20% of the distributed water is lost through pipe leakage. It is therefore important to ensure that leakage monitoring and control is given the attention it deserves by all water supply authorities and consumers (Mulwafu W. et al., 2003).

Water leak detection is a systematic method of locating visible and non-visible leaks in a distribution system through visual inspection; pipe locators and leak detection equipment (proactive leak detection); and pressure control, etc. Depending on the type, leakage could identify from the simplest of using visualization till using sophisticated equipments as discussed below.

2.7.1 Identifying Leaks through Visual inspection

In this method only those leaks that become self-evident are located and repaired. A leak may be self-evident because water shows on the surface or may become so upon investigation following consumer complaints such as poor pressure or noise in the plumbing system (Wallingford HR, 2003).

Bursts of large mains are often visible and are not considered as major causes of high water losses as these incidents are quickly spotted and repaired or isolated. This method is widely applied and requires regular inspection by the respective authority and it does not need special professional skills.

2.7.2 Identifying Leak using Detection Equipment

Most of the water is lost through numerous small holes, which are very difficult to locate, as the pipes are laid underground that usually need special equipments to locate the leak and repair. This method involves teams of inspectors seeking to locate leaks by systematic direct sounding on all stopcocks, hydrants and valves through the distribution system and listening for the characteristic noise of leaking water. As water under pressure exits a crack or a small hole, the pipe wall and the surrounding soil emit sound waves in the audible range. Water impacting the soil and circulating in a cavity creates lower frequency waves that have limited transmission through the ground. Through the use of surface microphones, leaks can be located with greater precision. The leak noise detected will depend upon the position at which a sounding is made (Wolday,2005).

2.7.3 Location of Large leaks by Pressure Control

A large leak in a small network can be located by measuring the pressure during the time of minimum water supply especially during the night. This can be done by shutting of the valves in successive sections of the distribution starting from the supply.

Pressure control does not directly involve leakage detection, but sudden drops in pressure may indicate to a possible leak. In general, reduction in pressure leads to reduced rate of escape through each leak and may also affect the number of leaks occurring. Pressure reduction is relatively cheap and can be quickly effected, but lower pressure may also increase the leak population by making them less detectable. Pressure reduction can be achieved in a number of ways such as reducing pumping heads, installing break pressure tanks and using pressure-reducing valves. The control of pressure surges and cycling is likely to reduce the numbers of bursts and leaks that occur, especially in plastic pipes. Pressure control is a necessary tool for the technical management of the system and combined with any other method of water loss estimation could give very useful information in order to identify the causes of water lost through leakage (Wolday, 2005).

Operational pressure control is a cost effective method for reducing leakage over whole sub systems, and for reducing the risk of further leaks by smoothing pressure variations. The planning studies under the assumption of perfect knowledge of demand and network model (including leakage) allow for the estimation of the maximum potential savings resulting from the optimal control of PRVs (Ulanicki and Bounds, 2000).demands such as hot dry summers cause increase lawn watering.

2.8 Water Distribution Network Building and Model Setup

The approach to building the model is to first sketch out the system practically on existing topographic maps. The concept of a network is fundamental to a water distribution model. The network contains all of the various components of the system, and defines how those elements are interconnected. Networks are comprised of nodes, which represent features at specific locations within the system, and links, which define relationships between nodes.

Water distribution models have many types of nodal elements, including junction nodes where pipes connect, storage tank and reservoir nodes, pump nodes, and control valve nodes. Models use link elements to describe the pipes connecting these nodes. In addition, elements such as valves and pumps are sometimes classified as links rather than nodes. Intelligent use of element labeling can make it much easier for users to query tabular displays of model data with filtering and sorting commands. Rather than starting pipe labeling at a random node, it is best to start from the water source and number outward along each pipeline. In addition, just as pipe elements were not laid randomly, a pipe-labeling scheme should be developed to reflect that.

2.8.1 Principles of Network Hydraulics

In networks of interconnected hydraulic elements, every element is influenced by each of its neighbors; the entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. Two basic equations that govern in Water CAD modeling network of these interconnections (Bentley Water CAD/GEMs , 2008).

- Conservation of mass or continuity principle.
- Conservation of energy or energy principle.

2.8.1.1 Conservation of Mass

For steady incompressible flow:

Net flow into junction = Use at junction

Net flow into junction = Use at junction.

Mass in = Mass out

$\sum Q_{IN}\,\Delta t = \sum(Q_{OUT}\,\Delta t + \Delta Vs)$-----------------(1)

Where: Q_{IN} = Total flow into the node (m^3/s, cfs)

Q_{OUT} = Total demand at the node (m^3/s, cfs)

ΔVs = Change in storage volume (m^3)

2.8.1.2 Conservation of Energy

The Energy equation is known as Bernoulli's equation (Daugherty.R.L, 1989). It consist the pressure head, elevation head, and velocity head. There may be also energy added to the system (such as by a pump), and energy removed from the system (due to friction). The changes in energy are referred to as head gains and head losses (Shaher,H,2003).

In hydraulics, energy is converted to energy per unit weight (ft-lb/lb) of water, reported in length units (ft) called "head". Balancing the energy across any two points in system, the energy equation will be as follow: Figure 2.2 shows head losses in a pipe line.

$$\frac{P}{r} + Z1 + \frac{V21}{2g} = \frac{P2}{r} + Z2 + \frac{V22}{2g} + hl \;--------(2)$$

Where: P = the pressure (Ib/ft^2or N/m2)

γ = the specific weight of the fluid (Ib/ft^3or N/m^3)

z = the elevation at the centroid (ft or m)

V = the fluid velocity (ft/s or m/s)

g = gravitational acceleration (ft/s^2 or m/ s^2)

hl = the combined head loss (ft or m)

There are three forms of energy for Hydraulic Network Modeling of Debre Markos Water supply networks.

- Pressure head -$\frac{P}{r}$
- Elevation head-Z
- Velocity head - V^2 / 2g

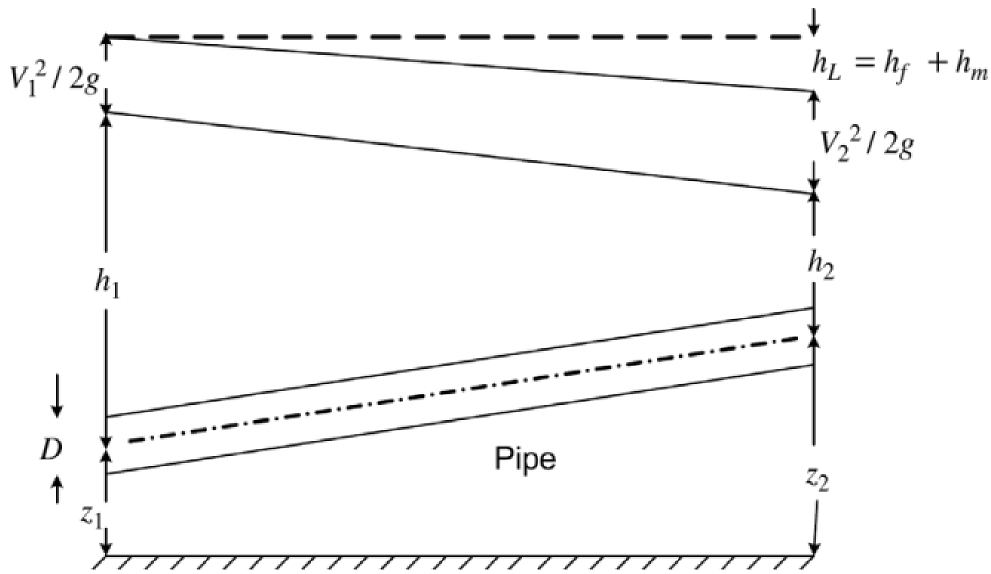


Figure 2.2: Forms of energy in water pipes

Source: (Bentley Water CAD/GEMs ,2008)

2.8.1.3 Water Flow Resistance (Head Loss)

The total water loss in a distribution pipe and pipe fittings between two points of consideration is called head loss. There two types of head losses.

2.8.1.3. 1 Surface Resistance

Head loss on the account of surface resistance, friction loss depends on:

- Pipe length.
- Coefficient of surface resistance, friction factor.

Surface resistance is categorized as major loss.

2.8.1.3.2 Form Resistance

The form-resistance losses are due to bends, elbows, valves, enlargers, reducers, and so forth categorized as minor loss.

2.8.1.3.3 Head Loss Equations

There are three main head loss equations. Head loss equations and their application area are shown in table 2.3.

- Darcy Weisbach
- Colebrook White
- Swamee Jain
- Hazen Williams
- Manning

Friction loss are estimated with Manning, Darcy- Weisbach and Hazen-Williams

Table 2.3: Head loss equations and their application area

Equation	Formula	Remarks
Manning's	$V = \frac{1}{n} R^{2/3} S^{1/2}$	This equation is commonly used for open channel flow.
Chezy's (Kutter's)	$V = C\sqrt{RS}$	Widely used in sanitary sewer design and analysis
Hazen-Williams	$V = 0.85CR^{0.63}S^{0.54}$	Commonly used in the design and analysis of pressure pipe systems
Darcy-Weisbach	$V = \sqrt{\frac{8g}{f} RS}$	Can be used for pressured pipe systems and open channel flows.

Source: (Bentley Water CAD/GEMs ,2008).

2.8.2 Water Distribution Modeling

2.8.2.1 Water CAD

Water CAD is a powerful, easy-to-use, which is:

- A water distribution modeling software;
- Used in the modeling and analysis of water distribution systems;
- Used for firefighting flow and constituent concentration analyses, energy consumption and capital cost management; and Popular for water supply design.

Water CAD provides sensitive access tool needed to model complex hydraulic situations. Some of the key features allow us to:

Perform steady state and extended period simulations.

- Analyze multiple time-variable demands at any junction node.
- Quickly identify operating inefficiencies in the system.
- Perform hydraulically equivalent network skeletonization including data scrubbing, branch trimming, and series and parallel pipe removal and efficiently manage large data sets and different "what if" situtions with database query and edite tools.

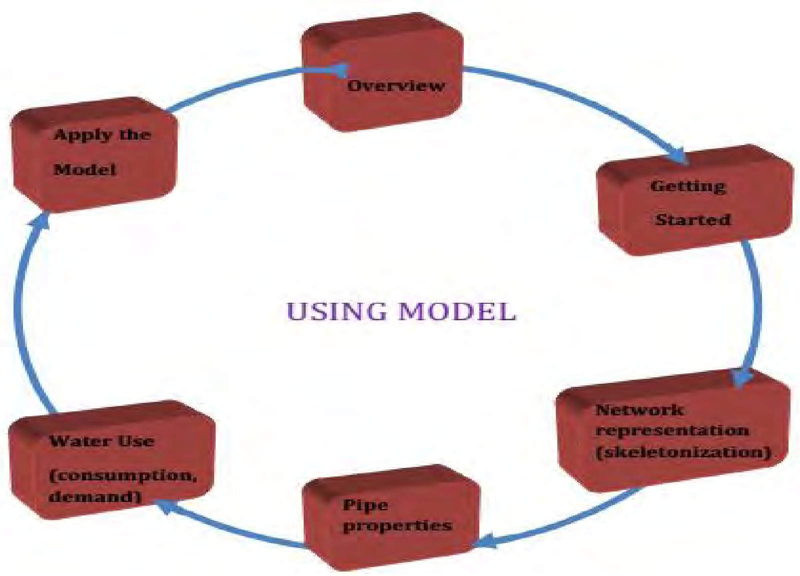


Figure 2.3: Diagrammatical representation modeling process

Source (Bentley Water CAD/GEMs, 2008).

2.8.2.2 Water Distribution Simulation

Simulation refers to the process of imitating the behavior of one system through the functions another. In our case, the term simulation refers to the process of using a mathematical representation or real system, called a model (Bentley, 2008).

Simulation can be used to predict system responses to under a wide range of conditions without disrupting the actual system, and solutions can be evaluated before time, money, and materials are invested in a real-world project.

There are two most basic types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. These are:

- Steady state simulation.
- Extended period simulation (EPS).

2.8.2.3 Steady State Simulation

It computes the state of the system (flows, pressures, pump operating attributes, valve position, and so on) assuming that hydraulic demands and boundary conditions do not change with respect to time.

A steady-state simulation provides information regarding the equilibrium flows, pressures, and other variables defining the state of the network for a unique set of hydraulic demands and boundary conditions.

Steady-state models are generally used to analyze specific worst-case conditions such as peak demand times, fire protection usage, and system component failures in which the effects of time are not particularly significant.

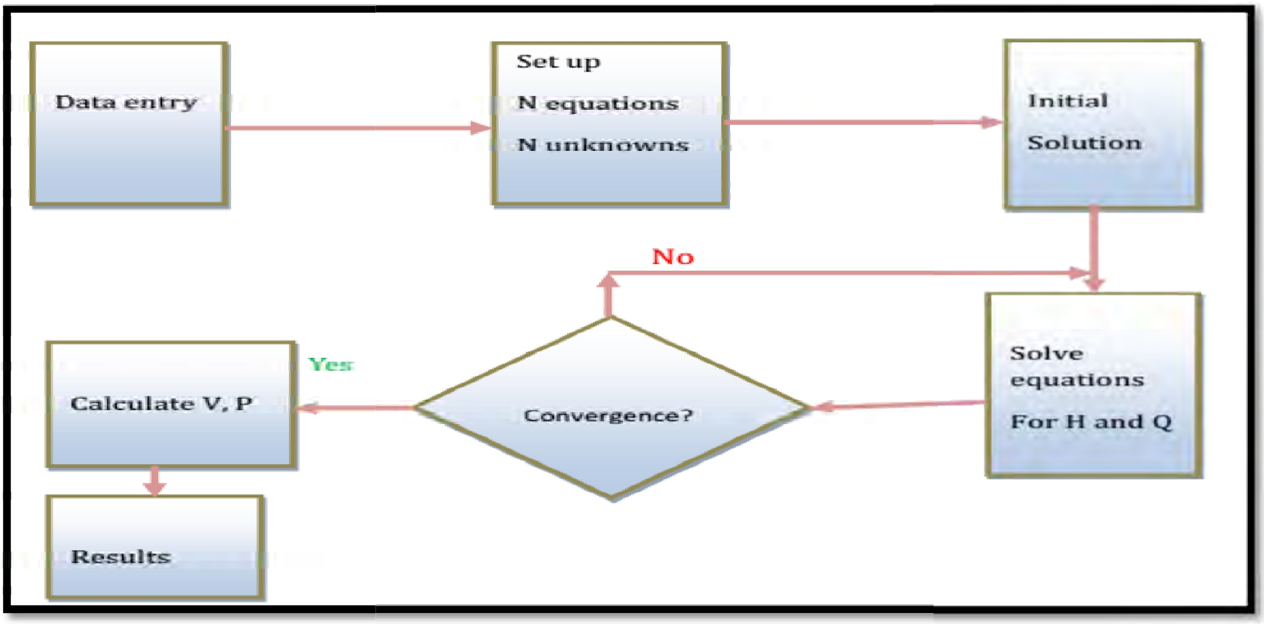


Figure 2.4: Flow chart for steady state simulation

Source: (Bentley Water CAD/GEMs ,2008)

2.8.2.4 Extended Period Simulation

Extended period simulation tracks a system over time, and it is a serious of linked steady state run. The need to run extended period simulation is because the system operations change over time.

- Demands vary over the course of the day.
- Pumps and wells go on and off.
- Valves open and close.
- Tanks fill and draw.

Depending on the purpose of the analysis, the most common simulation duration is typically a multiple of 24 hours, because the most recognizable pattern for demands and operations is a daily one.

CHAPTER THREE

RESEARCH METHODOLOGY

3.1 Description of the Study Area

3.1.1 General

Location: Debre Markos, the capital of East Gojjam Administrative Zone is located in the north west of the capital city of Ethiopia, Addis Ababa at a distance of 300 kilometers and 265 kilometers to the capital of Amhara Nation Regional State, Bahir Dar. Specifically it is located in the Amhara regional state, East Gojjam zone. Until 1995, Debre Markos was the town of the province of Gojjam. The town is located at 10^{0}20'N latitude and 37^{0}43'E longitude and it has an elevation of 2,400 meters above sea level. It has moderate temperature (The Enlightenment, 2009). The town is named Debre Markos after its principal church, which was constructed 1869 and is dedicated to Saint Mark.

Area: The area of Debre Markos Town is 6,160 ha and has oval shape; its Average Annual temperature is 18.5ºC; Mean Annual Rainfall is 1,380 mm and the existing wind direction is from north to south. The main natural constraints for the physical expansion of the Debre Markos town are hills, swamps, rivers and forests; while the manmade constraints are illegal settlements and urban rural boundary conflicts. The Debre-Markos water supply and sewerage service is a public institution in the town that is responsible for supplying of portable water and collection ,treatment and disposal of water and sludge for town, yet the disposal of sludge at present is being done by the municipality.

Population: According to CSA (2007), and population of the town was 92,470 and average annual growth rate of 4.5% .The town is administratively divided into seven administrative units (like locally named as kebeles).

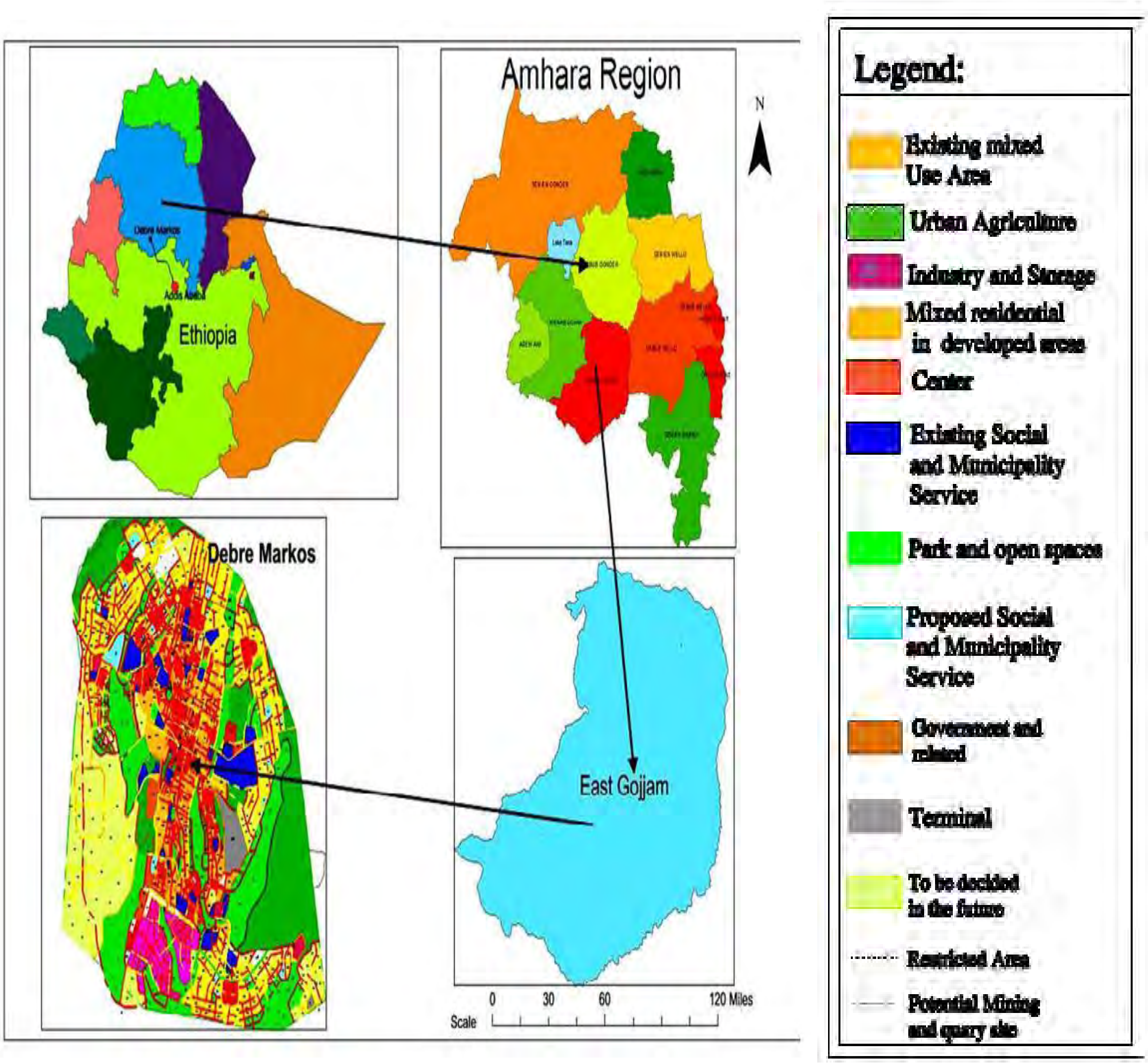


Figure 3.1: Location of the study area (Debre Markos town on the map)

Source: Adapted from Ethio GIS by Researcher, 2014

3.2 The Research Process

The water supply coverage of the town was first evaluated before analyzing the water loss, in evaluating the water supply coverage the focus was on the volume of consumption and level of water connection as these are highly related to the issue of water loss. After evaluating the distribution of water coverage and service year of pipes in the town the total water loss was analyzed. The total water produced and actual water consumption as aggregated from the individual contracts (customer meters) was used as an input for the analysis. After evaluating the total water losses at the two levels, the possible causes of water losses were tried to be identified by comparing the losses in conjunction with some factors having an effect to the water loss like ages of pipes and ground elevation differences (potential pressure) meter reading records of some sample customer meters that gets water from the same reservoir.

To investigate leakage prone points in the system pressure readings during the 24 hours at different zones were taken. These pressure readings were also used to determine the average pressure of the distribution system that is vital in the calculation of unavoidable annual real loss. In conjunction elevation readings were also taken at same zones that is planned to verify the relation between pressure and elevation.

3.3 Data Gathering Instrument

The instrument used to gather the required information includes interview and observation.

Interview

To secure additional information, unstructured interview question were conducted.

Observation

In order to make the research actual on site observation technique were carried out.

3.4 Data Collection

According to the work plan indicated in the proposal primary and secondary data were collected from the town water supply service and at the land in situ (field) testing was carried out. Some supplementary information was also collected from other respective offices, supportive qualitative information through discussion with local experts of water supply service was also important.

3.4.1 Primary Data Collection

3.4.1.1 Pressure Measurement

Pressure measurement throughout the entire day was conducted at different zones in the distribution system. At location where pressure gauges were installed, elevation readings were also taken. Critical times were selected while pressure gauges were taken. These critical times were fixed based on the demand rate of the users which covers the time between 8:00-12:00 (early mid noon) 2:00-6:00 (afternoon) 8:00-12:00 (early mid night) and 2:00-4:00 (early morning) (Lambert, 2003). Figure 3.2 below shows location of sample pressure nodes field-test on the distribution network.

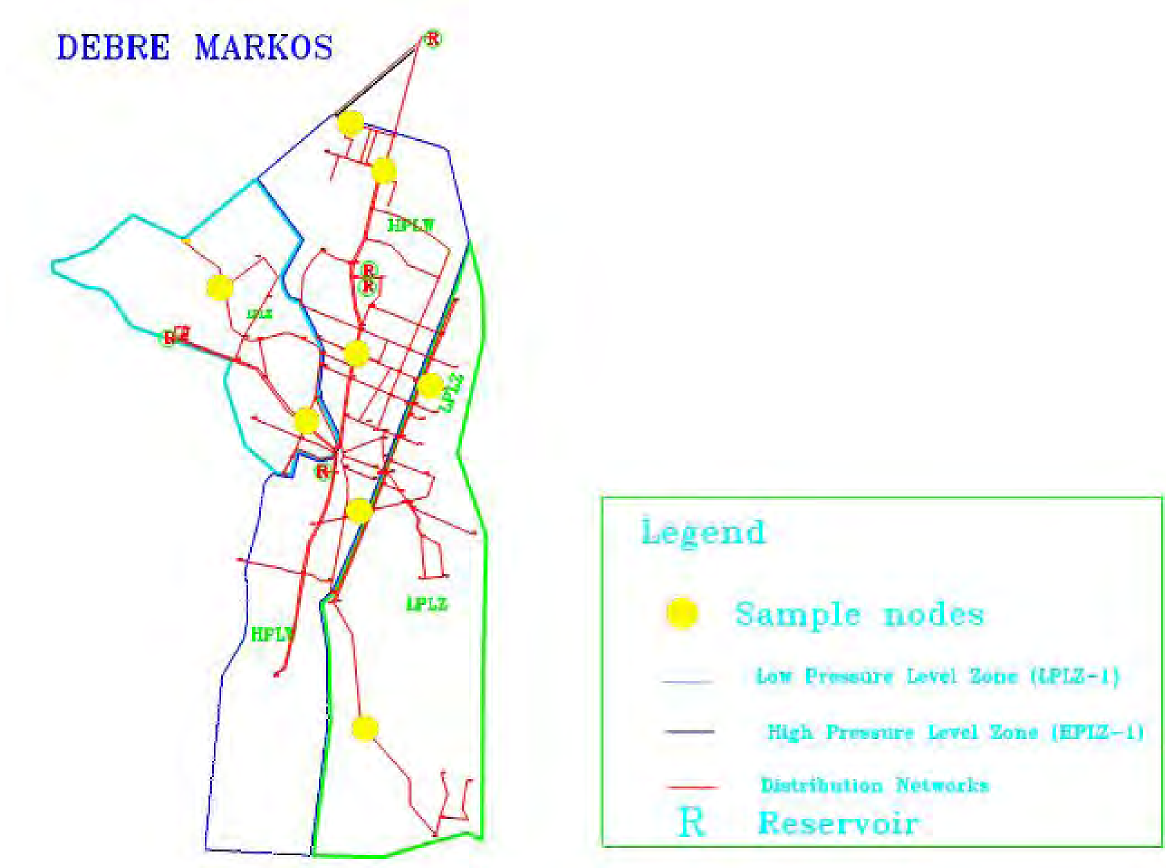


Figure 3.2: Locations of sampling site in distribution system

As shown in above figure 3.2 field investigations were done to identify the existing situation of the study area and based on reconnaissance survey eight sampling sites were selected to determine the average operating pressure for the calculation of unavoidable annual real loss and the distribution system each zone is vital for model calibration.

Table 3.1: Pressure reading value in selected zones in the distribution system

S. N	Zone name	Elevation (m)	Pressure day avg.			Pressure night avg.			Reservoir Elevation (m)		
			meter	bar	psi	meter	Bar	psi	R_1 &R_2	R_3	R_4
1	WSS office	2,392	45.50	4.55	64.7	57.60	5.76	81.9	2408	2499	2472.5
2	Abima	2,458	30.60	3.06	43.51	41.20	4.12	58.58			
3	Chew_berenda	2,374	54.80	5.48	77.92	62.30	6.23	88.59			
4	Mosque	2,427	31.70	3.17	45.08	45.60	4.56	64.84			
5	Hospital	2,464	29.80	2.98	42.38	38.60	3.86	54.88			
6	Tkelehaymanot	2,446	30.30	3.03	43.1	47.60	4.76	67.7			
7	Menkorer	2,440	31.10	3.11	44.22	40.60	4.06	57.73			
8	Debreza	2,396	40.90	4.09	58.1	58.10	5.81	82.6			
	Average pressure		36.84	3.68	52.3	48.95	4.90	69.6			
	Average working pressure		42.89m ,4.29bar and 61psi								

3.4.2 Secondary Data Collection

3.4.2.1 The Town Water Supply Networks

The entire town water supply network including their attribute like the length, diameter, material types, pressure capacity (PN) of the pipes, pumps characteristics, reservoir and tank section has been collected from the town water supply service office. The collected pipe network mainly comprises of main pipes and secondary pipes that covers the major part of the town. Extension networks are also included in the existing network during the site observation. The length of the entire network was summed up according to their diameter of further determination of unavoidable annual real loss.

3.4.2.2 Number and Type of Customers Meter

The number and type of customers with their corresponding meter type has been collected from the town water supply service office that can be used in the determination of real and apparent loss. The length of service connection is also summed up as it is required for the determination of real loss in the distribution system. Meters according to their size and type are also identified.

3.4.2.3 Water Meter Testing

Different type of water meters were installed in the distribution system during the lifetime of the project. These include water meter of Italy, India, China, Israel, France and Poland, samples of meter from each type were taken from the customers and tested at a calibration board installed (RODECO Consulting, 2007) "Debre Markos Water Supply project". The sample size was fixed according to IWA recommendation i.e. if testing all meters is not possible, a 5-10 percent sample test can represent the entire system (water loss manual-2005). Meter testing was conducted in two different scenarios, one under high pressure on the testing bench where a series of calibrating meters are arranged. Meters to be tested are installed next to the calibrating meter. Then water is pumped under recorded which then can be used to estimate the water lost due to water meter under registration. The other test is which very low pressure at which case the maximum leakage is fixed that would not be registered by the water meter.

In this case, a known and calibrated container and a stopwatch are used and the faucet is loosened slowly allowing the water to flow through the meter. At the critical time when the meter started to register the tap will be left open in the position and a known volume of water is collected to determine how much water is lost in such cases as most customers are were of it and adjust the tap in such a way as not the meter to start register as the water flowing through it.

3.4.2.4 Burst Frequency

Type of burst and their frequency has been collected. These data include the run time i.e. location, awareness and repair time, such data are important in the determination of leak quantity in the distribution system. The frequency of burst is determined form the run time report. The average run time is fixed from report format and with consultation of local experts of the water supply service. In addition to this type of bursts were identified and a large quantity is circular holes and cracks. The dimensions of the holes and cracks were measured form the replaced pipes and an average value were fixed for each type of burst.

Table 3.2: Burst frequency in the distribution system

Pressure zone	Average burst frequency/year
WSS office	66
Abima	96
Chew -Berenda	124
Mosque	148
Hospital	50
Treatment	94
Menkorer	68

Source: Debre Markos water service office, 2014

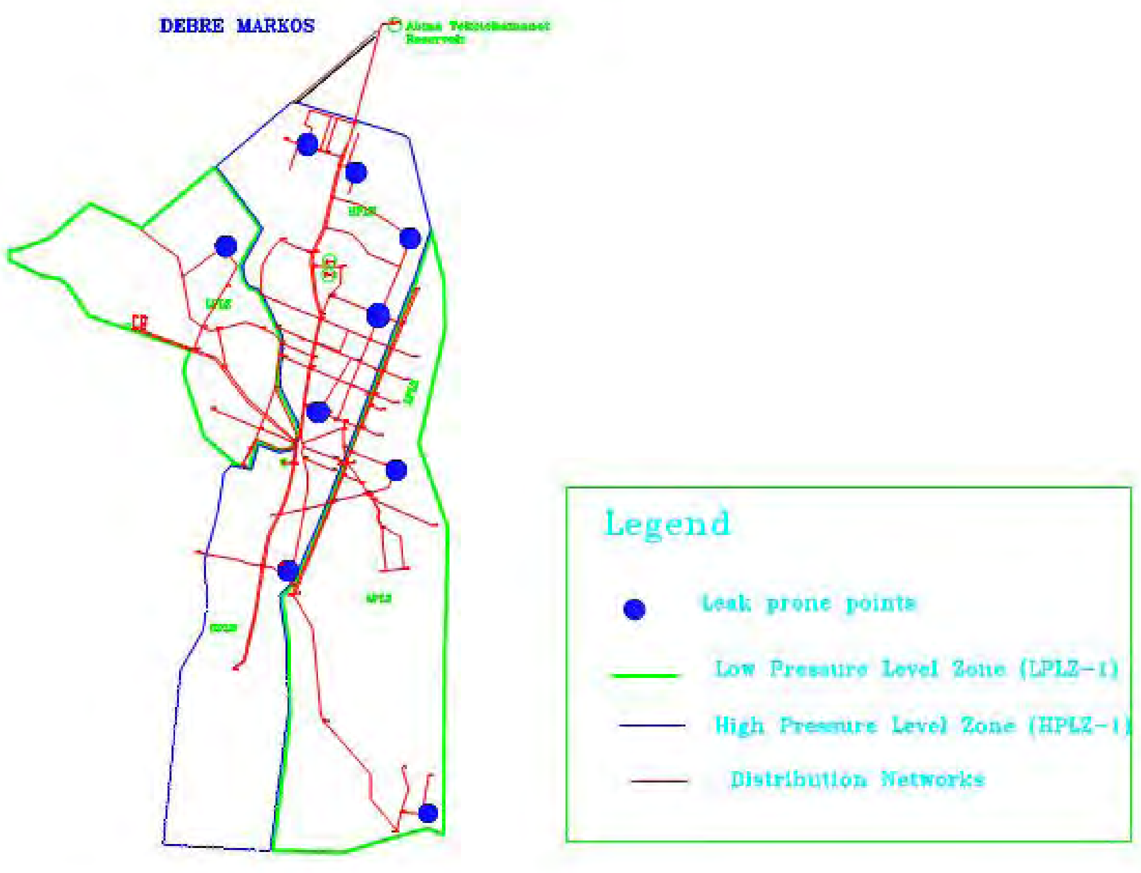


Figure 3.3: Leakage prone points in distribution systems

3.4.2.5 Town Water Reservoirs

The information on location of most of the water reservoirs were collected in conjunction with the main water network on the town. The reservoir's data including their capacity years of construction and material of construction were also collected. There are four concrete reservoirs in the distribution system located at three places. The two circular concrete reservoirs are located in Abma Mariam church at an elevation of 2480 masl in one place with a capacity of 500m^3 operated by a common valve chamber and each are erected side by side and water from the treatment plant is directly distrusted to the distribution network by booster pumps. The third palace reservoir locates in Betemengest (R_3) and its capacity is 200 m^3 concrete reservoir palace at an elevation of 2472 meter. The fourth reservoir (R_4) located in Abma Tkelehaymanot its capacity is 1500 m^3 concrete reservoir at an elevation of 2499 meter. Treated water from these reservoir flows to the distribution system by gravity flows. The location of town water reservoir is shown in figure 3.4 below.

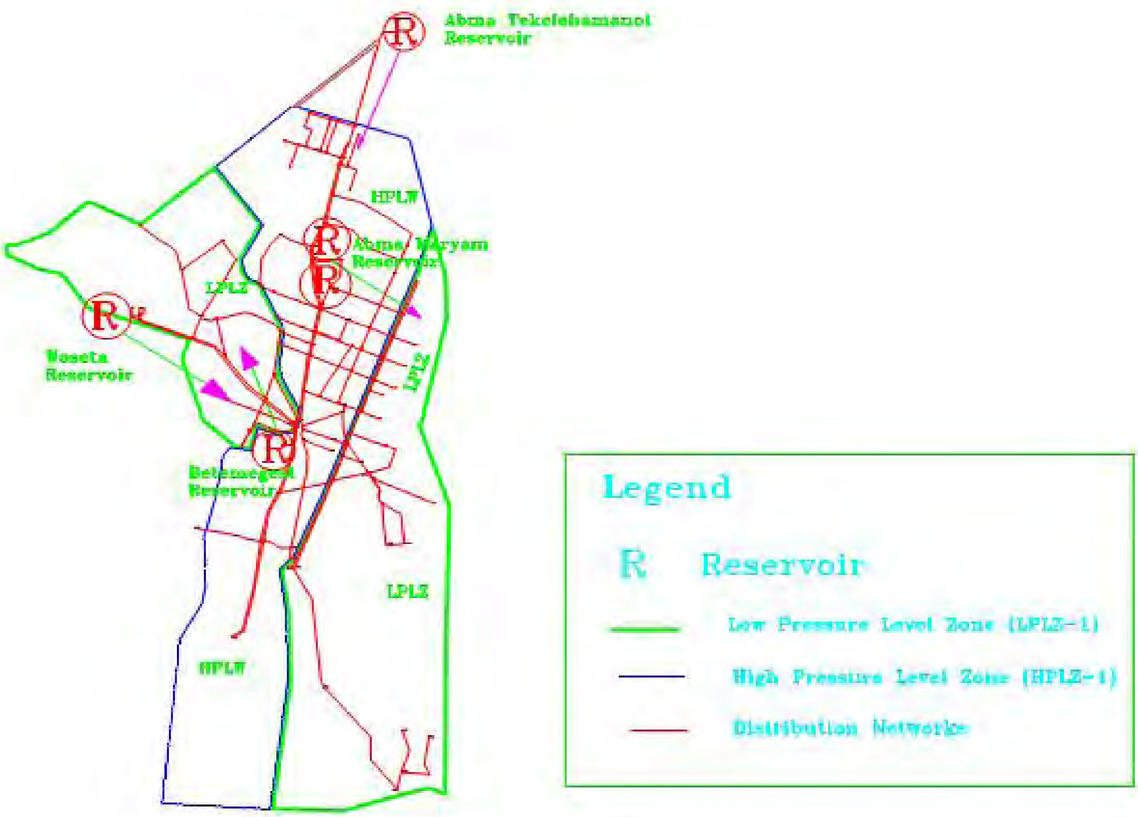


Figure 3.4: Reservoirs locations

3.4.2.6 Water Production

The existing Water Supply System was examined in close co-operation with the representatives of Debre Markos town water supply service regarding the fields of water production, treatment, transmission, and storage and water distribution network.

The source of water supply for the town is ground water. In 1971 seven boreholes were drilled to the west and southwest of the town .But now only five of them are giving service .In 2010 additional ten boreholes are drilled in the southwest of the town and out of these six boreholes are giving service . These functional boreholes are working for 20 hours per day. The other five are not functioning because of low yield of production and maintenance. Description and available data on the boreholes are presented in the Table 3.3.

Table 3.3: Description of existing boreholes in Debre Markos

S.N	Borehole	Average Current(l/s)	Current Working Hour/day	Daily Production (m3)	Monthly Production (m3)	Annual Production (m3)	Remark
1	BH1	3.64	20	261.81	7,854.41	94,252.90	Existing
2	BH2	2.50	20	180.00	5,400.00	64,800.00	Existing
3	BH3	2.52	20	181.44	5,443.20	65,318.40	Existing
4	BH4	4.99	20	359.28	10,778.40	129,340.80	Existing
5	BH5	5.00	20	360.00	10,800.00	129,600.00	Existing
6	BH1	5.00	20	360.00	10,800.00	129,600.00	New
7	BH2	5.00	20	360.00	10,800.00	129,600.00	New
8	BH3	5.00	20	360.00	10,800.00	129,600.00	New
9	BH4	5.00	20	360.00	10,800.00	129,600.00	New
10	BH5	5.00	20	360.00	10,800.00	129,600.00	New
11	BH6	5.00	20	360.00	10,800.00	129,600.00	New
12	BH7	5.00	20	360.00	10,800.00	129,600.00	New
13	Total	53.65		3,862.53	115,876.01	1,390,511.10	

Source: Aid Bank Underground Water Resources Development Project Drilling Data (GWE) and from water service office, 2002.

3.4.2.7 Water Consumption

In order to evaluate the water loss in the distribution system, consumption data of each customer were collected from the computer information section of WSS. There are more than 11,014 numbers of customers within the entire town. Water consumption in this context is metered billed and unbilled authorized consumption. While the consumption data was reviewed, significant differences between consecutive month were observed that might be caused due to non regular reading meter readings the authority do not have a cross checking mechanism

3.4.2.8 Population and Other Documents

Based on the CSA (2007), the numbers of the population figurer the year 2014 has been also collected from planning commission of the town.

3.4.2.9 Water Audit Study Period

The water audit study period was selected in such a way that it is long enough to analyze and evaluate total system water use. A 12 month study period starting in July and going through June is recommended. Most water system records are kept by either calendar or fiscal year. Either system normally makes 12 months of data available for review. It is recommended that a fiscal year (July through June) be used in order to reduce the effects of any meter reading lag time.

3.4.2.10 Units of Measurement

The measurement of units are the audit is selected to be m^3 bearing in mind that this unit of measurements should be maintained the same throughout the water audit analysis noting the type of register used for each device and verifying the appropriate conversion factor to be used when reading the device.

3.5 Method of Analysis

3.5.1 Water Supply Coverage Analysis

The water supply coverage of the town has been evaluated based on the average per capita consumption and by mode of service. The average per capita consumption has been derived from the yearly consumption that was aggregated from the individual domestic water meters. Beside to the average per capita water consumption, the distribution of number of domestic mode of service has been also evaluated. Statistical analysis was used to evaluate the supply coverage for the entire town.

3.5.1.1 Average daily per capital consumption

The volume of water consumed for domestic purpose has been aggregated to all 7 kebeles of the town so as to analyze the distribution of the water supply coverage among different localities. The annual consumption data has been converted to average daily per capita consumption using the number of population. The average daily per capita consumption of town was derived using the following expressions:

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

3.5.2 Water loss analysis

Estimation of real (physical) water losses were obtained by Greeley equations in manually. The quantity of leak in circular holes for the transmission, distribution, and connection pipes lines was calculated from Greeley's formula as:

$$\mathbf{Q = (43{,}767/1440) \times A \times \sqrt{p}}$$

Where, Q = flow in gallons per minute

A= the cross sectional area of the leak in square inches

P= the pressure in pounds per square inch (AWWA California Nevada section, 1992),

For leak rate from cracks and joints was estimated as

$$\mathbf{Q = (22.796) \times (A) \times (\text{square root of } P)}$$

Where "A" is the cross sectional area of the leak in square inches and P is the pressure in pounds per square inch (AWWA California Nevada section 1992).

Performance indicator Assessment

Unavoidable Annual Real Losses (UARL)

Considerable work was undertaken to assess the minimum level of leakage for any system (Lambert et al 1999) and after careful analysis a relatively simple and straightforward equation was developed as follow

$$\mathbf{UARL\ (liters/day) = 18xLm + 0.8xNC + 25xLP)\ x\ P}$$

Where Lm= Mains length (km) NC= number of service connection Lp = total length of private pie, property boundary to customer meter (km) P = average pressure (m).

Infrastructure leakage index

Calculating Infrastructure Leakage Index

The IWA methodology of determine and comparing leakage in water distribution system is now generally accepted as world's best practice. However, there has been healthy debate regarding the use of various performance indicators and this is expected to continue for many years to come. The ILI is sometimes criticized for being too simplistic and not incorporating some of the key factors, which can influence leakage from a water distribution system. The main points of discussion include,

- ILI values of less than 1.0 should not occur since this implies that the actual leakage is less than the theoretical minimum level of leakage
- The unavoidable annual real loss (UARL) equation is too simplistic as it is based only on the length of mains number of service connections length of underground pipe form the mains to the point of metering and the average system pressure.
- The use of the ILI in cases where a water utility operates under either abnormally high or unusually low pressures
- The use of the ILI for systems with less than 2000 connections
- Updating of the ILI parameters as more information that is reliable becomes available

While all of the above points are clearly valid concerns and can be debated at length. The ILI has proved to be extremely useful over the entire world

Although is recognized that an ILI of 1.0 is attainable from a theoretical viewpoints many utilities have challenged themselves to demonstrate economic viability. In other worda, a utility should not spend more than a dollar to save a dollar

The ILI indicators are defined as a ration of real losses (RL) and unavoidable annual real losses (URAL). It is a new indicator of water supply systems expressing the technical condition of the system from the point of view of water loss. This indicator is proposed and recommended by the international water association IWA (Lambert, 2002). As the operating records kept by the operator do not make it possible to determine the actual real losses (RL) individually for each pressure zone. The ILI calculation uses simplified values of non-revenue water (NRW) as

$$\mathbf{ILI = NRW/UARL}$$

Where, NRW = non revenue water (m^3/year

UARL= unavoidable annual real loss (m^3/year

The UARL is based on the results of an international survey containing data form 27 various water systems in 20 countries (lambert, 2002).

In addition, the water loss in the town water supply distribution system was evaluated using top-down water balance software. Detail analysis of the water loss components has been done using the AWWA water audit software version 4.2.

3.5.3 Distribution System Analysis

3.5.3.1 Hydraulic Modeling Software

The hydraulic modeling software Water CAD 6.5 simulation was carried out for the purpose of pressure regime for customers demand, velocity, and head loss and overall systematically studding and better understand network operation.

The use of the above software is recommended that the up to date Water CAD6.5 software for an unlimited number of pipes is appropriate for the development of the skeletal and all mains models of Debre Markos water supply network.

Software capabilities include graphical editing; image, CAD and GIS background support; steady-state and extended-period simulation; rule-based controls; pressure dependent demands; GIS interface for import/ export of data and results; automatic nodal demand allocation; pipe length-based demand loading; ground elevation extraction from shape files and CAD drawings; pressure zone management; automated model calibration.

3.5.3.2 Population Forecasting

Different population forecasting methods are in fact available and can be use for population projection. But their result varies from one method to another. Preference of the method appropriate for particular town needs to consider overall current situations of the targeted town.

For fast growing town, where relatively high economic activity is observed and at the same time continuous expansion of town due to various reasons is experienced, exponential method population forecasting is preferably used.

Exponential population forecasting method is expressed as follows;

$$\mathbf{P_n = P_o e^{rn}}$$

Where, P_n= population at year n

P_o= base year population

e = constant e, the base of natural logarithm

r = population growth rate

n =projection year

3.5.3.3 Nodal Demand Calculation

Demand allocation to consumption points are estimated using the following procedures (TAHAL Consulting, 2003).

1. Population size for each kebeles of Debre Markos woredas is projected.
2. From the known areas of kebeles and projected population for the design year, population density of the kebele is calculated.
3. Water demand is projected based on the pressure zones.
4. Location of nodal demand or consumption points is selected for demand allocating in the project area.
5. Service areas for each consumption point are delineated.
6. The delineated areas are overlapped to the kebeles and pressure zones.
7. Nodal demand is calculated using the following formulae.

$$\mathbf{\sum = dj\ pi\ Nd}$$

Where: Nd = Nodal demand

pi = population in each kebeles of the service area

dj = per capital demand for each pressure zones of the service area

i = subscript referring to the i-th kebele in the service area

j = subscript referring to the j-th pressure zone in the service area

3.5.3.4 Model Analysis

Analysis of the model of existing system has been made by running the model at current year daily average, at peaking and temporal variations of demand with different scenarios.

Steady-state Analysis

The model has been performed in steady state run for the average daily demand, which is the demand at every node not changing throughout 24 hours of a day. The software simulates Steady-State hydraulic calculation based on mass and energy conservation equations principle.

Extended-Period Simulation

The system conditions have been computed over twenty-four hours with a specified time increment of one hour and starting model run time at 7:00 A.M. The software simulates non-steady-State hydraulic calculation based on mass and energy conservation principle.

The model can be simulated for every one-hour time setup in the twenty-four hour duration. However, for the analysis the peak and minimum hours, demand has been simulated to identify the current problems of the system.

3.5.3.5 Model Calibration and Validation

For model calibration and validation effort data were collected from field visits. Collected data include: Tank section, reservoir elevation, pump characteristics, and pipe size, diameter, material types. For this study pipe roughness was taken as model parameter to be adjusted. Model calibration and validation were undertaken based on the following set of recommendations.

Acceptable levels of calibration

Pressure Criteria (Bentley Water CAD/GEMs ,2008)

(1) 85% of field test measurements should be within ± 0.5 m or ± 5% of the maximum head loss across the system, whichever is greater.

(2) 95% of field test measurements should be within ± 0.75 m or ± 7.5% of the maximum head loss across the system, whichever is greater.

(3) 100% of field test measurements should be within ± 2 m or ± 15% of the maximum head loss across the system, whichever is greater

CHAPTER FOUR

RESULTS AND DISCUSSION

4.1 Domestic Water Supply Coverage

Accesss to water supply may be evaluated using the amount of water consumed and by mode of service For evaluating the amount of water consumption, the annual water consumption is converted to average daily per capital consumption using the population data of town. Besides population distribution by mode of service has been also used as elaborated below.

4.1.1 Average Daily per Capital Consumption

The level of water consumed for domestic purpose has been aggregated to town so as to analysis the distribution of the water coverage among different localities. Evaluating the domestic water supply coverage using volume of consumption may not allow realizing the distribution comparison among the town. For this reason the annual consumption data has been converted to average daily per capital consumption using the number of population.

Table 4.1: Water production and consumption of Debre Markos

Year	Production m3/year	Consumption m3/year	Total Population	consumption l/person/day
2014	1,390,511	538,055	61,955	23.79

As shown on the table 4.1 the distribution of average domestic water supply coverage of the town in the year 2014 is found to be 23.79 l/capital/day. This average per capita consumption is very low while compared with the country standard used for design purpose (30 to 60 l/capital/day) as per EBCS 9. According to some literatures, a minimum quantity of 25 l/capita/day domestic water supply categorized as basic level of service (Wallingford HR, 2003) which is higher than the average domestic consumption of the town.

4.1.2 Population Distribution by Mode of Service

Mode of service is an important element on the one hand for evaluationg the level of water coverage that was the focus of this section and on the other hand it has a direct impact on the water loss that was dealt separately .

The adopted per capita water demands of each of the modes of services are described in the table here under.

Table 4.2: Population percentage distribution by mode of service

user	Total Number of Population served	percentage
Public Tap(PTU)	36,063	39%
YTU (Private)	15,720	17 %
YTU (Neighbouring)	7,398	8 %
House Connection	2,774	3%
Total	61,955	67%

Source: CSA (2007) Statistical Report on Housing Characteristics of Amhara Region, Debre Markos

As shown in the table 4.2 above the socio-economic household survey analysis identified the surveyed households who have access to safe water supply by mode of services. The survey indicates that, the majority of the inhabitants (about 39%) get their water from a tap outside their compound (i.e. from public fountains and vendors). About 17% have a private yard connection, 8% neighboring yard connection and 3% have house connection. The rest of the inhabitants about 33% use traditional water sources (like protected well/spring, unprotected well/spring, rivers and ponds) for their day to day water use.

4.1.3 Evaluating of the Distribution of the Water Supply Coverage

In this case section the 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.

4.1.3.1 Correlation Between Population and Billed Consumption

It is necessary to evaluate consumption with population. This has been evaluated using the correlation between the water billed consumption and number of population. Plotting water consumption by number of population graphically illustrates R-squared values for regression models are shown in figure 4.1.

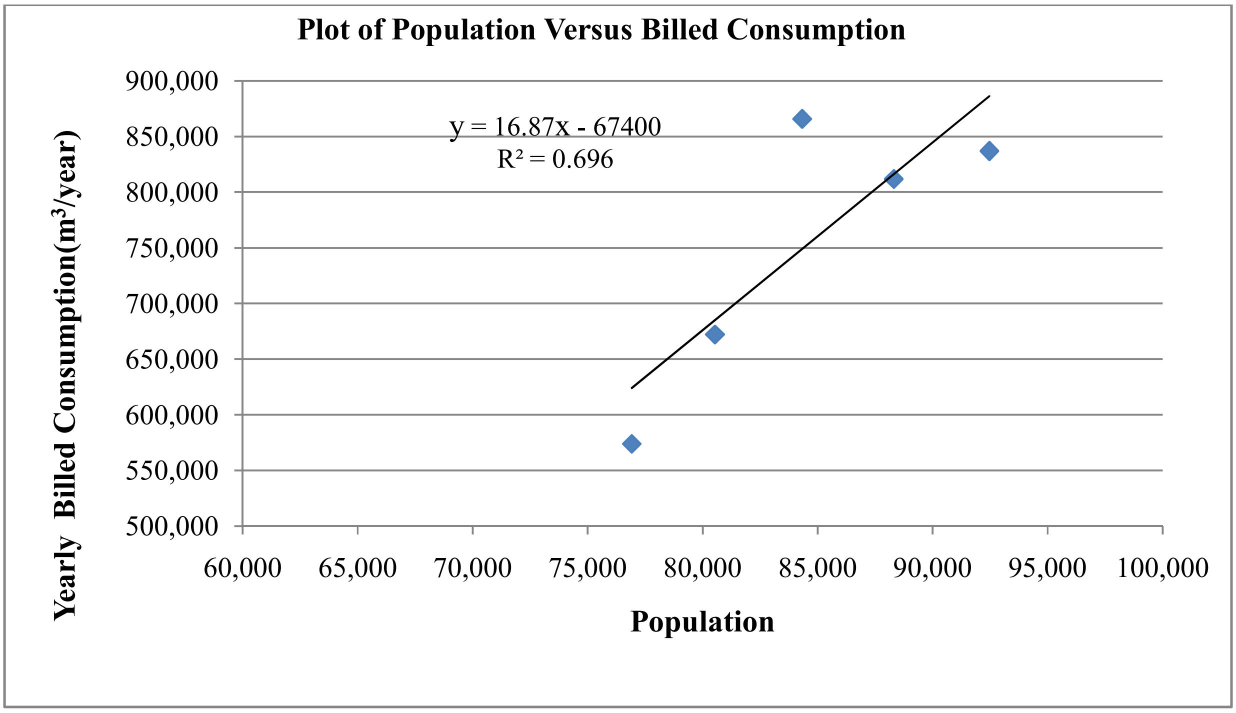


Figure 4.1: Scatter plot for volume of water consumption and number of population

As shown in the above figure 4.1 the coefficient of determination (r^{2}) is 0.69, indicates that the regression model accounts for 69% of water consumption is explained by population size.

4.2 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 town.

The water production and consumption of the water supply service are assessed based the past five years record. The production figures are taken from the water meter installed at the source and the consumption is read from the water meters installed for the customers and public fountains. The five years actual production and consumption figures obtained from the town water supply service is presented in figure 4.2.

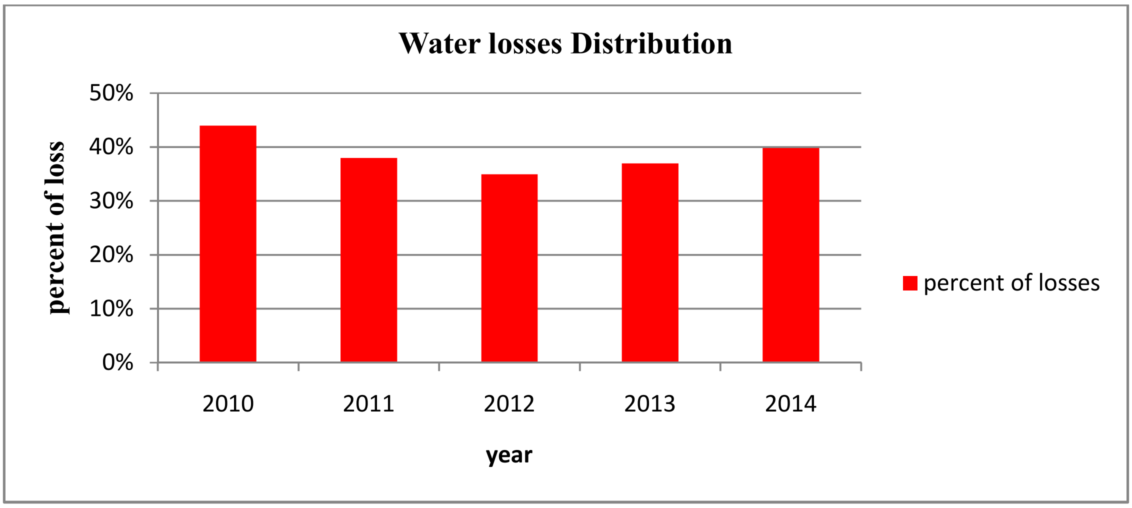


Figure 4 .2: Annual water losses of the town

As can be seen in the above figure 4.2 annual water produced and distributed to the system within specified year was 1,390,511 cubic meters and annual water loss as derived using the above expression was 553,663 cubic meters which account to 39.82% of the total production. Thus unaccounted for water is already on the higher side for a town size of Debre Markos. Decreasing the existing losses must be considered as part of the immediate rehabilitation plan of the town water supply system.

Water loss is usually expressed in terms of percentage (UFW), loss per kilometer length of main pipes and loss per properties or number of connections. The total water loss has been evaluated based on the three measurement approaches as explained here under.

4.2.1 Total Water Loss Expressed as Percentage (UFW)

The total annual water produced and distributed to the system within the specified year has been 1,390,511 cubic meters and the annual total water loss as derived using the above expression was 553,663 cubic meter that accounts to 39.81 % of the total water production. Taking the average tariff of water in the city as 3.25 birr/m^3, the water loss is estimated to be 1, 799, 4045 birr every year. However, the real loss is beyond this as the water tariffs like other developing countries are usually subsidized.

4.2.2 Water Loss Expressed 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 literatures, 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 town as 11,014 the water loss per connection for the similar duration was derived as,

Water loss = 553,663 x1000÷(11014×365) = 137.72 liter/connection /day. This figure shows as litters per service connection per day increase water losses also increases.

4.2.3 Water Loss Expressed as per Length of Pipes

Water loss expressed as per kilometer length of main pipes is also used as indicator to compare water loss. This indicator is usually recommended for non- densely populated areas. The total length of pipes of greater or equal to 50mm diameter have been used to evaluate total water loss of the entire town is 55.72km . Using total pipe length of the entire town, the water loss per kilometer length of main pipes was derived to be 553,663 ÷ (55.72 km×365days) = 27.22m^3/km/day**.** This figure shows that as length of the pipe increases the amount of water losses per day increases.

4.2.4 Pressure and Leakage

One of the major factors influencing leakage is the pressure in the water distribution system. In the past the conventional view was that leakage from water distribution systems is relatively insensitive to pressure, as described by Greeley formula is adopted (AWWA California Neveada section, 1992). Then the quantity of leak form small circular holes in the distribution line is calculated from Greeley's formula as:

Q= (43,767/1440)*A*square root of P

Where, Q = flow in gallons per minute

A= cross sectional area of the leak in square inches and

P= pressure in pounds per square inch (AWWA California Nevada section, 1992)

Figure 4.3 below illustrates shows the effect of the leakage rate on the pressure of sample nodes location in the distribution network.

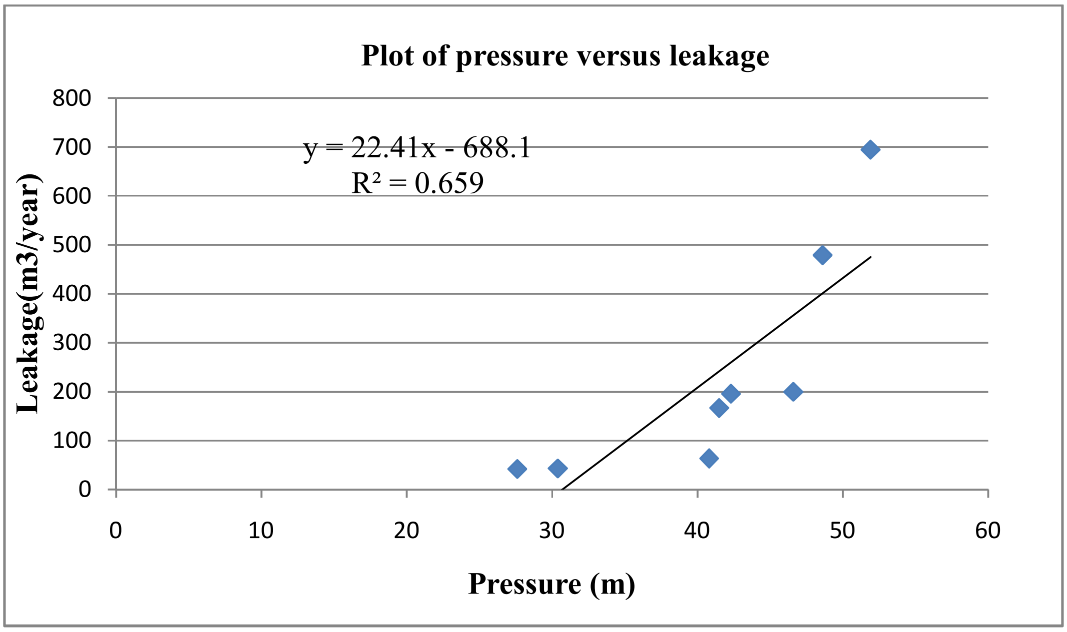


Figure 4.3: Pressure with leakage rate

The above numerical equation generating from continuous water flow and within the same duration the recorded the pressure and the water loss (leakage) at the same period, the pressure is varied with time to time during supply the water to the system record the pressure. As shown in the above figure 4.3 the coefficient of determination ($r^{2)}$ is nearly 66 % leakage is explained by the pressure.

4.2.5 Quantifying the Components of Non-Revenue Water

4.2.5.1 Quantifying Real Losses

These losses are measured form the pressurized point up to the point of measurement of the customer usage. These are physical losses from the infrastructure mains valves, service lines and main lines. There are many reasons for leaks: poor installation and workmanship; pressure transients, pressure fluctuation; lack of schedules maintenance on valves and hydrants or excessive pressure. All of these contributed to line loss. With proper system management, they can be kept to a minimum.

Real losses in a distribution system will occur in each of the components of the pipe system and the leakage is addressed under the following headings;

- Leakage from transmission mains
- Leakage from distribution mains
- Leakage from connections to property meter
- Service pipe leakage from property meter

4.2.5.1.1 Transmission Mains Leakages

Transmission mains tend to be large diameter pipes operating at high pressures with relatively few off- takes. As a result such pipes tend to experience few leaks and when a leak does occur it is so obvious and serious that it is repaired within a day if not within several hours. The frequency of leaks form transmission mains would be expected to be in the order of 0.030 per km mains per year with average leakage rate of $30m^3/h$. It is unlikely that there will be any unreported bursts on transmission mains due to the high pressures and large diameter pipes involved. However, if such leaks were to occur, is likely that the rate of leakage would be in the order of half that of the reported leaks. This estimation method is applicable if the required data are not available otherwise, there is different estimation formula and the most famous one is Greeley formula. For the estimation of leakage for Debre Markos town water supply system, the Greeley formula is adopted (AWWA California Neveada section, 1992).

From the water supply service office reports format it was found that is covers only 0.25% of the total leakage occurrences in a year. According to this statistical information the total leak occurrences in the transmission line is there times per year and the average duration of repairing time is 18 hours.

Then the quantity of leak form the transmission line is calculated from Greeley's formula as:

$$Q= (43,767/1440)\times A\times \text{square root of } P$$

Where, Q = flow in gallons per minute

A= the cross sectional area of the leak in square inches and P= the pressure in pounds per square inch (AWWA California Nevada section, 1992)

Accordingly the Leakage from transmission mains is calculated in table 4.3 below.

Table 4.3: Leakage from transmission mains (Collectors and Risers)

D (mm)	D (inch)	D ($inch^2$)	A ($inch^2$)	Coff.	$\sqrt{p}$ (p/inch)	Q (gal/min)	Q (gal)	Q (m^3/year)
100	3.937	15.49997	12.16748	30.394	7.820486	2,892.16	347,059.02	1,314.62
150	5.9055	34.87493	27.37682	30.394	7.820486	6,507.36	780,882.78	2,957.89
200	7.874	61.99988	48.6699	30.394	7.820486	11,568.63	1,388,236.06	5,258.47
250	9.8425	96.87481	76.04672	30.394	7.820486	18,075.99	2,169,118.84	8,216.36
350	13.7795	189.8746	149.0516	30.394	7.820486	35,428.94	4,251,472.93	16,104.06
500	19.685	387.4992	304.1869	30.394	7.820486	72,303.96	8,676,475.38	32,865.44
							Total	**66,716.84**

4.2.5.1.2 Distribution Mains Leakage

Distribution mains tend to be medium-size pipes operating at high to medium pressures with regular branches and off-takes. Such pipes can experience regular leakages and when a leak does occur, it is generally quite obvious and relatively serious, with the result that it is repaired within a day. The frequency of leaks form distribution mains would be expected to be in the order of 0.150 per km mains per year with average leakage rate of 12m3/h. while it is uncommon for distribution mains leaks to remain undetected for any length of time, some unreported mains leaks will occur. The frequency of such unreported leaks would be expected to be in the order of 0.008 per km or mains per year with an average leakage rate of 6m3/h-i.e. half the rate of the reported reticulation mains bursts.

As per the water supply service office operation and maintenance report, 2014 about 4% of the total real leak in the distribution system is lost through distribution mains and its frequency is 6 per year and the average duration of repairing time is 2 hours.

The leakage rate from this part is estimated from the Greely's formula as

$$Q = (43767/1400) \times A \times (\text{square root of } P)$$

Where, Q = flow in gallons per minute

A= the cross sectional area of the leak in square inches

P= the pressure in pounds per square inch (AWWA California Nevada section ,1992) .

Accordingly, leak in distribution main is tabulated below in table 4.4.

Table 4.4 Leakage from distribution mains

D (mm)	D (inch)	D ($inch^2$)	A ($inch^2$)	P^1/2	Coff.	Q (gal/min)	Q (gal)	Q (m^3/year)
50	1.97	3.87	3.04	7.82	30.394	723.04	260,294.26	985.96
80	3.15	9.92	7.79	7.82	30.394	1,850.98	666,353.31	2,524.07
100	3.94	15.50	12.17	7.82	30.394	2,892.16	1,041,177.05	3,943.85
150	5.91	34.87	27.38	7.82	30.394	6,507.36	2,342,648.35	8,873.67
200	7.87	62.00	48.67	7.82	30.394	11,568.63	4,164,708.18	15,775.41
250	9.84	96.87	76.05	7.82	30.394	18,075.99	6,507,356.53	24,649.08
300	11.81	139.50	109.51	7.82	30.394	26,029.43	9,370,593.41	35,494.67
350	13.78	189.87	149.05	7.82	30.394	35,428.94	12,754,418.80	48,312.19
400	15.75	248.00	194.68	7.82	30.394	46,274.54	16,658,832.72	63,101.64
450	17.72	313.87	246.39	7.82	30.394	58,566.21	21,083,835.16	79,863.01
Total								283,523.55

4.2.5.1.3 Connection Leakage

Leakage from connection pipes is normally the main source of leakage in any water distribution system and tends to exceed all other leakage combined. The frequency of leaks from connections would be expected to be in the order of 2.5 per 1000 connection of 0.0025 per connection with an average leakage rate of 1.6 m^3/h connection leaks are often unreported and such leaks can represent a sizeable portions of leakage in any system. The frequency of such unreported connection bursts would be expected to be in the order of 0.825 per 1000 connections per year with an average leakage rate of 1.6m^3/h i.e. the same rate as for the reported connections bursts.

As per the water supply service office operation and maintenance report about 85.75% of the total real leak in the connection pipes is lost through reticulation mains and its frequency is 6 per year and the average duration of repairing time is 3 hour. Of this figure about 67% of the leak is form small circular holes and accounts to a frequency of 99 per year.

The leakage rate from this part is estimated from the Greely's formula as Q= (30.394) × (A) × (square root of P) gallons per minute where A is the cross sectional area of the leak in square inches and P is the pressure in pounds per square inch (AWWA California Nevada section, 1992). Accordingly, leak in connection main is tabulated below in table 4.5.

Table 4.5: Leakage from Connection mains

D (mm)	D (inch)	D ($inch^2$)	A ($inch^2$)	P^1/2	Coff.	Q (gal/min)	Q (gal)	Q (m^3/year)
20	0.78	0.62	0.48	7.82	30.394	115.69	2,124,001.17	8,045.46
25	0.98	0.98	0.76	7.82	30.394	180.76	3,318,751.83	12,571.03
32	1.25	1.59	1.24	7.86	30.394	296.16	5,437,443.00	20,596.37
40	1.57	2.48	1.94	7.82	30.394	62.75	8,496,004.69	32,181.84
Total								73,394.70

Leaks from cracks and joints accounts for 33% of the leak from connection leak with a frequency of 90 per year and this leak rate is estimated from the Greeley's formula as Q= (22.796) × (A) × (square root of P) Where "A" is the cross sectional area of the leak in square inches and P is the pressure in pounds per square inch (AWWA California Nevada section 1992). Accordingly, Leaks from cracks and joints in connection main is tabulated below in table 4.6.

Table 4.6: Leaks from cracks and joints in connection mains

D (mm)	D (inch)	D ($inch^2$)	A ($inch^2$)	P^1/2	Coff.	Q (gal/min)	Q(gal)	Q (m^3/year)
20	0.7874	0.61	0.48	7.82	30.394	86.76	937,079.89	3,549.55
25	0.98425	0.96	0.76	7.82	30.394	135.57	1,447,918.59	5,484.54
32	1.25984	1.58	1.25	7.82	30.394	222.12	2,398,924.53	9,086.84
40	1.5748	2.47	1.94	7.82	30.394	347.07	3,748,319.58	14,198.18
Total								32,319.10

4.2.5.1.4 Service Pipe Leakage

Service pipe leaks are relatively common and tend to run undetected for longer periods than other forms of leakage, in cases where service pipe leakage is considered to be important the same figures can be used as suggested for the connection leakage. In cases where such leakage occurs after the customer meter and all water is being paid for by the consumer, the service pipe leakage can be disregarded from the calculation since it has been included as "consumption" in the water balance calculation, for the town of Debre Markos water supply case the service pipe leakage is omitted from the overall financial calculation as all the leakage occurs after the customer meter and all water is being paid for by the consumer.

4.2.5.1.5 Determination of Unavoidable Annual Real Loss (UARL)

This category represents the allowable volume of real losses from the system, which estimates a volume of leaks that are undetectable or would be uneconomical to repair during the year. This can help to evaluate the feasibility of real loss minimization (provides better understanding of real loss components).One of the most important concepts used in the BABE procedures concerns the minimum or unavoidable level of leakage for any given system. Effectively it is a simple concept based on the fact that no system can be entirely free from leakage, which cannot be reduced any further. Even a new distribution system with no use will have some level of leakage, although it may be relatively small. The minimum level of leakage for a system is termed the unavoidable annual real losses or UARA. This is the level of leakage that can be achieved if the system

- Is in top physical conditions and is well maintained
- All reported leaks are repaired quickly and effectively
- Active leakage control is practiced to reduce losses from unreported leaks

Considerable work was undertaken to assess the minimum level of leakage for any system (Lambert et al., 1999) and after careful analysis a relatively simple and straightforward equation was developed as follow

$$\mathbf{UARL = (18 \times Lm + 0.8 \times Nc + 25 \times Lp) \times P}$$

Where, UARL = unavoidable annual real losses (l/d)

Lm= length of mains (km)

Nc= Number of service connections (main to meter)

Lp= length of unmetered underground pipe form street edge to customer meter (km)

P= average operating pressure (m)

Accordingly, the cased of Deber Markos water supply UARL is calculated as

$$UARL = (18\times55.72+0.8\times11014+25\times0)\times43$$

$$=422008.88 \text{lit/day}$$

$$=422\ m^3/\text{day}$$

$$=422\times365 m^3/\text{year}$$

=**$154{,}030m^3$/year**

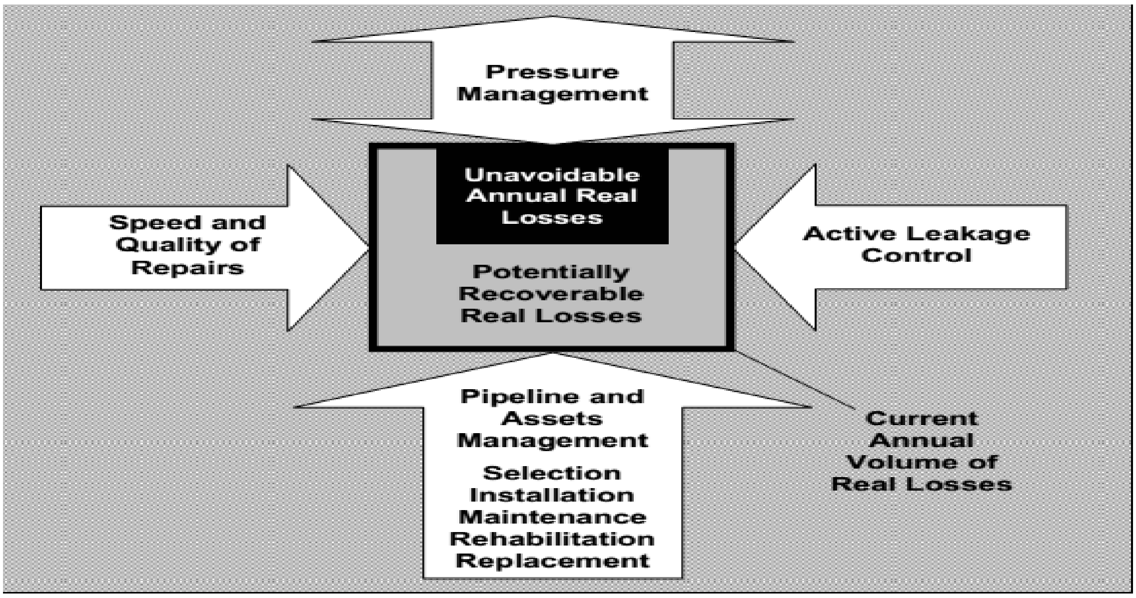


Figure 4.4: The four components of a successful leakage management policy

Source: Liemberge and Farley, 2003

The black box represents the unavoidable annual real losses (UARA) the lowest technically achievable volume of real losses at current operating pressure.

4.2.5.2 Quantifying Apparent Loss

Apparent water losses are made up of four components: the first, meter under registration, is possible the easiest to picture, whereby a revenue meter will not accurately measure the water supplied to a household, various type of water meter exist, with each type boasting different properties. The most common small revenue meters are the volumetric, single jet and multi jet models. A second apparent loss is water theft, whereby water is stolen from a water distribution network often via a meter bypass or through an illegal service connection. A third apparent loss is that of meter reading or collecting errors, whilst a fourth apparent loss is caused water billing and accounting errors.

4.2.5.2.1 Quantifying Loss Due to Meter under Registration

4.2.5.2.1.1 Quantifying Loss Due to Meter under Registration When the Meter is pressurized

As it is not possible to test all customers' meter a sample of meters as recommended by IWA task force (5-10% of the total water meters from the distribution system), A total of 1,099 customer meters were replaced 1008 DN15 meters (Class B, dry dial, multi-jet), 55 DN20 meters (Class B, dry dial, multi-jet) and 36 DN25 meters (Class B, dry dial, multi-jet) from each brand and different age categories is taken and tested on a testing bench and the result is tabulated as shown below). The annual water loss is calculated taking 200 lit consumption per connection per day, then to fetch 200 lit the discrepancy is as indicated in column no 8 in table 4.7 below .

Table 4.7: Annual water loss from meters when the meter is under pressure

Manufacturer	Type meter	Meter size	Number of tests	Master meter reading (lit)	Test meter reading (lit)	Reading difference (lit)	Annual loss (m^3)
Italian	class B,Dry Dial,Multijet	DN 15	3,243	200	174	26	30,776.07
French	class B,Dry Dial,Multijet	DN 15	96	200	150	50	1,752.00
Polish	class B,Dry Dial,Multijet	DN 15	978	200	181	19	6,782.43
Chinese	class B,Dry Dial,Multijet	DN 15	906	200	163.	37	12,235.53
Israeli	class B,Dry Dial,Multijet	DN 20	373	200	194	6	816.87
Indian	class B,Dry Dial,Multijet	DN 25	31	200	179	21	237.62
	Total		**5,627.00**				**52,600.52**

Source: RODECO Consulting (2007) "Debre Markos water supply project"

4.2.5.2.1.2 Quantifying Loss Due to Meter under Registration When the Meter is at Low Pressure

The other scenario of meter testing was when the flow through the meter is under very low pressure, as explained earlier taking samples, which are supposed to be representative total of 1099 customer meters were replaced 718 DN15 meters (Class B, dry dial, multi-jet), 187 DN20 meters (Class B, dry dial, multi-jet) and 187DN25 meters (Class B, dry dial, multi-jet) was taken and tested on a testing bench and the result is tabulated as shown below). The annual water loss is calculated taking 200 lit consumption per connection per day, then to fetch 200 lit the discrepancy is as indicated in column no 8 in table 4.8 below.

Table 4.8 Annual water loss from meters when the meter is under low pressure

Manufacturer	Type meter	Meter size	Number of tests	Master meter reading (lit)	Test meter reading (lit)	Reading difference (lit)	Annual Loss (m3)
Italian	class B,Dry Dial,Multijet	DN 15	403	200	154	46	6766.37
French	class B,Dry Dial,Multijet	DN 15	403	200	135	65	9561.18
Polish	class B,Dry Dial,Multijet	DN 15	403	200	151	49	7207.66
Chinese	class B,Dry Dial,Multijet	DN 15	403	200	163	37	5442.52
Israeli	class B,Dry Dial,Multijet	DN 20	403	200	173	27	3971.57
Indian	class B,Dry Dial,Multijet	DN 25	403	200	149	51	7501.85
	Total		**2,418**		**925**		**0,456.9**

Source: RODECO Consulting (2007) "Debre Markos water supply project"

4.2.5.2.2 Unbilled Metered Consumption

This is the volume of water used by the water supply service workers for their domestic use in their house in the administration office treatment plant and wells. This volume of water is considered non- revenue water as the water service office, 2014 report it is supplied free of charge though it is measured and it has volume of 799 m^3/year.

4.2.5.2.3 Unauthorized Consumption

This volume of water includes theft and illegal connections. As there is no any means to determine this quantity of water, its volume is estimated based on the system input volume. Accordingly, according to the water service office, 2014 report unauthorized consumption amounts 8,646.02m^3/year.

4.2.5.2.4 Calculating Infrastructure Leakage Index

The infrastructure leakage index (ILI) indicators are defined as a ratio of real losses (RL) and unavoidable annual real losses (URAL). It is a new indicator of water supply systems expressing the technical condition of the system from the point of view of water loss. This indicator is proposed and recommended by the international water association IWA (Lambert, 2002). As the operating records kept by the operator do not make it possible to determine the actual real losses (RL) individually for each pressure zone. The ILI calculation uses simplified values of NRW as

$$\mathbf{ILI} = \frac{\mathbf{NRW}}{\mathbf{UARL}}$$

Where, ILI= infrastructure leakage index (ILI)

NRW= Non -Revenue water (m^3/year)

UARL =unavoidable annual real loss (m^3/year)

The UARL is based on the results of an international survey containing data from 27 various water systems in 20 countries (Lambert, 2002).

Based on equation above the infrastructure leakage index for the Debre Markos town water supply distribution system is calculated as 553,663 ÷154,030=**3.59**

This shows us that the current annual real losses are assessed as being around four times as high as the unavoidable annual real losses for the system.

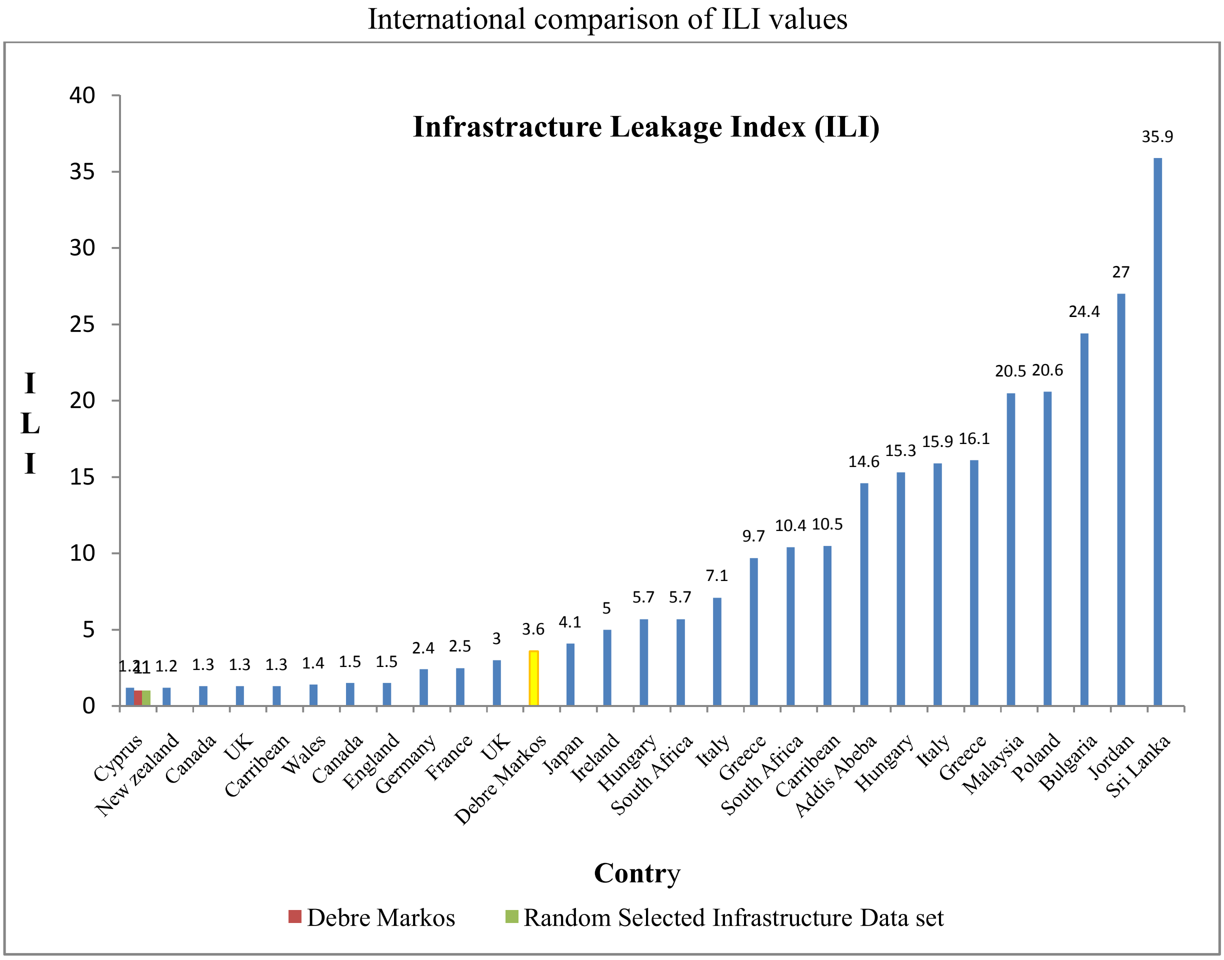


Figure 4.5: Infrastructure leakage index (ILI) values for 35 supply system in 30 counties

4.2.5.2.4.1 Interpreting ILI Values

If the ILI for a particular system is calculated and as say 4.0 this means that the current annual real losses are assessed as being around four times as high as the unavoidable annual real losses for a system with this length of mains, number of connections and customer meter location under the same pressure management regime as the particular system under review.

Options may exist for lowering annual real losses to around one fourth of the current annual real losses if there are no changes in the current pressure management regime.

Additional changes in real losses will result from changes in the pressure management regime.

In practical terms ILI values close to 1.0 mean that world class leakage management is ensuring that annual real losses are close to the unavoidable or technical minimum value at current operating pressures.

However, such low ILI values are only likely to be economically justified when marginal costs of water supply are relatively high (e.g. Desalination) or water scarce or both. Results of water audit model are attached in appendix A.

4.2.5.3 Quntifying loss by Water Balance Method

For the years 2014 IWA water balance components are obtained by using data and estimated in the above. The results are summarized in table 4.9 below.

Table 4.9: Water balance (m^3) for fiscal year 2006 (E.C)

<table>
<tr><td rowspan="7">System Input Volume
1,390,511
100%</td><td rowspan="4">Authorized Consumption
837,717
60.24%</td><td rowspan="2">Billed Authorized Consumption
836,918
60.18%</td><td>Billed metered consumption (including water exported)
836,918
60.18%</td><td rowspan="2">Revenue water
836,918
60.19%</td></tr>
<tr><td>Billed unmetered consumption
0
0.0%</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption
799
0.05%</td><td>Unbilled metered consumption
799
0.06%</td><td rowspan="5">Non Revenue Water
554,462
39.87%</td></tr>
<tr><td>Unbilled unmetered consumption
0
0.0%</td></tr>
<tr><td rowspan="3">Water losses
553,663
39.81%</td><td rowspan="2">Apparent Losses
101703.44
7.31%</td><td>Unauthorized use
8,646.02
0.62%</td></tr>
<tr><td>Metering inaccuracies
93,057.65
6.62%</td></tr>
<tr><td colspan="2">Real Losses
451,959.56
32.5%</td></tr>
</table>

As shown on the above table 4.9, the results of Non-Revenue water by water balance method high levels of NRW (39.87% of System Input Volume) and water losses (39.81% of System

Input Volume) have serious impact on Debre Markos water supply service office's finances and available water resources.

4.2.6 Evaluating Possible Causes of the Water Loss

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

4.2.6.1 Evaluating loss based on Age of Pipe Network

Pipe age is one of the factors that affects the magnitude of the loss specially that of physical loss. Aged pipes are more likely having more water loss through leakage than newly installed pipes. It is estimated that nearly more than 57.74% of the pipe network in the town was laid over 25 years ago. These lines are including DCI (ducktail cast iron), polyvinyl chloride (PVC) and steel pipes. The aged pipe is especially in the central part of the town and in densely population areas. 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. The ages of pipes in the system and the corresponding loss per length of pipes is summarised as shown in Table 4.10 below.

Table 4.10: Distribution pipes by age categories

S.N	Material type	Age Categories	Length (km)	Length (%)
1	UPVC	10 year &less	19.27	34.58
2	Cast iron	10 year &less	7.7	13.82
3	Galvanized steel pipe	20-30	20.8	37.33
		30-40	5.36	9.62
		40 and more	2.53	4.54
4	Cast iron	30 and more	0.06	0.11
Total			55.72	100.00
Loss per length			27.22m3/km/day	

Source: Debre Markos water service office, 2013

As shown in Table 4.10 nearly more than 35% of the UPVC pipe network in the system central town having an age less than 10 years, and while for Menkorer and Abama were above 40 years 3%. Only considering age factor, central part of town have lower percentage. The loss is found to be higher in Menkorer and Abma relatively having older pipes systems.

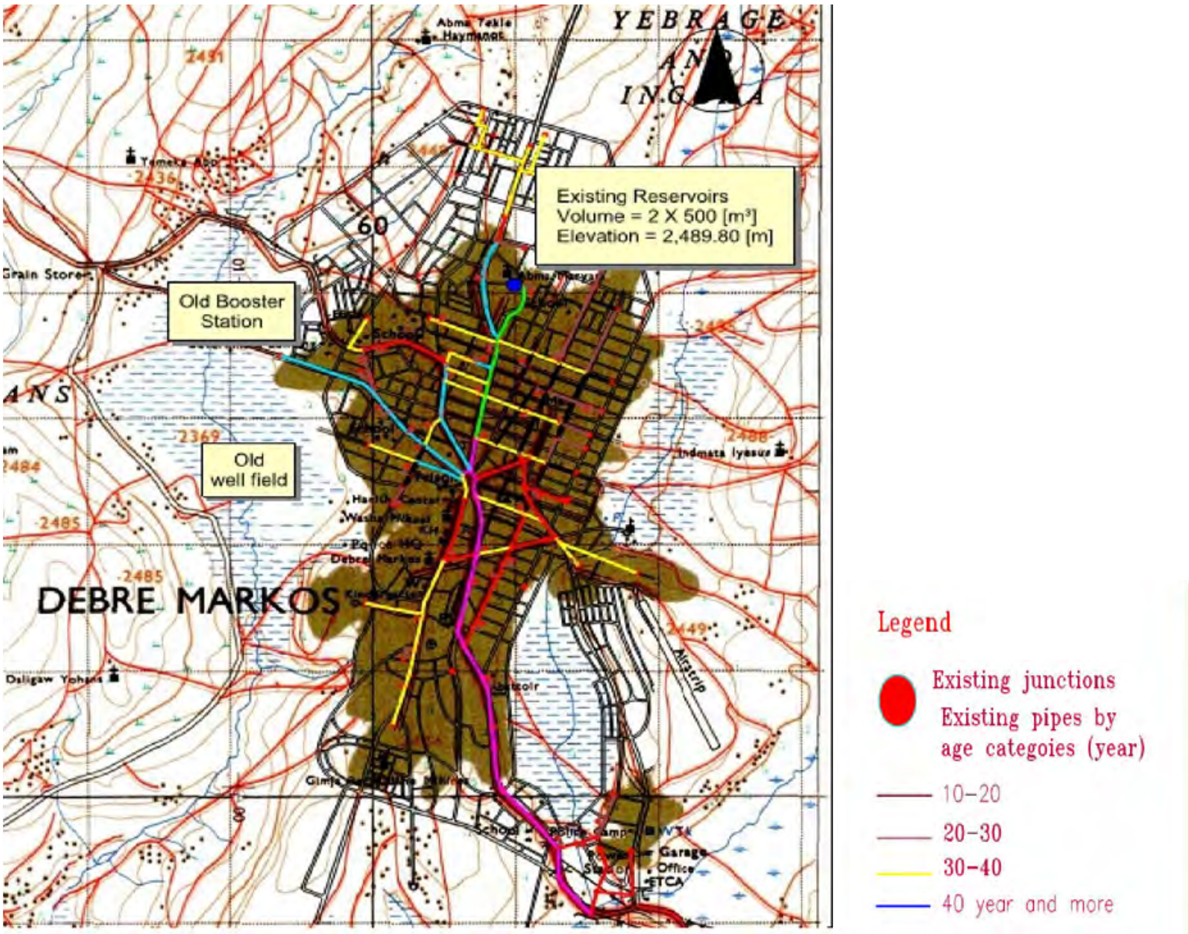


Figure 4.6: Existing distribution pipes networks by age categories

Source: Debre Markos water service office, 2013

4.2.6.2 Poor Maintenance of Networks

In some places like expansion areas water supply office service has performed a maintenance program for distribution system, and in recent years to WWS nearly 7% of network system was replaced in the expansion areas. In some parts of the town, Poor materials or workmanship to maintenance of water supply network. Thus, the lack of finance to buy proper materials and poor construction resulted in increased leakage in the system.

4.2.6.3 Evaluating Loss based on Customer Meter Record

As discussed in the previous section the total water loss of water due to meter inaccuracies accounts to 93,057.42m^3/year and this cover 16.8% of the total water loss of the distribution system.

4.2.6.4 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 town, increased levels of leakage are experienced due to stress being inflicted on the pipes causing them to rupture. There is clear paradox 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.

4.2.7 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 Debre Markos town, 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.

4.2.7.1 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 labour 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).

4.2.7.2 Establishing Pressure Management

Pressure management is related to the establishment of zoning and district meter areas. In the previous sections this has been discussed in detail with the perspective of identifying and quantifying losses. While establishing zoning and district meter areas, the scope of pressure management should be evaluated in all cases, the relationship between pressure and bust frequency is explained in detail in the previous section.

4.3 Distribution System Modelling

4.3.1 Model Representation

Frequently system maps are drawn as combination of various system components enclosed in water distribution system. It is common to include; Reservoirs, Tanks, Pipes, Pumps and Valves as much as possible and the resulting sketch fairly represent the actual water network. With little difference the real water distribution system represented as combination of nodes and links. Junctions, reservoirs and tanks are usually referred as nodes. Pipes pump and valves are categorized as links. Figure 4.7 below illustrates layout of Debre Markos distribution system. The sketch was extracted from town water distribution system map and represented in the model according to available drawing options. Model system is numerically summarizes the plot is presented Table 4.11.

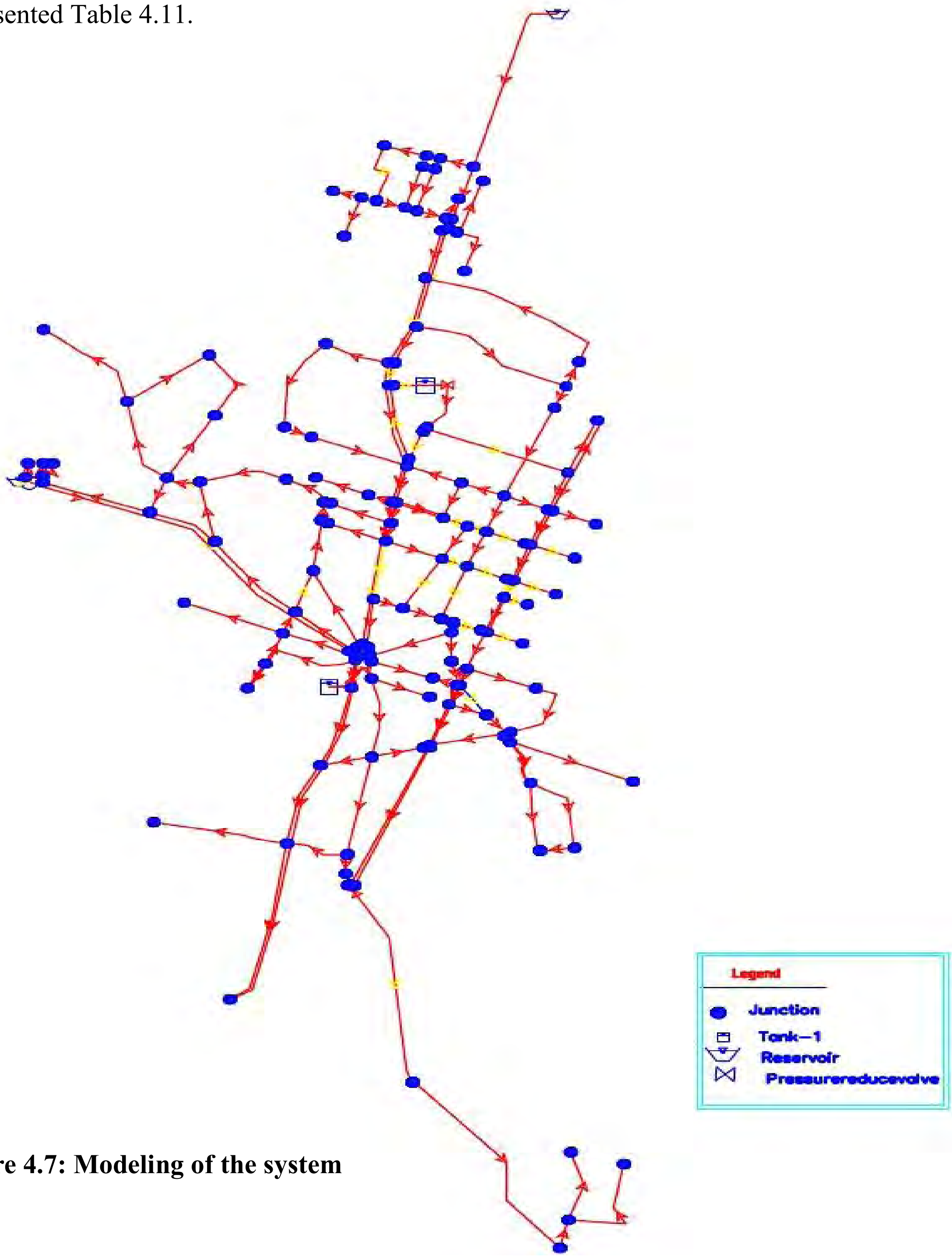


Figure 4.7: Modeling of the system

Table 4.11: Summary of system elements

System components	Number
Junction	133
Pipes	197
Pumps	2
Reservoirs	2
Tanks	3
Pressure reducer valves	3

4.3.2 Population Forecasting

In order to forecast the current population (2014) of the study area based on last population census report population and housing census report of 2007 which was prepared by Ethiopian Central Statistical .Growth rate of 4.5% which was reported by CSA for town of Debre Markos was used for the current projection**.** Moreover the exponential growth rate model has been used. Estimated population town level is presented in Table 4.12.

Table 4.12: Population Growth rates

Year		Growth rate
2003	2008	4.9%
2009	2013	4.4%
2014	2018	4.5%
2019	2023	4.3%
2024	2028	4.1%

Source: CSA (200) National census figures

Applying the growth rate in the exponential model, the urban population of Debre Markos is projected up to year 2020 and is presented in Table 4.13.

Table 4.13: Population projection (2006-2020)

		2014	2018	2023	2028
Annual growth rate (urban)		4.40%	4.50%	4.30%	4.10%
Population (urban)		92,470	110,645	136,007	163,880

Source: CSA (2007) National census figures are used as a base

Based on the above table 4.13 the population of the Debre Markos is 92,470 (2014) and 163,880 (2028).

4.3.3 Water Demand

The current estimation of water use by entire population required to review a set of various data. Water production data along with present and future population data were repeatedly reviewed.

Accordingly, the two mentioned data were primarily used for water demand analysis. Prior to assigning nodal water demand, it was required to establish average use of water per person. Per capita water demand was taken as average water use. Maximum water use and minimum water use usually related to average water use by multiplication of peaking factors.

According to Debre Markos town water supply service unpublished report in 2014, WSS is currently supplying mean per capita demand of 80 litters per day at town level. This figure includes water loss in the system and average water loss was 38%. Results of water demand projection are attached in appendix F.

4.3.4 Nodal Water Demand

Nodal demand, nodes that representing each kebele were identified by overlapping water distribution system map with Debre Markos town master plan .Results are attached in appendix B.

4.3.5 Model Calibration and Validation

Model calibration is the process of comparing the results of model simulation to actual field data and making corrections and adjustments to the model in order to achieve close agreement between model predicted values and field measurements. Typical comparison values include pressures, flow rates and service reservoir water levels. Model parameters that may require correction during the calibration process include system connectivity, node ground elevations, control valve settings, pump characteristics. Estimated model parameter values that may require adjustment include pipe roughness coefficients and nodal demand allocation and peak factors.

4.3.5.1 Hydraulics Calibration and Validation

Even though the required data have been collected and entered into a hydraulic simulation software package, the modeler cannot assume that the model is an accurate mathematical representation of the system. The hydraulic simulation software simply solves the equations of continuity and energy using the supplied data. Thus, the quality of the data will dictate the quality of the results. The accuracy of a hydraulic model depends on how well it has been calibrated, so a calibration analysis should always be performed before a model is used for decision-making purposes.

4.3.5.2 Sampling Location

A typical network representation of a water network may include hundreds or thousands of links and nodes. Ideally, during the water distribution model calibration process is adjusted for each link and each node. However, only a small percentage of representative sample measurements can be made available for the use of model calibration due to the limited financial and labor requirements for data collection. Therefore, it is of utmost importance to have a comprehensive methodology and efficient tool that can assist the engineer in achieving a highly accurate model under practical conditions (Walski et al., 2003).

Selection of sampling sites is typically a compromise between selecting sites that provide the greatest amount of information and sites that are most amenable to sampling. Sites should be spread throughout the study area and should reflect a variety of situations of interest, such as distribution mains, high pressure zone, low pressure zones, and leakage prone area at different zone in the systems. Pressure measurement of sample allocation point presented in chapter three (methodology part).

Eight representative sample measurements (eight data sets observed data and eight data sets from simulated) the water main spread throughout the study area have been selected for the calibration. It was difficult to take measurement at a direct connection to the water main nodes, due to size of pressure gauge available only in this junction. The size of water main in the study model integrates a size greater or equal to 50 mm as previously stated.

For the calibration, the head loss between the supply main nodes and the site where pressure is measured had been considered. The head loss included the elevation head and pipe friction loss between a two corresponding locations. These head losses and the total head loss are shown in table 4.14

As a result, 100% of the field test measurements were within ± 2 m, showing an acceptable level of pressure calibration criteria in chapter three (methodology part). The comparisons of model simulated and field test are described below in table 4.15 and figure 4.8. Calibrations have been carried on the base scenarios within the acceptable level. Hence, the model is valid for the scenarios.

Table 4.14: Locations of the representative samples of supply main nodes and the corresponding zones

S. N	Level	Sample Node			Corresponding Field Test Measurement Location			Head Loss between sample node and field test location			scenario
		X	Y	Z	X	Y	Z	Elevation Head (m)	Friction loss (m)	Total Head Loss (m)	
1	J-120	361,109.9	1,145,050	2,461.00	361,763.9	1,144,181.59	2,460	1	0.5	1.5	Base scenario
2	J-46	360,421.4	1,142,782.7	2,426.00	360,042.4	1,144,069.29	2,427.2	1.2	1.0	2.2	
3	J-16	360,812.3	1,145,475.8	2,445.00	361,813.2	1,143,946.36	2,445.0	0	1.2	1.2	
4	J-131	361,449.4	1,143,161.7	2,427.50	360,355.3	1,143,532.00	2,427.3	0.7	4.0	4.7	
5	J-137	361,084.3	1,144,994.8	2,459.00	360,632.0	1,143,355.67	2,458.7	0.3	0.5	0.8	
6	J-116	359,320.1	1,144,413.7	2,380.00	359,553.0	1,142,278.79	2,379.0	1	10.6	11.6	
7	J-98	360,934.4	1,140,059.6	23,98.00	360,632.0	1,143,353.67	2397.1	0.1	5.3	5.4	
8	J-114	360,981.4	1,141,996.6	2,423.00	361,453.9	1,144,739.47	2,424.5	1.5	0.7	2.2	

Table 4.15: Comparison of simulated pressure results with field-measured data

S.N	Sample Node	Simulated Model Pressure (mH2O)	Field measured pressure at customer tap(mH2O)	Total Head Loss between the two locations (m)	The Likely simulated at supply mains node(m)	Error (m)	Time from start (hr)	Scenario
1	J-120	54.61	51.56	1.50	53.11	1.56	8:00-12:00 (early mid noon) 2:00-6:00 (afternoon) 8:00-12:00 (early midnight) 2:00-4:00 (early morning)	Base scenario
2	J-46	38.92	35.91	1.20	37.72	1.82		
3	J-16	58.39	58.55	1.20	57.19	-1.36		
4	J-131	44.80	38.65	4.70	40.10	1.45		
5	J-120	33.60	34.20	0.80	32.80	-1.40		
6	J-116	42.30	38.95	1.60	40.70	1.75		
7	J-105	39.36	35.84	5.40	33.96	-1.88		
8	J-134	52.95	49.15	2.20	50.75	1.61		

Figure 4.8 below is illustrating plots of observed versus simulated values along with minimum and maximum difference between them. The regression model accounts for 95% of the variance this shows that there is a strong correlation between observed and simulated values except the time from and the calculation detailed is attached in Appendix F.

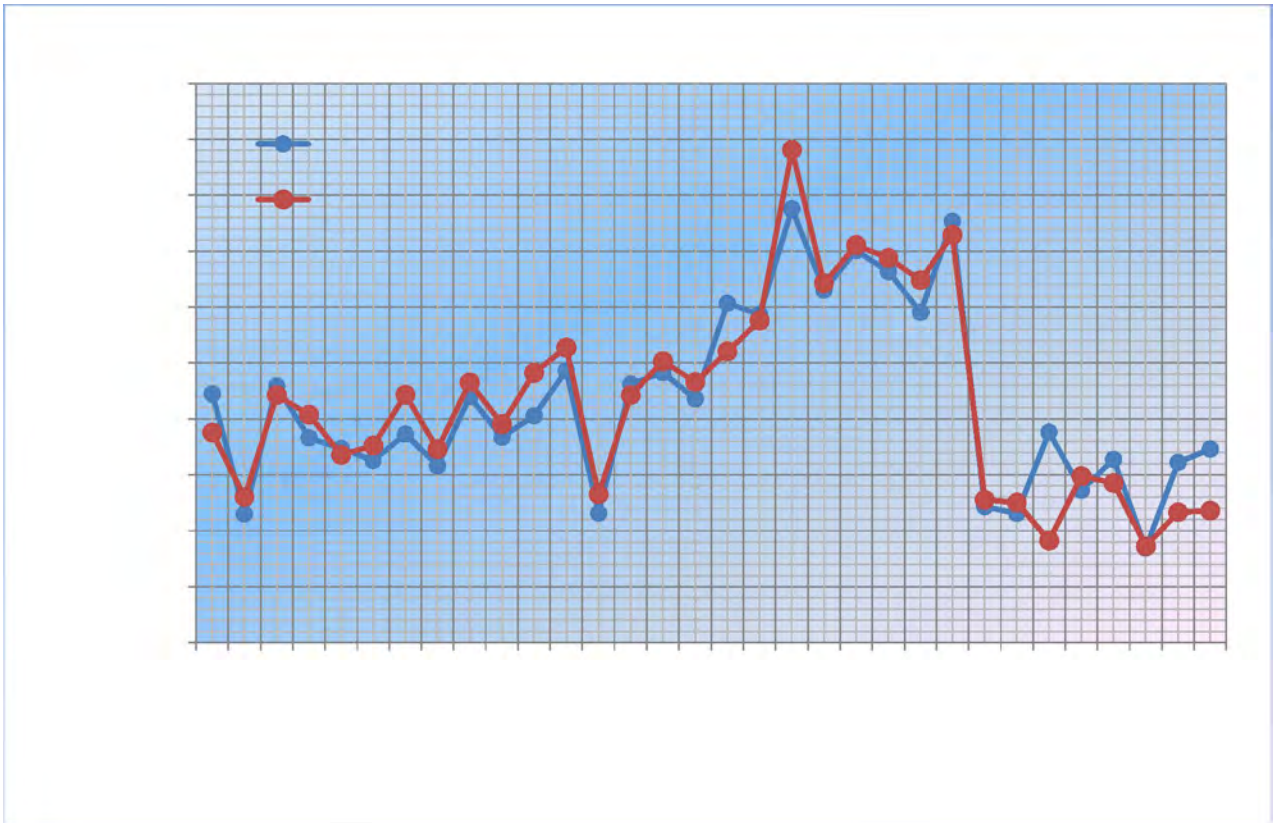


Figure 4.8: Model calibration using time series data

4.3.6 Simulation Results

Single period and extended period simulation were subsequently performed. It was required to run single period simulation at the beginning of the simulation as to observe the model under snap shot situation. In line with this, running single period simulation was helpful while performing preliminary model calibration. However, it should not be used for network assessment as water distribution system is likely to experience variations. Hence, only extended period simulation was exclusively used for entire model calibration and model assessment effort. Demand patterns used in simulating extended period simulation is presented in (Appendix C, D and E).

4.3.6.1 Pressure

Pressure in water distribution system has to be maintained optimum; as to efficiently make water available to each demand category including at instances of fire fighting (high withdrawal period) and as to reduce leakage as well as pipe breakage across the system. The former one is frequently achieved in setting minimum pressure to be maintained at each junction. The later one is achieved differently in setting maximum allowable pressure to be maintained in the system.

According to (Swamee et al., 2008) the minimum design nodal pressures are prescribed to discharge design flows onto the properties. It is based on population served, types of dwellings in the area, and fire fighting requirements. The general consideration is that the water should reach up to the upper stories of low-rise buildings in sufficient quality and pressure, considering fire fighting requirements. In case of high-rise buildings, booster pumps are installed in the water supply system to water for the pressure head requirements. With these considerations, various codes recommend minimum ranging from 8 m to 20 m for residential areas.

Similarly, (Johnson et al., 2009) recommend;

1. Minimum pressures at peak hour demand: sufficient to serve the highest supply point in the network. Typically a mains pressure of not less than 15 to 20 m would be required to serve buildings up to three storeys high. Higher pressures may be necessary in some areas where there are significant numbers of dwellings exceeding three-storey height; but high rise buildings are normally required to have their own boosted supply.

2. Maximum static pressures during low demand periods: typically at night, should be as low as practicable to minimize leakage. For flat areas a maximum static pressure in the range 30 to 45 m is desirable.

For town of Debre Markos water supply service is using an operating pressure which ranges from 15m to 80m. However, there was no defined maximum and minimum pressure ranges set by the office. Therefore, literature based recommendation for optimum operating pressure was used to assess system hydraulic performance.
With regard to current simulation, result for pressure at peak flow is summarized in table 4.16 and detailed in appendix D.

Table 4.16: Distribution of pressure at peak hour flow

Pressure(m)	Nodes(number)	Percentage (%)
15-20	7	5.26
21-30	15	11.28
31-40	31	23.31
31-40	53	37.32
41-50	28	21.05
51-60	26	19.55
61- 70	25	18.79
>70	1	0.75

As shown in table 4.16, 0.75 % of nodes exceed maximum allowable pressure of 70 meter. While 80.45% of nodes are in the permissible pressure range of minimum 15m and maximum 60m.

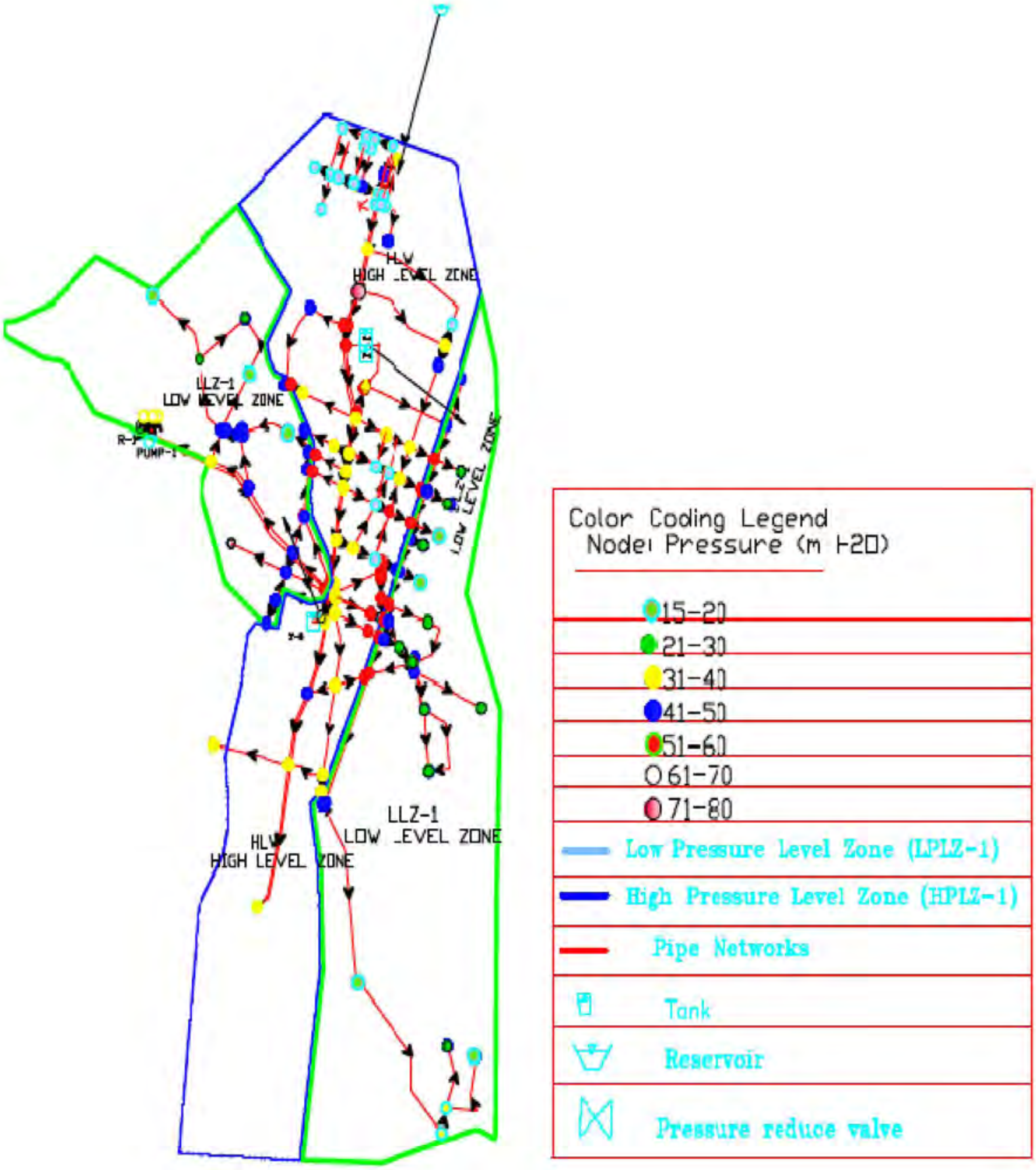


Figure 4.9: Pressure distribution plot peak hour flow

As shown in the figure 4.9 there are extreme high pressure throughout the system mainly due to the topography of the area and the elevation of the distribution reservoir. In few instances nodes positioned in left and right side of network are susceptible to low pressure. Whereas majority of nodes located in central networks are relative perfect loop region receive optimum pressure which doesn't violate minimum or maximum allowable pressure range.

Table 4.17: Pressure distribution during low flow

Pressure(m)	Nodes(number)	Percentage (%)
21-30	1	0.75
31-40	12	9.02
41-50	20	15.04
51-60	22	16.54
61-70	24	18.05
71-80	22	16.54
>80	32	24.06

During low flow typically at mid-night when most of the customers are sleep and not using water distribution system of case study is marked by excessive pressure. As shown in table 4.17 and detailed in appendix E, 24 % of nodes are liable to extremely high pressure. Minimum pressures are also observed at the morning and evening's time which accounts only 9.77% of nodes. Figure 4.10 as shown below the pressure distribution during low flows.

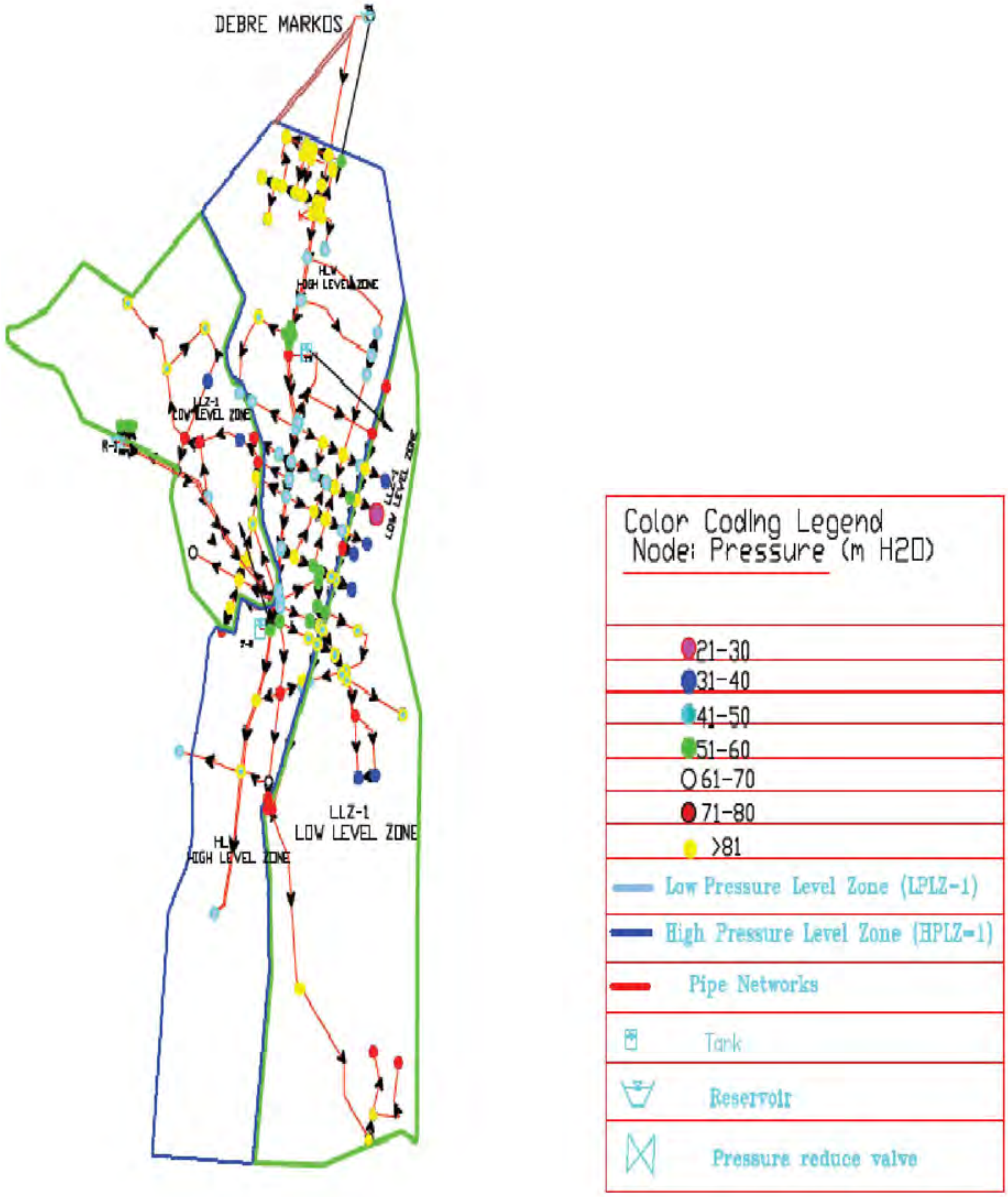


Figure 4.10: Pressure distribution plot during low flow

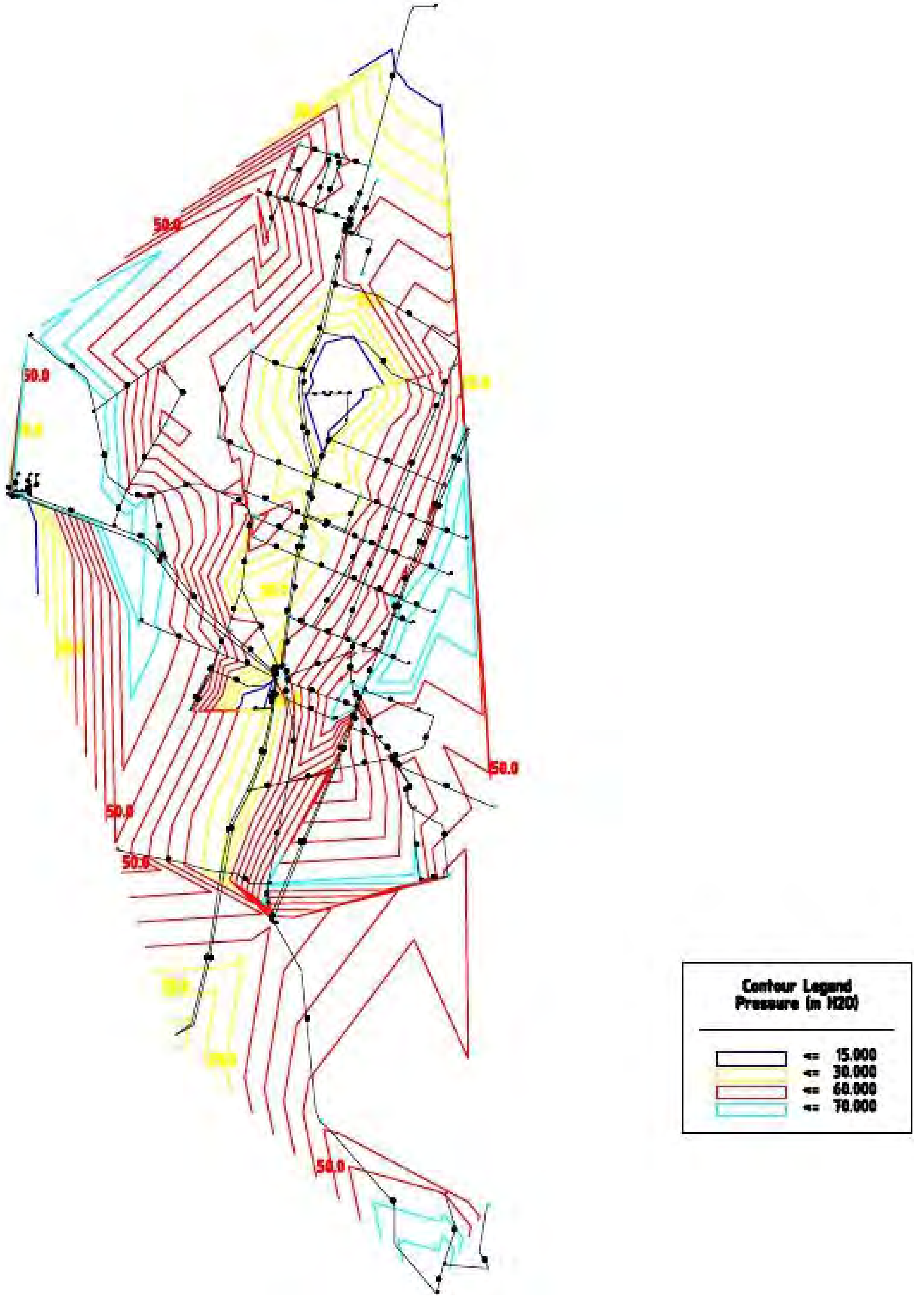


Figure 4.11: Graph showing pressure contour maps at maximum consumption hour

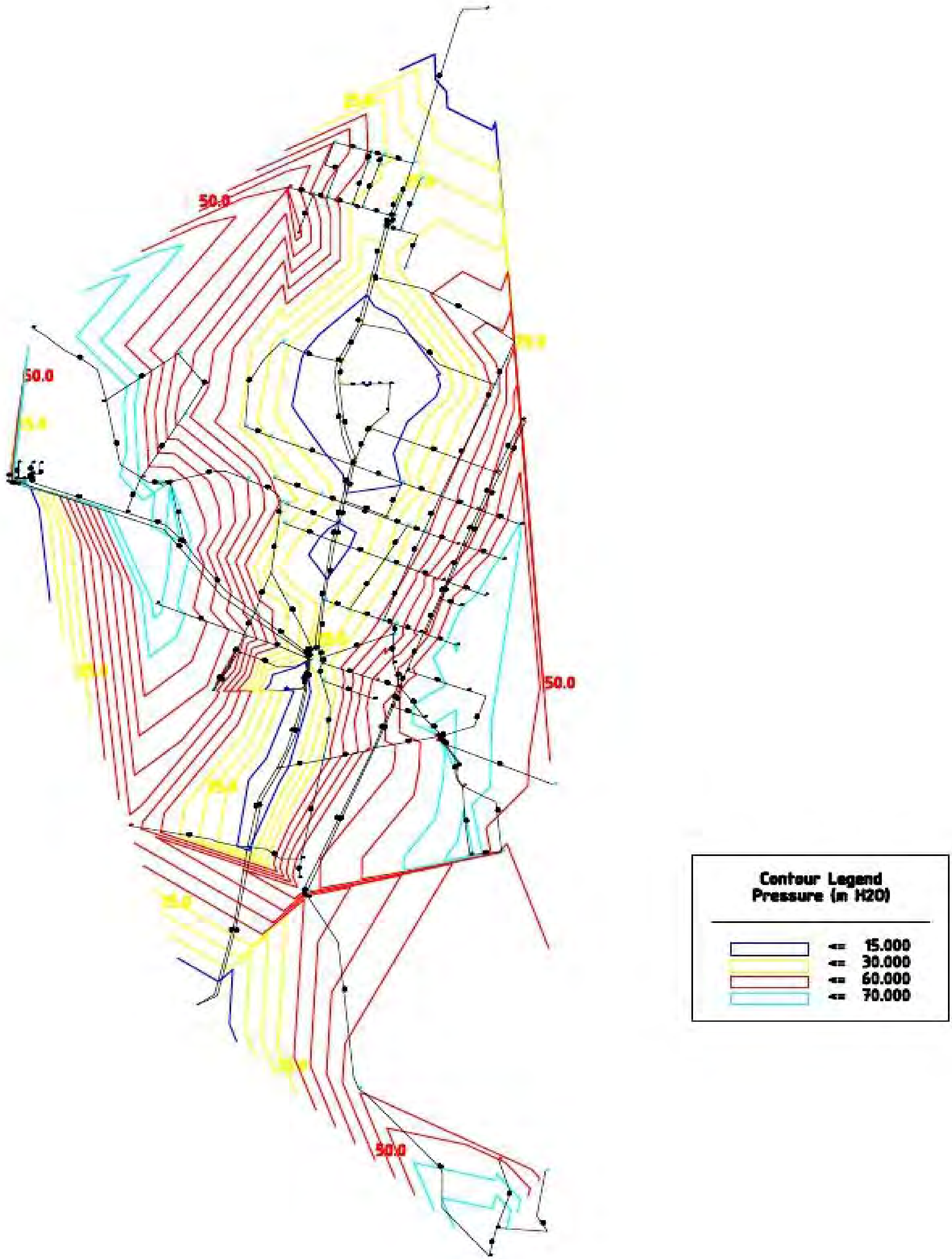


Figure 4.12: Graph showing pressure contour maps at minimum consumption hour

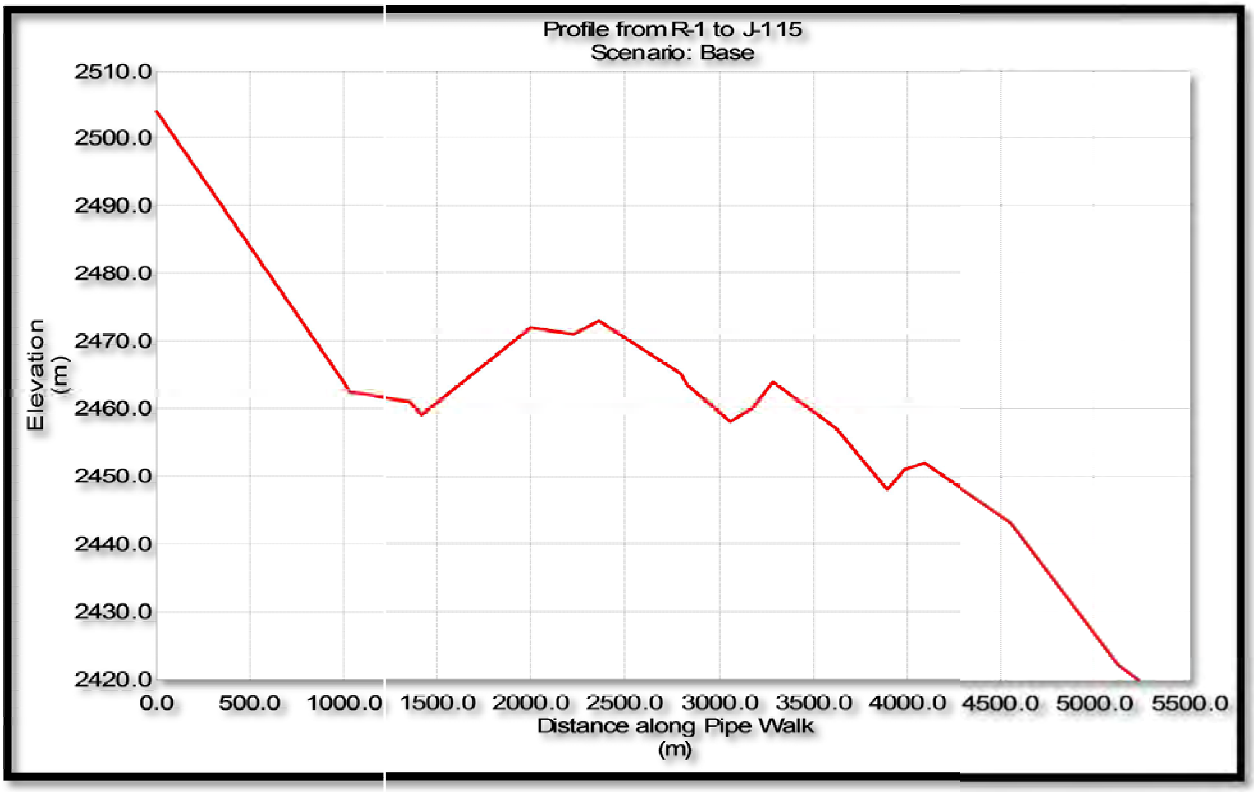


Figure 4.13: Graph showing profile of elevation from reservoir to the main line pipes

As shown in above figure 4.13 shows that, as the distance increase the water loss also increase.

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

- For the water loss analysis a one Ethiopia fiscal year water production and consumption was taken. Despite the low water supply coverage of the town the total water loss is found to be high enough (up to 40%). The total water loss was computed by subtracting the consumption form the water supplied. Three approaches were used to compare the loss among the sub-system (i) the UFW expressed as a percentage, (ii) loss per length of pipes and (iii) loss per connections, comparison using the percentage has reversed the results of the comparison using the loss per length of mains and loss per number of connection. Therefore, even though the total water loss expressed as percentage is an important tool to know the extent of the loss within a given environment, comparison of losses from one location to another using the percentage has limitations as the percentage of loss highly depends on the amount of water produced. This is also the experience of many international comparisons as explained by the international water association (IWA) task forces. Depending on the hierarchy of the network system, both the loss per kilometers length of main pipes (m3/km/day) and loss per connections (liters, connection /day) may be appropriate to measure the loss in the Debre Markos context.

- For the purpose of real loss determination, pursue readings were taken at different elevations and it was found that its magnitude is high in lower elevations, in connections to this burst frequency data were collected from the operation and maintenance section and it was found that the frequency of burst was larger in these high pressure zones. In addition, pressure magnitude was found to be higher during early morning (between 2:00-4:00 A.M) of the time when most of customer is using water.
- Customer water meter test was conducted under high pressure and low pressure scenarios and it was observed that in both tests almost all customers' meters have discrepancy from the calibration meter.
- For the low pressure test case, it was observed that the French brand has the maximum leakage rate 9,561.18 m^3/year in which case a customer can fetch his family demand in

three-hour duration and that is the usual case which customers having high type of meter do as it is observed form their monthly consumption file. Other brands still have significant leakage rate when tested under low pressure exception is the Italian brand which has no any free allowance of water through it. It was estimated that the total amount of water lost in this case was 6,766.37 m^3/year.

- Meter tests under high pressure also showed a discrepancy. Though there is no significant difference between each band a larger volume of water was lost under this condition. The quantity of water lost under this case was found to be 52,600.52m3/year.
- The total loss of water due to meter under registration amounts to 93,057.42 m^3/year and this cover 16.8% of the total water loss of the distribution system.
- Unauthorized consumption volume of water which comprises of theft illegal connection and others was determined as 8,646.02m^3/year which covers 1.56% of the total loss.
- Unbilled metered consumption was collected from the water supply service office data file and is 799m^3/year that accounts only 0.14% of the total loss, this volume of water is consumed by the water supply service office workers for their domestic consumption.
- In the water loss analysis it was found that larger amount of water loss is a physical loss through transmission mains and distribution connections. Water loss in the distribution network is commonly called real loss and the total real loss was estimated as 451,959.56m^3/year and this loss accounts to 81.63% of the total loss.
- High levels of Non-Revenue Water (39.8% of System Input Volume) have a serious impact on Debre Markos water service finances as well as on available water resources in a water scarce environment.
- Every water distribution system has always a minimum amount of leakage that cannot be avoided. This loss even exists in a new distribution network regardless of its volume. As such in this research and avoidable annual real loss of 15,4030m^3/year was estimated.
- Form the above explanations it can be concluded that the main causes of the water loss may be characterized as big caused by leakage and customer meter errors (under recording of meters).
- From the pressure measurement and burst frequency data it can be concluded that greater amount of leakage would occur in higher pressure zones as is proved by many researchers.
- The infrastructure leakage index value (3.56) for this system shows that the current annual real losses are assessed as being around four times as high as the unavoidable

annual real losses for the system. This value may not look very bad in an international comparison context; however it cannot be acceptable in Debre Markos water scarce situation.

5.2 Recommendations

- From the water meter test analysis it can be seen that sample amount of water is lost through meter under registration and customer meter calibration should be made periodically and those meters found non functional and lower performance have to be replaced accordingly.
- When tested in very low pressure the lost of water is found significant in the French brand water meter, and this brand should be totally replaced with other brands.
- One main cause of water meter malfunctioning is age, therefore, water meters having service age of greater than 10 years should be replaced by new water meters.
- From the burst frequency data analysis it was observed that larger bursts and hence larger leakage volume was found in high pressure areas. Hence pressure reducing valves must be provided in such locations.
- In the water loss analysis the main constraint was the distribution network situation. The system was not designed in such a way as to enable to conduct minimum high flow analysis. The network system is not divided in to sub system and district meter areas and it was impossible to conduct this analysis. Though it is not a simple task to divide the existing network into sub systems and district meter areas (DMAS) it is necessary to do this work if water loss analysis is to be carried out in a reliable way.
- The other problem in the analysis of the water loss is data management problem. It is anticipated that the quality of billing data will improve through the use of the new billing database to be made available by the on-going Billing and IT contract. It is strongly recommended that the billing period is increased to at least two months from the present one month so as to provide enough time to Office staff and/ or IT Support Process staff to carry out anomaly checking. Burst type size awareness, location and repair time were not recorded accordingly and hence there must be a proper data management practice in the water supply service office.
- The quality of data used to calculate the System Input Volume must also be improved. It is essential that the treated water supply from the treatment plant and the Sentera and Wotren well field are properly metered.
- The quality of pipe construction and repairs as well as the quality of service connections in WWS must be improved in order to limit Real Losses resulting from an increased System Input Volume. Establishing continuous water supplies and, correspondingly, improved pressure regimes is also essential in controlling Real Losses.

REFERENCES

AWWA.(1987). American Water Works Association leak in water Distribution system.A technical/Economical overview.

AWWA California Nevada section, (1992).Water audit and leak detection guidebook.

AWWA M36 Publication Rewrite.(2004).Water audits and loss control programs.USA, America.

Bentley.(2006). *Efficient Pressure Dependent Demand Model for Large Water Distribution SystemAnalysis.* Haestad Methods Solution Center, Bentley Systems, Incorporated. RetrievedMay5,2008fromftp://ftp2.bentley.com/dist/collateral/whitepaper/WDSA2006 _EfficientPDD haestad_eng_lowres.pdf .

Central Statistical Agency of Ethiopia . (2007). Summary and Statistical Report of the 2007. Population and Housing Census, Population size by Age and Sex, Addis Ababa, Ethiopia, United Nations Population Fund.

Clark,T and Grayman,M. (1998).Water Distribution Network Networks:Selection and Schediling of pipes Rehabilitation Alternatives.PhD Thesis,University of Toronto,Toronto ,Canada.

DARAL_PMRAM CONSULTING.(2014). Non-revenue water reduction, hydraulic modeling and GIS development for Addis Ababa water supply system.

Farley ,M. (2003): Non -revenue water international best practice for assessment.

Farley, M and Trow, S. (2000). Loss in Water Distribution Networks: A Practitioner's Guide to Assessment Monitoring and Control.

Farly,M.,Wyeth,G.,mdGhazali,Z.B.,Istander,A.and Singh,S.(2008).The manger's Non-revenue water Hand Book:Aguide to understanding water losses.UASD& Rahill utilities Berhad&USAID publication,report.

Global Water Supply and Sanitation Assessment. (2000).Managing water for African Cities.

Kingdom,B.,Liemberge,R.,Marine.P.(2006).The challenge of reducing non-revenu water(NRW) in developing countries.

IRC,(1996).Construction Innovation : defecting corrosion in water mains ,1(3).

Labert ,A. (2003).International report on water loss management and techniques .

Lambert ,A.(2001). Review of performance indicators for real losses from water supply systems.

Lambert ,A. (2004).Practical Experience in using the infrastructure leakage index .

Lambert ,A.O.(2002).What do we know about pressure: Leakage relationships in a distribution system?

Lambert, A.O. and Lalonde, A.(2005). Using practical predications of economic intervention frequency to calculate short run economic leakage level, with or without pressure management.

Lambert,A.O and. Mekenzie, R.D. (2005) .Practical experience in using the infrastructure leak

Lambert,A.and Wallace,E.(1993).Accounting for losses-the “bust” and background concepts .The Institution of water and Environment management symposium on leakage control in water industry.

Liemberger ,R., Farley ,M.(2005). Developing a non -revenue water reduction strategy part I: Investigation and assessing water losses.

Liemberger,R., Marin,P. (2006).The challenge of reducing nor -revenue water (NRW) in developing countries. age index.

Mckenzie,R.,Wagelin,W.(2003). Lekage reduction through Pressure management in South Africa.

Mitchell,.V.G,R.G.,M.and T.A.M.(200).Modeling the urban water cycle.

Mulwafuw.,Chigeta,G., Chavula,A., Ferguson ,B.G. and Nkhoma,G.C.(2003). Water demand management in Malawi: Problems and prospects for its promotion.

MWAC.(1999).Managing water for Africa cities Reports and Strategy Background papers.

Obradovie,.D.200.Modelling of demand and losses in real-life water distribution systems.

Pearson ,D. Trow,S. (2005). Leakage Conference Proceedings.

Pilcher, R. (2003). A Practical Approach to developing a sustainable water loss reduction strategy in Sandakan ,Sabah, Malaysia .

RODECO Consulting. (2007). “Debre Markos Water Supply Project”.

Richard ,G.,K.,A.nad L.,R 2000.Monitoring and managing Uanccounted For Water Analysis.

Tabesh, M. Asaqiani, A. H. (2005). A Software tool for non- revenue water calculations in urban water system.

TAHAL Consulting Engineering .(2003). "Addis Ababa Water Supply Project III A (AAWSP-III A)".

Thomas ,M.,Walski.(2003).Advanced water distribution modeling and management 1st edition.

Thornton J. (2003). Managing Leakage by Managing Pressure. Water 21, October 2003

Thornton,J. (2005): System water audits and leak detection .

Tooms, S. and Pilcher ,R. (2006). Practical Guidelines on Efficient Water Loss Management, Water

Tripartite group.(2002). Best practice principles in the economic level of leakage calculation.

Trow,S. and Farly,M.(2005). Developing a strategy for leakage management in water distribution system.

Tuhovcak,L. M Svoboda ,Z Svitak, K Tothova. (2005).The Technical audit of water distribution network using the difference leakage indicators .

Twort,A.C., Ralo F.M., craoleF.M and Retnayaka D.D.(1994). Water supply.

Ulanicki,B. and Bounds ,P.L.M.(2000).open and closed loop pressure control for leakge reduction.

UNEP.(200).Malta Water Service Corporation ,Leakage Control and Unaccounted For Water Analysis.

UN-HABITAT.(2003). Slums of the world :The faces of urban poverty in the new millennimum.

UN-HABITAT .(2008). Water and sanitation initiative fast track capacity building program, Leakage reduction and repair guideline.

Waldron,T.(2005).Where are the advancement in leak detection?

Wallingford., HR. (2003). Handbook for assessmentof catchment water demand and use.

Walski, M., Chase,V., and Savic, A.(2001). Advanced Water distribution Modeling and Management (first edition)

Walski, M., Chase,V., and Savic, A. (2003). Advanced Water distribution Modeling and Management (first edition)

Warren, RP. (2005).The service pipes a forgotten asset in leak detection.

Welday,B.(2005).Comprehensive asset management in water supply and sanitation.

Wnukowska,K. (2004) .Management of Urban Water Resources.

WHO, (2000). World Health Organization, Global Water Supply and Sanitation assessment.

Yates, C.D .(2005).Water Accountability the new way leakage 2005 conference proceedings 2000 report.

Yepes,G.(1995). The World Bank retrieved on July 6, 2004. Reduction of Unaccounted for water, Best practices.

Appendix-A
Water Audit Software Resuts

AWWA Water Loss Control Committee (WLCC) Water Audit Software v3.0

WASv3.0

PURPOSE: This spreadsheet-based water audit tool is designed to help quantify and track water losses associated with water distribution systems and identify areas for improved efficiency and cost recovery

USE: The spreadsheet contains several separate worksheets. Sheets can be accessed using the tabs towards the bottom of the screen, or by clicking the buttons on the left below. Descriptions of each sheet are also given below.

THE FOLLOWING KEY APPLIES THROUGHOUT:

- Value must be entered by user
- Value may be entered by user
- Value calculated based on input data
- These cells contain recommended default values

Please begin by providing the following information, then proceed through each sheet in the workbook:

NAME OF CITY OR UTILITY: Debre Markos   COUNTRY: Ethiopia

REPORTING YEAR: 2015   START DATE (MM/YYYY): Jan-15   END DATE (MM/YYYY): Jun-15

NAME OF CONTACT PERSON: Melaku Abebaw   E-MAIL: melakua841@gmail.com   TELEPHONE: 251910067443
Ext.

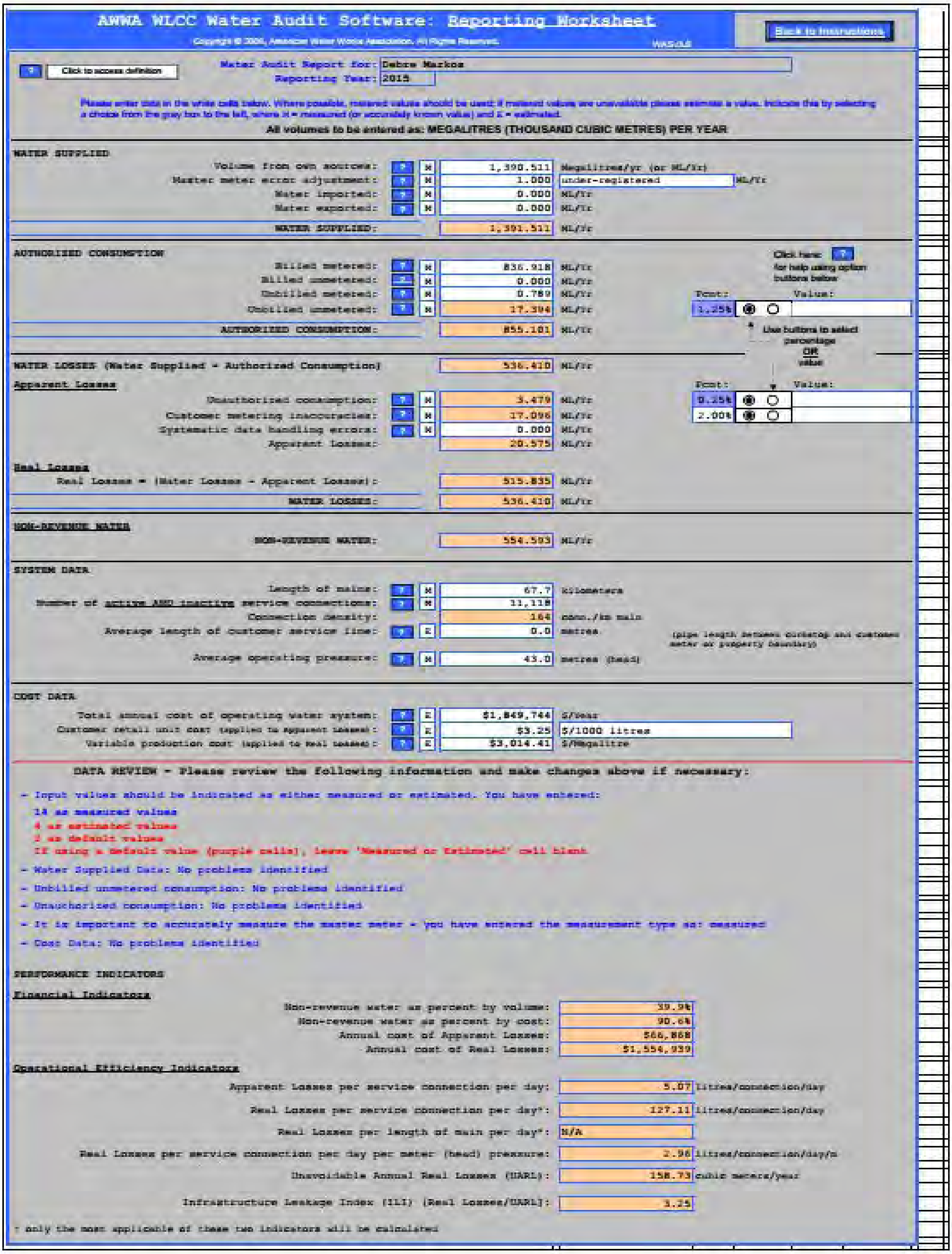
PLEASE SELECT PREFERED REPORTING UNITS FOR WATER VOLUME: Megalitres (thousand cubic metres)

Click to advance to sheet...   Click here: ? for help about units and conversions

Instructions	The current sheet
Reporting Worksheet	Enter the required data on this worksheet to calculate the water balance
Water Balance	The values entered in the Reporting Worksheet are used to populate the water balance
Definitions	Use this sheet to understand terms used in the audit process
Water Loss Standing	Use this sheet to help interpret the results of the performance indicators

If you have questions or comments regarding the software please contact us at: wlc@awwa.org

AWWA Water Loss Control Committee   Instructions 1



AWWA WLCC Water Audit Software: Reporting Worksheet

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Back to Instructions

? Click to access definition

Water Audit Report for: Debre Markos
Reporting Year: 2015

Please enter data in the white cells below. Where possible, metered values should be used; if metered values are unavailable please estimate a value. Indicate this by selecting a choice from the gray box to the left, where M = measured (or accurately known value) and E = estimated.

All volumes to be entered as: MEGALITRES (THOUSAND CUBIC METRES) PER YEAR

WATER SUPPLIED

Volume from own sources:	M	1,390.511	Megalitres/yr (or ML/Yr)
Master meter error adjustment:	M	1.000	under-registered ML/Yr
Water imported:	M	0.000	ML/Yr
Water exported:	M	0.000	ML/Yr
WATER SUPPLIED:		1,391.511	ML/Yr

AUTHORIZED CONSUMPTION

Click here: ? for help using option buttons below

				Pcnt:	Value:
Billed metered:	M	836.918	ML/Yr		
Billed unmetered:	M	0.000	ML/Yr		
Unbilled metered:	M	0.789	ML/Yr		
Unbilled unmetered:	M	17.394	ML/Yr	1.25%	
AUTHORIZED CONSUMPTION:		855.101	ML/Yr		

Use buttons to select percentage OR value

WATER LOSSES (Water Supplied - Authorized Consumption) 536.410 ML/Yr

Apparent Losses

				Pcnt:	Value:
Unauthorized consumption:	M	3.479	ML/Yr	0.25%	
Customer metering inaccuracies:	M	17.096	ML/Yr	2.00%	
Systematic data handling errors:	M	0.000	ML/Yr		
Apparent Losses:		20.575	ML/Yr		

Real Losses

Real Losses = (Water Losses - Apparent Losses):	515.835	ML/Yr
WATER LOSSES:	536.410	ML/Yr

NON-REVENUE WATER

NON-REVENUE WATER: 554.593 ML/Yr

SYSTEM DATA

Length of mains:	M	67.7	kilometers
Number of active AND inactive service connections:	M	11,118	
Connection density:		164	conn./km main
Average length of customer service line:	E	0.0	metres (pipe length between curbstop and customer meter or property boundary)
Average operating pressure:	M	43.0	metres (head)

COST DATA

Total annual cost of operating water system:	E	$1,849,744	$/Year
Customer retail unit cost (applied to Apparent Losses):	E	$3.25	$/1000 litres
Variable production cost (applied to Real Losses):	E	$3,014.41	$/Megalitre

DATA REVIEW - Please review the following information and make changes above if necessary:

- Input values should be indicated as either measured or estimated. You have entered:
  - 14 as measured values
  - 4 as estimated values
  - 3 as default values
  - If using a default value (purple cells), leave 'Measured or Estimated' cell blank
- Water Supplied Data: No problems identified
- Unbilled unmetered consumption: No problems identified
- Unauthorized consumption: No problems identified
- It is important to accurately measure the master meter - you have entered the measurement type as: measured
- Cost Data: No problems identified

PERFORMANCE INDICATORS

Financial Indicators

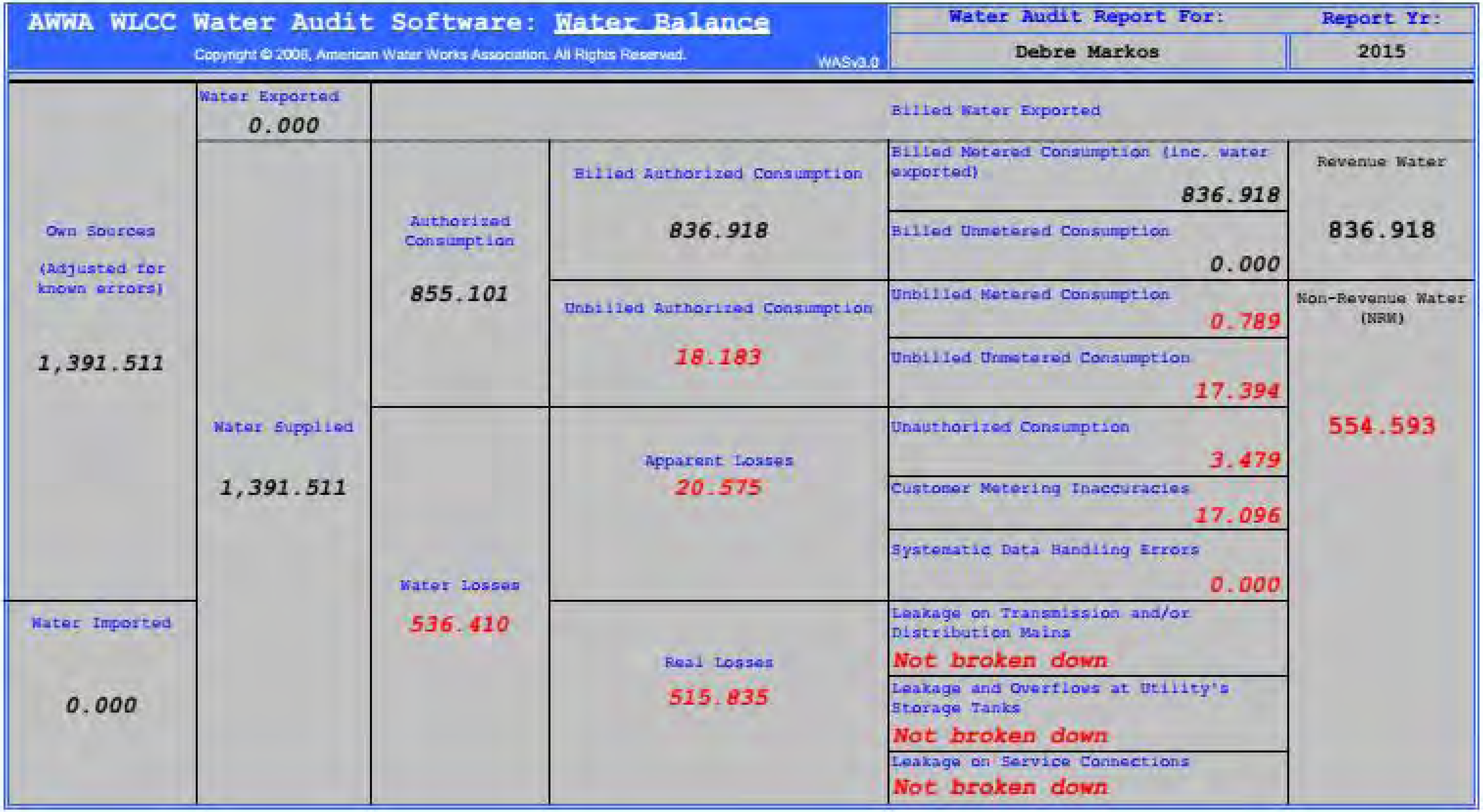
Non-revenue water as percent by volume:	39.9%	
Non-revenue water as percent by cost:	90.6%	
Annual cost of Apparent Losses:	$66,868	
Annual cost of Real Losses:	$1,554,939	

Operational Efficiency Indicators

Apparent Losses per service connection per day:	5.07	litres/connection/day
Real Losses per service connection per day*:	127.11	litres/connection/day
Real Losses per length of main per day*:	N/A	
Real Losses per service connection per day per meter (head) pressure:	2.96	litres/connection/day/m
Unavoidable Annual Real Losses (UARL):	158.73	cubic meters/year
Infrastructure Leakage Index (ILI) [Real Losses/UARL]:	3.25	

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

AWWA Water Loss Control Committee Reporting Worksheet 1


AWWA WLCC Water Audit Software: Water Balance
Copyright © 2006, American Water Works Association. All Rights Reserved.
WASv3.0
Water Audit Report For:
Debre Markos
Report Yr:
2015
Own Sources (Adjusted for known errors)
1,391.511
Water Imported
0.000
Water Exported
0.000
Water Supplied
1,391.511
Billed Water Exported
Authorized Consumption
855.101
Water Losses
536.410
Billed Authorized Consumption
836.918
Unbilled Authorized Consumption
18.183
Apparent Losses
20.575
Real Losses
515.835
Billed Metered Consumption (inc. water exported)
836.918
Billed Unmetered Consumption
0.000
Unbilled Metered Consumption
0.789
Unbilled Unmetered Consumption
17.394
Unauthorized Consumption
3.479
Customer Metering Inaccuracies
17.096
Systematic Data Handling Errors
0.000
Leakage on Transmission and/or Distribution Mains
Not broken down
Leakage and Overflows at Utility's Storage Tanks
Not broken down
Leakage on Service Connections
Not broken down
Revenue Water
836.918
Non-Revenue Water (NRW)
554.593

AWWA WLCC Water Audit Software: Definitions

 WAS v3.0

Back to Instructions

Item Name		Description
Volume from own sources	Find	The volume of treated water input to system from own production facilities
Master meter error adjustment	Find	An estimate or measure of the degree of any inaccuracy that exists in the master meters measuring the Volume from own sources. Please also indicate if this adjustment is because the master meters under-registered (did not capture all the flow) or over-registered (overstated the actual flow)
Water imported	Find	Bulk water purchased to become part of the water supplied. Typically this is water purchased from a neighboring water utility or regional water authority. Be sure to account for any import meter inaccuracy in reporting this volume
Water exported	Find	Bulk water sold and conveyed out of the water distribution system. Typically this is water sold to a neighboring water utility. Be sure to account for any export meter inaccuracy in reporting this volume
Authorized consumption		= billed metered + billed unmetered + unbilled metered + unbilled unmetered The volume of metered and/or unmetered water taken by registered customers, the water supplier and others who are implicitly or explicitly authorized to do so by the water supplier, for residential, commercial and industrial purposes. This does NOT include water sold to neighboring utilities (water exported). Authorized consumption may include items such as fire fighting and training, flushing of mains and sewers, street cleaning, watering of municipal gardens, public fountains, frost protection, building water, etc. These may be billed or unbilled, metered or unmetered.
Billed Authorized Consumption		All consumption that is billed and authorized by the utility. This may include both metered and unmetered consumption. See "Authorized Consumption" for more information.
Unbilled Authorized Consumption		All consumption that is unbilled, but still authorized by the utility. See "Authorized Consumption" for more information.
Billed metered consumption	Find	All metered consumption which is billed. This includes all groups of customers such as domestic, commercial, industrial or institutional. It does NOT include water sold to neighboring utilities (water exported) which is metered and billed. The metered consumption data can be taken directly from billing records for the water audit period. The accuracy of yearly metered consumption data can be refined by including an adjustment to account for customer meter reading lagtime, however additional analysis is necessary to determine the adjustment value, which may or may not be significant.
Billed unmetered consumption	Find	All billed consumption which is calculated based on estimates or norms but is not metered. This might be a very small component in fully metered systems (for example billing based on estimates for the period a customer meter is out of order) but can be the key consumption component in systems without universal metering. It does NOT include water sold to neighboring utilities (water exported) which is unmetered but billed.
Unbilled metered consumption	Find	Metered Consumption which is for any reason unbilled. This might for example include metered consumption of the utility itself or water provided to institutions free of charge. It does NOT include water sold to neighboring utilities (water exported) which is metered but unbilled.
Unbilled unmetered consumption	Find	Any kind of Authorized Consumption which is neither billed nor metered. This component typically includes items such as fire fighting, flushing of mains and sewers, street cleaning, frost protection, etc. In most water utilities it is a small component which is very often substantially overestimated. It does NOT include water sold to neighboring utilities (water exported) which is unmetered and unbilled – an unlikely case. This component has many sub-components of water use which are often tedious to identify and quantify. Because of this, and the fact that it is usually a small portion of the water supplied, it is recommended that the auditor apply the default value of 1.25% of the volume from own sources. Select the default percentage to enter this value. If the water utility already has well validated data that gives a value substantially higher or lower than the default volume, then the auditor should enter their own volume. However the default approach is recommended for most water utilities. Note that a value of zero is not permitted, since all water utilities have some volume of water in this component occurring in their system.
WATER LOSSES		= apparent losses + real losses The difference between System Input and Authorized Consumption. Water losses can be considered as a total volume for the whole system, or for partial systems such as transmission or distribution systems, or individual zones. Water Losses consist of Real Losses and Apparent Losses.

Apparent Losses	= unauthorized consumption + meter under-registration + data handling errors Includes all types of inaccuracies associated with customer metering as well as data handling errors (meter reading and billing), plus unauthorized consumption (theft or illegal use). NOTE: Over-registration of customer meters, leads to under-estimation of Real Losses. Under-registration of customer meters, leads to over-estimation of Real Losses.
Unauthorized consumption	Includes water illegally withdrawn from hydrants, illegal connections, bypasses to consumption meter or meter reading equipment tampering. While this component has a direct impact on revenue, in most water utilities the volume is low and it is recommended that the auditor apply a default value of 0.25% of the volume from own sources. If the auditor has well validated data that indicates the volume from unauthorized consumption is substantially higher or lower than that generated by the default value then this value can be entered. However, for most water utilities it is recommended to apply the default value. Note that a value of zero will not be accepted since all water utilities have some volume of unauthorized consumption occurring in their system.
Customer metering inaccuracies	Apparent water losses caused by the collective under-registration of customer water meters. Many customer water meters will wear as large cumulative volumes of water are passed through them over time. This causes the meters to under-register. The auditor has two options for entering data for this component of the audit. The auditor can enter a percentage under-registration (typically an estimated value), this will apply the selected percentage to the two categories of metered consumption to determine the volume of water not recorded due to customer meter inaccuracy. Alternatively, if the auditor has substantial data from meter testing to arrive at their own volumes of such losses, this volume may be entered directly. Note that a value of zero will be accepted but an alert will appear asking if the customer population is unmetered. Since all metered systems have some degree of inaccuracy, then a positive value should be entered. A value of zero in this component is valid only if the water utility does not meter its customer population.
Systematic data handling errors	Apparent water losses caused by systematic data handling errors in the meter reading and billing system.
Real Losses	Physical water losses from the pressurized system and the utility's storage tanks, up to the point of customer consumption. In metered systems this is the customer meter, in unmetered situations this is the first point of consumption (stop tap/tap) within the property. The annual volume lost through all types of leaks, breaks and overflows depends on frequencies, flow rates, and average duration of individual leaks, breaks and overflows.
Non-revenue water	= Apparent Losses + Real Losses + Unbilled Metered + Unbilled Unmetered Water which does not provide any revenue to the utility
Revenue Water	Water which is charged to customers to provide revenue to the utility.
Length of mains	Length of all pipelines (except service connections) in the system starting from the point of system input metering (for example at the outlet of the treatment plant). It is also recommended to include in this measure the total length of fire hydrant lead pipe. Hydrant lead pipe is the pipe branching from the water main to the fire hydrant. Fire hydrant leads are typically of a sufficiently large size that is more representative of a pipeline than a service connection. The average length of hydrant leads across the entire system can be assumed if not known, and multiplied by the number of fire hydrants in the system, which can also be assumed if not known. This value can then be added to the total pipeline length. Total length of mains can therefore be calculated as: Length of Mains, miles = (total pipeline length, miles) + [((average fire hydrant lead length, ft) x (number of fire hydrants)) / 5,280 ft/mile] or Length of Mains, kilometres = (total pipeline length, kilometres) + [((average fire hydrant lead length, metres) x (number of fire hydrants)) / 1,000 metres/kilometre]
Number of active AND inactive service connections	Number of service connections, main to curb stop. Please note that this includes the actual number of distinct piping connections whether active or inactive. This may differ substantially from the number of Customers (or number of accounts)
Connection density	=number of connections / length of mains
Average length of customer service line	This is entered for unmetered services and in cold or other areas where meters are installed inside homes and buildings. It is the length of customer service line either between the utility's service connection (often at the curbstop) and the meter, or to the building line (first point of customer consumption) if customers are unmetered. Note that the length of service connection between the main and customer service line is owned by the utility and its length and potential leakage is accounted for in the UARL formula by the number of service connections.
Average operating pressure	The average pressure may be approximated when compiling the preliminary water audit. Once routine water auditing has been established, a more accurate assessment of average pressure should be pursued. If the water utility infrastructure is recorded in a Geographical Information System (GIS) the average pressure at many locations in the distribution system can be readily obtained. If a GIS does not exist, a weighted average of pressure data can be calculated from water pressure measured at various fire hydrants scattered across the water distribution system.

AWWA Water Loss Control Committee Definitions 2

Total annual cost of operating the water system — Find

These costs include those for operations, maintenance and any annually incurred costs for long-term upkeep of the system, such as repayment of capital bonds for infrastructure expansion or improvement. Typical costs include employee salaries and benefits, materials, equipment, insurance, fees, administrative costs and all other costs that exist to sustain the drinking water supply. These costs should not include any costs to operate wastewater, biosolids or other systems outside of drinking water.

Customer retail unit cost — Find

The Customer Retail Unit Cost represents the charge that customers pay for water service. This unit cost is applied to the components of apparent loss, since these losses represent water reaching customers but not (fully) paid for. It is important to compile these costs per the same unit cost basis as the volume measure included in the water audit. For example, if all water volumes are measured in million gallons, then the unit cost should be dollars per million gallon ($/mil gal). The software allows the user to select the units that are charged to customers (either $/1,000 gallons, $/hundred cubic feet or $/1,000 litres) and automatically converts these units to the units that appear in the "WATER SUPPLIED" box. Since most water utilities have a rate structure that includes a variety of different costs based upon class of customer, a weighted average of individual costs and number of customer accounts in each class can be calculated to determine a single composite cost that should be entered into this cell.

Variable production cost (applies to Real Losses) — Find

The cost to produce and supply the next unit of water. (E.g., $/million gallons) This cost is determined by calculating the summed unit costs for ground and surface water treatment and all power used for pumping from the source to the customer. It should also include the unit cost of bulk water purchased as an import if applicable.

Unavoidable Annual Real Losses (UARL) — Find

UARL (gallons/day)=(5.41Lm + 0.15Nc + 7.5Lp) xP,
or
UARL (litres/day)=(18.0Lm + 0.8Nc + 25.0Lp) xP

where:
Lm = length of mains, (miles or kilometres)
Nc = number of service connections
Lp = total length of private pipe, (miles or km)
= Nc x average distance of private pipe

The UARL is a theoretical reference value representing the technical low limit of leakage that could be achieved if all of today's best technology could be successfully applied. It is a key variable in the calculation of the Infrastructure Leakage Index (ILI). It is not necessary that water utilities set this level as the target level of leakage, unless water is unusually expensive, scarce or both.

NOTE: The UARL calculation has not yet been fully proven as effective for very small water distribution systems. If,
(Lm x 32) + Nc < 3000 (gallons per day) or
(Lm x 20) + Nc < 3000 (litres per day)
then the calculated UARL value may not be valid. The software does not display a value of UARL or ILI if either of these conditions is true.

Infrastructure Leakage Index (ILI) — Find

The ratio of the Current Annual Real Losses (Real Losses) to the Unavoidable Annual Real Losses (UARL). The ILI is a highly effective performance indicator for comparing (benchmarking) the performance of utilities in operational management of real losses.

Use of Option Buttons — Find

To use the percent value choose this button

To enter a value choose this button and enter the value in the cell to the right

Units and Conversions

The user may develop an audit based on one of three unit selections:
1) Million Gallons (US)
2) Megalitres (Thousand Cubic Metres)
3) Acre-feet
Once this selection has been made in the instructions sheet, all calculations are made on the basis of the chosen units. Should the user wish to make additional conversions, a unit converter is provided below (use drop down menus to select units from the yellow unit boxes):

Enter Units:	Convert From...		Converts to.....	
1	Million Gallons (US)	=	1	Million Gallons (US)

(conversion factor = 1)

Appendix -B: Input parameter for water CAD6.5

Node Name	Node Title	External Demand (l/s)	Junction Elevation (m)	External Grade (m)
J-1		-	2,465.0	
J-10		0.19	2,461.5	
J-100		-	2,449.0	
J-101		-	2,449.0	
J-102		-	2,452.0	
J-103		-	2,448.0	
J-104		0.34	2,448.0	
J-105		0.35	2,446.0	
J-106		-	2,395.0	
J-107		-	2,380.0	
J-108		-	2,478.0	
J-109		0.93	2,478.0	
J-11		0.32	2,459.5	
J-110		-	2,424.0	
J-111		0.30	2,423.0	
J-112		0.44	2,416.0	
J-113		0.14	2,422.0	
J-114		0.79	2,423.0	
J-115		2.38	2,420.0	
J-116		3.84	2,380.0	
J-117		0.66	2,440.0	
J-118		-	2,450.0	
J-119		5.76	2,453.0	
J-12		0.45	2,460.0	
J-120		-	2,461.0	
J-121		2.46	2,392.0	
J-122		2.20	2,410.0	
J-123		-	2,472.0	
J-124		-	2,442.0	
J-125		-	2,420.0	
J-126		1.23	2,380.0	
J-127		3.35	2,410.0	
J-129		-	2,422.5	
J-13		0.47	2,455.5	
J-130		-	2,420.0	
J-131		-	2,427.5	
J-132		-	2,427.5	
J-133		-	2,422.0	
J-134		-	2,462.5	
J-135		-	2,459.0	
J-136		-	2,456.0	
J-137		-	2,459.0	
J-139		-	2,401.0	
J-14		0.23	2,451.5	
J-15		1.03	2,434.0	
J-16		0.61	2,445.0	

Node Name	Node Title	External Demand	Junction Elevation	External Grade
J-17		0.52	2,443.5	
J-18		0.89	2,432.0	
J-19		0.45	2,459.8	
J-2		3.58	2,471.0	
J-20		0.78	2,460.0	
J-21		2.40	2,445.0	
J-22		1.09	2,441.0	
J-23		5.96	2,454.0	
J-24		2.35	2,442.0	
J-25		1.12	2,427.5	
J-26		1.14	2,409.0	
J-27		0.67	2,427.5	
J-28		2.90	2,434.0	
J-29		1.04	2,419.5	
J-3		0.26	2,459.0	
J-30		0.51	2,442.0	
J-31		0.40	2,441.0	
J-32		0.35	2,420.0	
J-33		0.48	2,464.0	
J-34		0.59	2,408.0	
J-35		0.63	2,450.0	
J-36		0.15	2,451.5	
J-37		0.21	2,446.0	
J-38		0.45	2,407.0	
J-39		0.93	2,457.0	
J-4		1.55	2,463.5	
J-40		0.03	2,448.0	
J-41		0.09	2,438.0	
J-42		0.17	2,448.0	
J-43		-	2,450.0	
J-44		0.68	2,446.0	
J-45		0.04	2,449.0	
J-46		0.66	2,426.0	
J-47		0.09	2,452.0	
J-48		-	2,422.0	
J-49		0.57	2,410.0	
J-5		1.60	2,458.0	
J-50		1.11	2,414.0	
J-51		1.04	2,416.0	
J-52		1.42	2,396.0	
J-53		0.76	2,464.0	
J-54		1.47	2,466.0	
J-55		0.03	2,434.0	
J-56		0.84	2,404.0	
J-57		0.39	2,422.5	
J-58		0.39	2,421.0	
J-59		0.56	2,410.0	
J-6		0.19	2,458.0	
J-60		1.69	2,402.0	
J-61		0.82	2,400.0	
J-62		2.27	2,367.0	
J-63		0.24	2,363.5	
J-64		-	2,368.0	
J-65		-	2,367.0	

Node Name	Node Title	External Demand	Junction Elevation	External Grade
J-66		0.91	2,438.0	
J-67		0.08	2,365.5	
J-68		0.24	2,367.5	
J-69		0.28	2,433.0	
J-7		0.45	2,461.0	
J-70		-	2,460.0	
J-71		0.41	2,450.0	
J-72		1.02	2,423.0	
J-73		1.36	2,418.0	
J-74		-	2,443.0	
J-75		-	2,451.0	
J-76		0.07	2,452.0	
J-77		0.45	2,414.0	
J-78		0.51	2,417.0	
J-79		0.55	2,405.0	
J-8		0.41	2,457.5	
J-80		0.13	2,434.0	
J-81		-	2,422.0	
J-82		0.85	2,405.0	
J-83		0.51	2,401.0	
J-84		0.83	2,397.0	
J-85		0.84	2,408.0	
J-86		1.88	2,417.0	
J-87		0.41	2,396.0	
J-88		2.25	2,400.0	
J-89		0.66	2,374.0	
J-9		0.34	2,462.0	
J-90		3.81	2,374.0	
J-91		1.24	2,391.0	
J-92		4.19	2,478.0	
J-93		3.10	2,462.0	
J-94		2.39	2,431.0	
J-95		2.45	2,446.0	
J-96		-	2,471.0	
J-97		-	2,473.0	
J-98		3.00	2,398.0	
J-99		-	2,460.0	
Pump-1		----	2,374.0	2,375.0
RV-1		----	2,480.0	
RV-2		----	2,420.0	
RV-3		-	2,402.0	
RV-5		-	2,401.0	
T-1	R1 + R2 = 1000	----	2,480.0	2,485.0
T-2		----	2,472.5	2,475.5
T-3		----	2,499.0	2,500.0
Pump-2			2,440.0	2,509.1

Appendix -C : Extended Period Simulation Pipes line Results

Nodes at Peak Hour

Pipe Name	Node Names		Length	Dia.	Roughness Coeff.	Flow rate	Head loss	Velocity	HL/1000	Remark
	Node 1	Node 2	(m)	(mm)		(l/s)	(m)	(m/s)	(m/m)	
P-1	J-2	J-5	516	150	56	0.5	0.0	0.0	0.0	Existing-uPVC
P-10	J-7	J-9	122	100	75	0.7	0.1	0.1	0.4	Existing-uPVC
P-100	J-78	J-79	174	100	56	2.7	1.3	0.3	7.6	Existing-uPVC
P-101	J-78	J-72	252	75	56	0.8	0.9	0.2	3.4	Existing-uPVC
P-102	J-80	J-55	61	150	56	2.2	0.0	0.1	0.7	Existing-uPVC
P-103	J-80	J-81	169	150	56	0.2	-	0.0	0.0	Existing-uPVC
P-104	J-81	J-125	139	150	56	0.2	-	0.0	0.0	Existing-uPVC
P-105	J-136	J-16	192	100	103	3.4	0.8	0.4	4.0	New-uPVC
P-106	J-77	J-79	209	150	56	7.9	1.7	0.5	8.1	Existing-uPVC
P-107	J-79	J-83	122	150	56	9.5	1.4	0.5	11.2	Existing-uPVC
P-108	J-83	J-72	330	150	56	-6.5	1.8	0.4	5.6	Existing-uPVC
P-109	@~RV-5	J-139	2	150	103	13.8	0.0	0.8	7.3	New-uPVC
P-11	J-7	J-10	144	100	75	-0.8	0.1	0.1	0.5	Existing-uPVC
P-110	J-84	J-85	582	100	56	1.7	2.0	0.2	3.4	Existing-uPVC
P-111	J-84	J-87	266	50	56	0.3	1.1	0.2	4.3	Existing-uPVC
P-112	J-74	J-133	576	250	56	9.6	0.6	0.2	1.0	Existing-uPVC
P-113	J-72	J-86	886	150	56	3.3	1.4	0.2	1.6	Existing-uPVC
P-114	J-86	@~RV-2	41	250	56	22.4	0.2	0.5	4.6	Existing-uPVC
P-115	J-89	J-88	616	150	56	4.6	1.8	0.3	2.9	Existing-uPVC
P-116	J-89	J-91	413	150	56	2.5	0.4	0.1	1.0	Existing-uPVC
P-117	J-90	J-89	165	150	56	8.5	1.5	0.5	9.1	Existing-uPVC
P-118	J-54	J-93	478	150	56	3.2	0.7	0.2	1.5	Existing-uPVC
P-119	J-93	J-94	596	100	56	4.9	14.1	0.6	23.6	Existing-uPVC
P-12	J-10	J-11	52	100	75	1.8	0.1	0.2	2.1	Existing-uPVC
P-120	J-93	J-92	966	100	56	0.7	0.6	0.1	0.7	Existing-uPVC
P-121	T-3	J-134	1036	450	96	185.3	5.0	1.2	4.9	New-Fe
P-122	J-96	J-97	132	400	96	121.1	0.5	1.0	3.9	New-Fe
P-123	J-96	J-2	22	200	103	30.7	0.2	1.0	8.0	New-uPVC
P-124-CV	J-97	T-1	142	300	96	59.0	0.6	0.8	4.2	New-Fe
P-125	J-97	J-1	437	350	96	62.1	1.0	0.7	2.2	New-Fe
P-126	J-99	T-3	1786	400	96	117.5	6.6	0.9	3.7	New-Fe
P-127	@~Pump-2	J-99	1154	400	96	117.5	4.3	0.9	3.7	New-Fe
P-128	J-71	J-117	255	100	56	1.4	0.6	0.2	2.2	Existing-uPVC
P-129	J-98	J-90	1198	250	56	16.2	3.0	0.3	2.5	Existing-uPVC
P-13	J-10	J-12	254	100	75	-3.0	1.4	0.4	5.5	Existing-uPVC
P-130	Pump-1	J-100	1776	200	56	19.2	18.1	0.6	10.2	Existing-uPVC

Pipe Name	Node Names		Length	Dia.	Roughness	Flow rate	Head loss	Velocity	HL/1000	Remark
P-131	J-102	J-101	45	200	103	35.9	0.5	1.1	10.7	New-uPVC
P-132	J-40	J-45	36	250	103	18.2	0.0	0.4	1.0	New-uPVC
P-133	J-42	J-40	22	250	103	18.2	0.0	0.4	1.0	New-uPVC
P-134	J-33	J-104	276	100	56	0.7	0.2	0.1	0.6	Existing-uPVC
P-135	J-105	J-70	291	100	56	-0.7	0.2	0.1	0.7	Existing-uPVC
P-136	J-106	J-73	416	100	56	4.2	7.5	0.5	18.0	Existing-uPVC
P-137	J-61	RV-4	18	150	103	19.2	0.3	1.1	13.6	New-uPVC
P-138	J-107	J-106	214	150	103	13.4	1.5	0.8	7.0	New-uPVC
P-139	J-136	J-13	65	100	103	3.2	0.2	0.4	3.5	New-uPVC
P-14	J-11	J-13	249	100	75	-2.2	0.8	0.3	3.2	Existing-uPVC
P-140	J-108	J-110	675	300	96	59.0	2.8	0.8	4.2	New-Fe
P-141	J-110	J-29	326	50	56	0.4	1.8	0.2	5.5	Existing-uPVC
P-142	J-1	J-109	184	300	56	1.9	-	0.0	0.0	Existing-uPVC
P-143	J-112	J-77	100	250	103	47.3	0.6	1.0	6.0	New-uPVC
P-144	J-112	J-82	322	50	56	0.6	4.6	0.3	14.4	Existing-uPVC
P-145	J-129	J-112	229	250	103	48.8	1.5	1.0	6.4	New-uPVC
P-146	J-111	J-125	113	75	56	-0.2	0.0	0.0	0.2	Existing-uPVC
P-147	J-110	J-29	372	75	103	1.8	1.8	0.4	4.8	New-uPVC
P-148	J-83	J-82	463	75	103	1.1	1.0	0.3	2.1	New-uPVC
P-149	J-77	J-78	119	150	103	8.5	0.4	0.5	3.0	New-uPVC
P-15	J-11	J-14	134	100	75	3.3	0.9	0.4	6.7	Existing-uPVC
P-150	J-5	J-28	928	100	103	2.9	2.7	0.4	2.9	New-uPVC
P-151	J-133	J-115	109	250	56	4.9	0.0	0.1	0.3	Existing-uPVC
P-152	J-133	J-93	284	150	103	4.7	0.3	0.3	1.0	New-uPVC
P-153	J-106	J-126	502	150	103	15.7	4.7	0.9	9.4	New-uPVC
P-154	RV-2	J-98	1151	250	56	22.4	5.3	0.5	4.6	Existing-uPVC
P-155	J-2	J-119	300	150	103	14.6	2.5	0.8	8.2	New-uPVC
P-156	J-118	J-95	132	150	56	2.2	0.1	0.1	0.7	Existing-uPVC
P-157	J-119	J-95	538	100	103	2.8	1.5	0.4	2.8	New-uPVC
P-158	J-6	J-23	247	100	103	8.6	5.5	1.1	22.4	New-uPVC
P-159	J-72	J-86	910	250	103	22.9	1.4	0.5	1.6	New-uPVC
P-16	J-14	J-17	68	100	75	5.0	1.0	0.6	14.4	Existing-uPVC
P-160	J-72	J-78	263	250	103	-34.0	0.9	0.7	3.3	New-uPVC
P-161	J-78	J-77	139	250	103	-29.9	0.4	0.6	2.6	New-uPVC
P-162	J-110	J-132	263	250	103	56.4	2.2	1.2	8.3	New-uPVC
P-163	J-93	J-92	1000	200	103	7.8	0.6	0.3	0.6	New-uPVC
P-164	J-47	J-53	191	300	96	17.3	0.1	0.2	0.4	New-Fe
P-165	J-53	J-54	490	200	103	15.0	1.0	0.5	2.1	New-uPVC
P-166	J-84	J-87	277	150	103	10.0	1.1	0.6	4.1	New-uPVC
P-167	J-87	J-121	396	100	103	5.0	3.2	0.6	8.1	New-uPVC
P-168	J-87	J-122	473	100	103	4.5	3.2	0.6	6.8	New-uPVC
P-169	J-121	J-122	159	100	56	0.0	-	-	-	Existing-uPVC
P-17	J-14	J-16	379	100	75	-2.2	1.2	0.3	3.0	Existing-uPVC
P-170	J-120	J-137	63	400	96	170.6	0.5	1.4	7.4	New-Fe
P-171	J-120	J-7	18	150	103	4.3	0.0	0.2	0.8	New-uPVC
P-172	J-123	J-96	232	400	96	151.8	1.4	1.2	6.0	New-Fe
P-173	J-123	J-124	767	150	103	9.5	2.8	0.5	3.7	New-uPVC

Pipe Name	Node Names		Length	Dia.	Roughness	Flow rate	Head loss	Velocity	HL/1000	Remark
P-174	J-124	J-28	151	100	56	3.0	1.5	0.4	9.8	Existing-uPVC
P-175	J-102	J-49	482	100	103	4.8	3.6	0.6	7.4	New-uPVC
P-176	J-46	J-61	537	150	103	20.5	8.3	1.2	15.4	New-uPVC
P-177	J-126	J-116	582	100	103	7.8	10.9	1.0	18.7	New-uPVC
P-178	J-129	J-131	567	250	103	-51.3	4.0	1.1	7.0	New-uPVC
P-179	J-132	J-131	246	250	103	53.2	1.8	1.1	7.5	New-uPVC
P-18	J-17	J-15	130	100	75	2.1	0.4	0.3	2.9	Existing-uPVC
P-180	J-126	J-127	445	100	103	5.4	4.2	0.7	9.3	New-uPVC
P-181	J-73	J-127	419	75	103	1.4	1.4	0.3	3.3	New-uPVC
P-182	J-49	J-50	176	100	103	2.1	0.3	0.3	1.6	New-uPVC
P-183	@~RV-3	J-106	145	100	56	6.5	5.9	0.8	40.6	Existing-uPVC
P-184	J-1	J-6	282	250	103	37.1	1.1	0.8	3.8	New-uPVC
P-185	J-6	J-70	144	250	103	22.6	0.2	0.5	1.5	New-uPVC
P-186	J-70	J-33	131	250	103	22.2	0.2	0.5	1.5	New-uPVC
P-187	J-33	J-42	647	250	103	21.5	0.9	0.4	1.4	New-uPVC
P-188	J-54	J-93	534	200	103	11.8	0.7	0.4	1.4	New-uPVC
P-189	@~RV-4	J-107	325	150	103	19.2	4.4	1.1	13.6	New-uPVC
P-19	J-17	J-18	236	75	75	1.8	2.1	0.4	9.0	Existing-uPVC
P-190	J-137	J-123	577	400	96	161.3	3.9	1.3	6.7	New-Fe
P-191	J-137	J-3	18	150	103	9.4	0.1	0.5	3.6	New-uPVC
P-192	J-84	J-139	66	100	56	-2.2	0.4	0.3	5.6	Existing-uPVC
P-194	J-139	J-84	70	150	103	11.5	0.4	0.7	5.3	New-uPVC
P-2	T-1	@~RV-1	107	300	56	59.0	1.2	0.8	11.4	Existing-uPVC
P-20	J-8	J-19	312	100	75	0.9	0.2	0.1	0.6	Existing-uPVC
P-21	J-8	J-21	292	100	75	4.9	4.1	0.6	13.9	Existing-uPVC
P-22	J-4	J-20	257	100	56	4.7	5.7	0.6	22.2	Existing-uPVC
P-23	J-20	J-22	205	100	56	2.3	1.2	0.3	5.9	Existing-uPVC
P-24	J-20	J-23	217	75	56	0.8	0.8	0.2	3.5	Existing-uPVC
P-25	J-22	J-24	556	100	56	-1.7	1.8	0.2	3.2	Existing-uPVC
P-26	J-22	J-25	202	100	56	2.3	1.2	0.3	5.8	Existing-uPVC
P-27	J-132	J-26	220	75	56	2.3	5.4	0.5	24.3	Existing-uPVC
P-28	J-132	J-131	217	75	56	1.3	1.8	0.3	8.5	Existing-uPVC
P-29	J-24	J-124	134	100	56	-6.5	5.3	0.8	39.6	Existing-uPVC
P-3	J-2	J-1	593	200	56	8.4	1.3	0.3	2.2	Existing-uPVC
P-30	J-83	RV-5	1	150	103	13.8	0.0	0.8	7.3	New-uPVC
P-31	J-22	J-30	221	75	56	-0.6	0.4	0.1	1.7	Existing-uPVC
P-32	J-6	J-23	222	75	56	2.4	5.5	0.5	24.9	Existing-uPVC
P-33	J-23	J-37	118	75	56	-0.3	0.1	0.1	0.7	Existing-uPVC
P-34	J-30	J-31	218	75	103	-2.4	1.8	0.5	8.2	New-uPVC
P-35	J-31	J-32	190	100	56	0.7	0.1	0.1	0.7	Existing-uPVC
P-36	J-31	J-35	119	100	56	-1.6	0.4	0.2	3.2	Existing-uPVC
P-37	J-131	J-130	223	75	56	2.0	4.2	0.5	18.6	Existing-uPVC
P-38	J-30	J-27	186	50	56	1.4	12.2	0.7	65.5	Existing-uPVC
P-39	J-131	J-34	216	50	56	1.2	11.2	0.6	51.8	Existing-uPVC

Pipe Name	Node Names		Length	Dia.	Roughness	Flow rate	Head loss	Velocity	HL/1000	Remark
P-4	J-1	J-4	40	300	56	31.5	0.1	0.5	3.6	Existing-uPVC
P-40	J-110	J-132	232	50	56	0.5	2.2	0.2	9.4	Existing-uPVC
P-41	J-35	J-33	267	100	56	-3.3	3.0	0.4	11.4	Existing-uPVC
P-42	J-35	J-36	333	75	56	-1.0	1.7	0.2	5.2	Existing-uPVC
P-43	J-37	J-30	87	75	56	0.6	0.2	0.1	1.9	Existing-uPVC
P-44	J-35	J-37	221	75	56	1.4	2.0	0.3	9.0	Existing-uPVC
P-45	J-130	J-38	205	50	56	0.9	6.4	0.5	31.3	Existing-uPVC
P-46	J-6	J-70	118	300	56	22.3	0.2	0.3	1.9	Existing-uPVC
P-47	J-33	J-39	338	300	56	17.7	0.4	0.3	1.2	Existing-uPVC
P-48	J-39	J-42	277	250	56	13.4	0.5	0.3	1.8	Existing-uPVC
P-49	J-39	J-36	138	100	56	2.4	0.9	0.3	6.5	Existing-uPVC
P-5	J-4	J-118	451	100	56	2.2	2.4	0.3	5.4	Existing-uPVC
P-50	J-36	J-41	180	100	56	1.1	0.3	0.1	1.5	Existing-uPVC
P-51	J-55	J-57	137	100	56	0.8	0.1	0.1	0.8	Existing-uPVC
P-52	J-42	J-43	58	250	63	16.4	0.1	0.3	2.1	Existing-Fe
P-53	J-103	J-44	476	200	56	16.0	3.5	0.5	7.3	Existing-uPVC
P-54	J-101	J-103	39	200	56	16.0	0.3	0.5	7.3	Existing-uPVC
P-55	J-101	J-46	332	150	103	19.8	4.8	1.1	14.5	New-uPVC
P-56	J-45	J-47	42	250	103	18.1	0.0	0.4	1.0	New-uPVC
P-57	J-46	J-44	247	100	56	-1.9	1.0	0.2	4.2	Existing-uPVC
P-58	J-46	J-51	136	75	56	-0.2	0.0	0.0	0.2	Existing-uPVC
P-59	J-49	J-50	162	50	56	0.2	0.3	0.1	1.7	Existing-uPVC
P-6	J-4	J-6	222	250	56	21.4	0.9	0.4	4.2	Existing-uPVC
P-60	J-51	J-49	193	75	56	-1.3	1.7	0.3	8.7	Existing-uPVC
P-61	J-51	J-52	466	100	56	2.9	4.2	0.4	9.0	Existing-uPVC
P-62	J-102	J-51	348	100	56	3.8	5.3	0.5	15.1	Existing-uPVC
P-63	J-47	J-53	177	100	56	0.6	0.1	0.1	0.5	Existing-uPVC
P-64	J-53	J-54	471	100	56	1.4	1.0	0.2	2.2	Existing-uPVC
P-65-CV	T-2	J-102	265	200	103	44.5	4.2	1.4	15.9	New-uPVC
P-66	J-43	J-80	384	150	56	2.6	0.4	0.2	1.0	Existing-uPVC
P-67	J-41	J-55	57	100	56	-1.3	0.1	0.2	2.1	Existing-uPVC
P-68	J-41	J-31	325	100	56	2.3	1.8	0.3	5.6	Existing-uPVC
P-69	J-134	J-120	318	400	96	174.9	2.5	1.4	7.8	New-Fe
P-7	J-5	J-3	293	100	75	-5.7	5.4	0.7	18.3	Existing-uPVC
P-70	J-130	J-58	103	75	56	1.1	0.6	0.3	6.0	Existing-uPVC
P-71	J-129	J-56	178	75	56	1.7	2.5	0.4	13.8	Existing-uPVC
P-72	J-58	J-129	214	75	56	-0.8	0.8	0.2	3.7	Existing-uPVC
P-73	J-58	J-59	113	50	56	1.1	5.3	0.6	47.0	Existing-uPVC
P-74	RV-1	J-108	288	300	56	59.0	3.3	0.8	11.4	Existing-uPVC
P-75	J-44	J-66	293	200	56	12.7	1.4	0.4	4.8	Existing-uPVC
P-76	J-62	J-107	500	100	103	-5.8	5.3	0.7	10.6	New-uPVC
P-77	J-100	T-2	295	200	103	19.2	1.0	0.6	3.4	New-uPVC
P-78	J-62	J-64	29	75	75	1.1	0.1	0.3	3.8	Existing-uPVC
P-79	J-64	J-63	149	75	75	0.5	0.1	0.1	0.8	Existing-uPVC
P-8	J-3	J-7	70	100	75	-3.5	0.5	0.4	7.4	Existing-uPVC
P-80	J-64	J-65	17	75	75	0.7	0.0	0.2	1.4	Existing-uPVC

Pipe Name	Node Names		Length	Dia.	Roughness	Flow rate	Head loss	Velocity	HL/1000	Remark
P-81	J-65	J-68	104	75	75	0.5	0.1	0.1	0.8	Existing-uPVC
P-82	J-65	J-67	65	75	75	0.2	0.0	0.0	0.1	Existing-uPVC
P-83	J-66	J-69	108	200	56	10.9	0.4	0.4	3.6	Existing-uPVC
P-84	J-134	J-135	157	150	103	10.4	0.7	0.6	4.4	New-uPVC
P-85	J-69	J-48	228	200	56	10.3	0.7	0.3	3.2	Existing-uPVC
P-86	J-70	J-33	106	300	56	22.0	0.2	0.3	1.8	Existing-uPVC
P-87	J-135	J-136	61	100	103	6.6	0.8	0.8	13.4	New-uPVC
P-88	J-6	J-71	127	200	56	2.2	0.0	0.1	0.2	Existing-uPVC
P-89	J-135	J-12	62	100	103	3.9	0.3	0.5	5.1	New-uPVC
P-9	J-3	J-8	76	100	75	6.7	1.9	0.9	24.6	Existing-uPVC
P-90	J-48	J-60	395	150	56	10.3	5.2	0.6	13.1	Existing-uPVC
P-91	J-60	RV-3	8	100	56	6.5	0.3	0.8	40.6	Existing-uPVC
P-92	J-61	J-60	354	50	56	-0.3	1.6	0.2	4.6	Existing-uPVC
P-93	J-43	J-75	31	250	56	13.8	0.1	0.3	1.9	Existing-uPVC
P-94	J-74	J-114	230	100	56	1.6	0.7	0.2	3.0	Existing-uPVC
P-95	J-74	J-54	233	150	56	1.7	0.1	0.1	0.5	Existing-uPVC
P-96	J-75	J-76	104	250	56	13.3	0.2	0.3	1.8	Existing-uPVC
P-97	J-75	J-111	285	75	56	0.5	0.4	0.1	1.2	Existing-uPVC
P-98	J-76	J-74	462	250	56	12.9	0.8	0.3	1.6	Existing-uPVC
P-99	J-76	J-113	276	100	56	0.3	0.0	0.0	0.1	Existing-uPVC

Appendix -D: Extended Period Simulation

End node results: Peak Hour

Node Name	Node Title	External Demand	Hydraulic Grade (m)	Node Elevation (m)	Pressure Head (m)	Node Pressure (kpa)	Remark
J-1		0.00	2,488.7	2,465.0	-	232.7	
J-10		0.39	2,496.0	2,461.5	34.5	338.0	
J-100		0.00	2,477.4	2,449.0	28.4	278.3	
J-101		0.00	2,471.7	2,449.0	22.7	222.6	
J-102		0.00	2,472.2	2,452.0	20.2	197.9	
J-103		0.00	2,471.4	2,448.0	23.4	229.6	
J-104		0.69	2,487.1	2,448.0	39.1	383.0	
J-105		0.71	2,487.2	2,446.0	41.2	404.4	
J-106		0.00	2,436.9	2,395.0	41.9	410.9	
J-107		0.00	2,438.4	2,380.0	58.4	572.7	
J-108		0.00	2,480.8	2,478.0	2.8	27.6	No demand
J-109		1.90	2,488.7	2,478.0	10.7	105.2	
J-11		0.65	2,495.9	2,459.5	36.4	356.5	
J-110		0.00	2,478.0	2,424.0	54.0	529.3	
J-111		0.61	2,485.8	2,423.0	62.8	615.9	
J-112		0.90	2,468.5	2,416.0	52.5	515.0	
J-113		0.29	2,485.9	2,422.0	63.9	627.0	
J-114		1.61	2,484.5	2,423.0	61.5	603.2	
J-115		4.86	2,484.6	2,420.0	64.6	633.9	
J-116		7.83	2,421.3	2,380.0	41.3	405.2	
J-117		1.35	2,487.1	2,440.0	47.1	461.6	
J-118		0.00	2,486.2	2,450.0	36.2	354.7	
J-119		11.75	2,487.6	2,453.0	34.6	339.1	
J-12		0.92	2,497.4	2,460.0	37.4	366.4	
J-120		0.00	2,495.9	2,461.0	34.9	342.3	
J-121		5.02	2,460.2	2,392.0	68.2	668.3	
J-122		4.49	2,460.2	2,410.0	50.2	491.8	
J-123		0.00	2,491.6	2,472.0	19.6	192.1	
J-124		0.00	2,488.8	2,442.0	46.8	458.5	
J-125		0.00	2,485.8	2,420.0	65.8	645.5	
J-126		2.51	2,432.2	2,380.0	52.2	511.6	
J-127		6.83	2,428.0	2,410.0	18.0	176.7	
J-129		0.00	2,470.0	2,422.5	47.5	465.6	
J-13		0.96	2,496.6	2,455.5	41.1	403.4	
J-130		0.00	2,469.8	2,420.0	49.8	488.3	
J-131		0.00	2,473.9	2,427.5	46.4	455.4	
J-132		0.00	2,475.8	2,427.5	48.3	473.5	
J-133		0.00	2,484.7	2,422.0	62.7	614.5	
J-134		0.00	2,498.4	2,462.5	35.9	351.8	
J-135		0.00	2,497.7	2,459.0	38.7	379.3	
J-136		0.00	2,496.9	2,456.0	40.9	400.7	
J-137		0.00	2,495.4	2,459.0	36.4	357.3	
J-139		0.00	2,464.9	2,401.0	63.9	626.1	
J-14		0.47	2,495.0	2,451.5	43.5	426.2	
J-15		2.10	2,493.6	2,434.0	59.6	584.5	
J-16		1.24	2,496.1	2,445.0	51.1	501.2	
J-17		1.06	2,494.0	2,443.5	50.5	495.1	
J-18		1.82	2,491.9	2,432.0	59.9	587.0	

Node	Node Title	External	Hydraulic	Node	Pressure	Node	Remark
J-19		0.92	2,493.3	2,459.8	33.5	328.6	
J-2		7.30	2,490.0	2,471.0	19.0	186.6	
J-20		1.59	2,482.9	2,460.0	22.9	224.4	
J-21		4.90	2,489.4	2,445.0	44.4	435.8	
J-22		2.22	2,481.7	2,441.0	40.7	398.8	
J-23		12.16	2,482.1	2,454.0	28.1	275.8	
J-24		4.79	2,483.5	2,442.0	41.5	406.5	
J-25		2.28	2,480.5	2,427.5	53.0	519.7	
J-26		2.33	2,470.4	2,409.0	61.4	602.4	
J-27		1.37	2,469.9	2,427.5	42.4	415.4	
J-28		5.92	2,487.3	2,434.0	53.3	522.5	
J-29		2.12	2,476.2	2,419.5	56.7	555.9	
J-3		0.53	2,495.4	2,459.0	36.4	356.7	
J-30		1.04	2,482.0	2,442.0	40.0	392.6	
J-31		0.82	2,483.8	2,441.0	42.8	420.0	
J-32		0.71	2,483.7	2,420.0	63.7	624.7	
J-33		0.98	2,487.2	2,464.0	23.2	227.9	
J-34		1.20	2,462.8	2,408.0	54.8	537.0	
J-35		1.29	2,484.2	2,450.0	34.2	335.4	
J-36		0.31	2,485.9	2,451.5	34.4	337.6	
J-37		0.43	2,482.2	2,446.0	36.2	355.0	
J-38		0.92	2,463.4	2,407.0	56.4	552.8	
J-39		1.90	2,486.8	2,457.0	29.8	292.4	
J-4		3.16	2,488.6	2,463.5	25.1	246.0	
J-40		0.06	2,486.3	2,448.0	38.3	375.7	
J-41		0.18	2,485.7	2,438.0	47.7	467.3	
J-42		0.35	2,486.3	2,448.0	38.3	375.9	
J-43		0.00	2,486.2	2,450.0	36.2	355.1	
J-44		1.39	2,467.9	2,446.0	21.9	215.0	
J-45		0.08	2,486.3	2,449.0	37.3	365.5	
J-46		1.35	2,466.9	2,426.0	40.9	401.1	
J-47		0.18	2,486.2	2,452.0	34.2	335.7	
J-48		0.00	2,465.4	2,422.0	43.4	425.7	
J-49		1.16	2,468.6	2,410.0	58.6	574.7	
J-5		3.26	2,490.0	2,458.0	32.0	313.8	
J-50		2.26	2,468.3	2,414.0	54.3	532.7	
J-51		2.12	2,466.9	2,416.0	50.9	499.3	
J-52		2.90	2,462.7	2,396.0	66.7	654.3	
J-53		1.55	2,486.1	2,464.0	22.1	217.2	
J-54		3.00	2,485.1	2,466.0	19.1	187.4	
J-55		0.06	2,485.8	2,434.0	51.8	507.7	
J-56		1.71	2,467.5	2,404.0	63.5	622.9	
J-57		0.80	2,485.7	2,422.5	63.2	619.4	
J-58		0.80	2,469.2	2,421.0	48.2	472.4	
J-59		1.14	2,463.9	2,410.0	53.9	528.2	
J-6		0.39	2,487.7	2,458.0	29.7	290.8	
J-60		3.45	2,460.2	2,402.0	58.2	571.0	
J-61		1.67	2,458.6	2,400.0	58.6	574.7	
J-62		4.63	2,433.1	2,367.0	66.1	648.3	
J-63		0.49	2,432.9	2,363.5	69.4	680.3	
J-64		0.00	2,433.0	2,368.0	65.0	637.4	
J-65		0.00	2,433.0	2,367.0	66.0	646.9	
J-66		1.86	2,466.5	2,438.0	28.5	279.7	
J-67		0.16	2,433.0	2,365.5	67.5	661.6	
J-68		0.49	2,432.9	2,367.5	65.4	641.2	

Node	Node Title	External	Hydraulic	Node	Pressure	Node	Remark
J-69		0.57	2,466.1	2,433.0	33.1	325.0	
J-7		0.92	2,495.9	2,461.0	34.9	342.1	
J-70		0.00	2,487.4	2,460.0	27.4	269.0	
J-71		0.84	2,487.6	2,450.0	37.6	369.0	
J-72		2.08	2,466.7	2,423.0	43.7	428.6	
J-73		2.77	2,429.4	2,418.0	11.4	111.8	
J-74		0.00	2,485.2	2,443.0	42.2	414.0	
J-75		0.00	2,486.2	2,451.0	35.2	344.7	
J-76		0.14	2,486.0	2,452.0	34.0	333.1	
J-77		0.92	2,467.9	2,414.0	53.9	528.7	
J-78		1.04	2,467.6	2,417.0	50.6	495.8	
J-79		1.12	2,466.2	2,405.0	61.2	600.5	
J-8		0.84	2,493.5	2,457.5	36.0	353.1	
J-80		0.27	2,485.8	2,434.0	51.8	508.2	
J-81		0.00	2,485.8	2,422.0	63.8	625.9	
J-82		1.73	2,463.9	2,405.0	58.9	577.6	
J-83		1.04	2,464.9	2,401.0	63.9	626.4	
J-84		1.69	2,464.5	2,397.0	67.5	661.7	
J-85		1.71	2,462.5	2,408.0	54.5	534.4	
J-86		3.84	2,465.3	2,417.0	48.3	473.4	
J-87		0.84	2,463.3	2,396.0	67.3	660.4	
J-88		4.59	2,441.1	2,400.0	41.1	403.0	
J-89		1.35	2,442.9	2,374.0	68.9	675.6	
J-9		0.69	2,495.8	2,462.0	33.8	331.9	
J-90		7.77	2,444.4	2,374.0	70.4	690.4	
J-91		2.53	2,442.5	2,391.0	51.5	505.0	
J-92		8.55	2,483.7	2,478.0	5.7	56.3	
J-93		6.32	2,484.4	2,462.0	22.4	219.4	
J-94		4.88	2,470.3	2,431.0	39.3	385.5	
J-95		5.00	2,486.1	2,446.0	40.1	393.0	
J-96		0.00	2,490.2	2,471.0	19.2	188.3	
J-97		0.00	2,489.7	2,473.0	16.7	163.6	
J-98		6.12	2,447.4	2,398.0	49.4	484.7	
J-99		0.00	2,510.1	2,460.0	50.1	490.8	
Pump-1		----	2,495.5	2,374.0	121.5	1,191.6	
RV-1		----	2,484.1	2,480.0	4.1	40.0	
RV-2		----	2,452.7	2,417.0	35.7	350.0	
RV-3		0.00	2,459.9	2,402.0	57.9	567.8	
RV-4		0.00	2,458.4	2,400.0	58.4	572.3	
RV-5		0.00	2,464.9	2,401.0	63.9	626.3	
T-1	R1 + R2 = 1000	----	2,489.1	2,480.0	9.1	89.1	
T-2		----	2,476.4	2,472.5	3.9	38.1	
T-3		----	2,503.4	2,499.0	4.4	43.3	
Pump-2		----	2,514.3	2,440.0	74.3	729.0	
RV-1		0.00	2,487.9	2,480.0	7.9	77.2	
RV-2		0.00	2,465.1	2,417.0	48.1	471.6	
RV-3		----	2,442.8	2,402.0	40.8	400.0	
RV-4		----	2,442.8	2,400.0	42.8	420.0	
RV-5		----	2,464.9	2,401.0	63.9	626.3	

APPENDIX-E: Simulation Results

End Node Results: Night Demand (Minimum Consumption)

Node Name	Node Title	External Demand (l/s)	Hydraulic Grade (m)	Node Elevation (m)	Pressure Head (m)	Node Pressure (m)
J-1		0.00	2500	2465	45	339
J-10		0.06	2500	2462	58	375
J-100		0.00	2477	2449	38	270
J-101		0.00	2475	2449	36	259
J-102		0.00	2475	2452	33	229
J-103		0.00	2475	2448	37	268
J-104		0.10	2500	2448	62	505
J-105		0.10	2500	2446	64	525
J-106		0.00	2443	2395	58	467
J-107		0.00	2443	2380	73	615
J-108		0.00	2484	2478	16	59
J-109		0.28	2500	2478	32	212
J-11		0.10	2500	2460	50	395
J-110		0.00	2484	2424	70	587
J-111		0.09	2499	2423	86	750
J-112		0.13	2484	2416	78	663
J-113		0.04	2500	2422	88	760
J-114		0.24	2499	2423	86	750
J-115		0.71	2499	2420	89	779
J-116		1.15	2442	2380	72	610
J-117		0.20	2500	2440	70	584
J-118		0.00	2500	2450	60	485
J-119		1.73	2500	2453	57	456
J-12		0.14	2500	2460	50	391
J-120		0.00	2500	2461	49	380
J-121		0.74	2470	2392	88	767
J-122		0.66	2470	2410	70	590
J-123		0.00	2500	2472	38	271
J-124		0.00	2500	2442	68	565
J-125		0.00	2499	2420	89	780
J-126		0.37	2443	2380	73	613
J-127		1.00	2442	2410	42	318
J-129		0.00	2484	2423	71	600
J-13		0.14	2500	2456	54	434
J-130		0.00	2484	2420	74	624
J-131		0.00	2484	2428	66	552
J-132		0.00	2484	2428	66	553
J-133		0.00	2499	2422	87	760
J-134		0.00	2500	2463	47	366
J-135		0.00	2500	2459	51	400
J-136		0.00	2500	2456	54	430
J-137		0.00	2500	2459	51	400
J-139		0.00	2470	2401	79	680

Node Name	Node Title	External Demand	Hydraulic Grade	Node Elevation	Pressure Head	Node Pressure
J-14		0.07	2500	2452	58	473
J-15		0.31	2500	2434	76	644
J-16		0.18	2500	2445	65	537
J-17		0.16	2500	2444	16	551
J-18		0.27	2500	2432	78	664
J-19		0.14	2500	2460	50	391
J-2		1.07	2500	2471	39	281
J-20		0.23	2499	2460	49	386
J-21		0.72	2500	2445	65	535
J-22		0.33	2499	2441	68	572
J-23		1.79	2499	2454	55	445
J-24		0.70	2499	2442	67	563
J-25		0.34	2499	2428	82	705
J-26		0.34	2484	2409	85	732
J-27		0.20	2499	2428	82	702
J-28		0.87	2500	2434	76	643
J-29		0.31	2484	2420	74	631
J-3		0.08	2500	2459	51	400
J-30		0.15	2499	2442	67	563
J-31		0.12	2499	2441	68	573
J-32		0.10	2499	2420	89	779
J-33		0.14	2500	2464	46	348
J-34		0.18	2483	2408	85	740
J-35		0.19	2499	2450	59	485
J-36		0.05	2500	2452	58	471
J-37		0.06	2499	2446	63	524
J-38		0.14	2483	2407	86	750
J-39		0.28	2500	2457	53	417
J-4		0.47	2500	2464	46	354
J-40		0.01	2500	2448	62	505
J-41		0.03	2499	2438	71	603
J-42		0.05	2500	2448	62	505
J-43		0.00	2500	2450	60	485
J-44		0.20	2475	2446	39	287
J-45		0.01	2500	2449	61	495
J-46		0.20	2475	2426	59	483
J-47		0.03	2500	2452	58	466
J-48		0.00	2475	2422	63	522
J-49		0.17	2475	2410	75	640
J-5		0.48	2500	2458	52	408
J-50		0.33	2475	2414	71	601
J-51		0.31	2475	2416	69	581
J-52		0.43	2475	2396	89	776
J-53		0.23	2500	2464	46	348
J-54		0.44	2499	2466	43	328
J-55		0.01	2499	2434	75	642
J-56		0.25	2484	2404	90	781
J-57		0.12	2499	2423	87	755
J-59		0.17	2484	2410	84	721

Node Name	Node Title	External Demand	Hydraulic Grade	Node Elevation	Pressure Head	Node Pressure
J-6		0.06	2500	2458	52	407
J-60		0.51	2475	2402	83	716
J-61		0.25	2475	2400	85	735
J-62		0.68	2443	2367	86	741
J-63		0.07	2443	2364	89	775
J-64		0.00	2443	2368	85	731
J-65		0.00	2443	2367	86	741
J-66		0.27	2475	2438	47	365
J-67		0.02	2443	2366	87	755
J-68		0.07	2443	2368	85	736
J-69		0.08	2475	2433	52	414
J-7		0.14	2500	2461	49	380
J-70		0.00	2500	2460	50	388
J-71		0.12	2500	2450	60	486
J-72		0.31	2484	2423	71	594
J-73		0.41	2442	2418	34	240
J-74		0.00	2499	2443	66	554
J-75		0.00	2500	2451	59	476
J-76		0.02	2500	2452	58	466
J-77		0.14	2484	2414	80	683
J-78		0.15	2484	2417	77	653
J-79		0.17	2484	2405	89	770
J-8		0.12	2500	2458	52	414
J-80		0.04	2499	2434	75	642
J-81		0.00	2499	2422	87	760
J-82		0.25	2484	2405	89	770
J-83		0.15	2484	2401	93	809
J-84		0.25	2470	2397	83	719
J-85		0.25	2470	2408	72	611
J-86		0.56	2484	2417	77	653
J-87		0.12	2470	2396	84	729
J-88		0.68	2452	2400	62	513
J-89		0.20	2452	2374	88	769
J-9		0.10	2500	2462	48	371
J-90		1.14	2452	2374	88	769
J-91		0.37	2452	2391	71	602
J-92		1.26	2499	2478	31	210
J-93		0.93	2499	2462	47	367
J-94		0.72	2499	2431	78	667
J-95		0.74	2500	2446	64	525
J-96		0.00	2500	2471	39	281
J-97		0.00	2500	2473	37	261
J-98		0.90	2453	2398	65	535
J-99		0.00	2506	2460	56	446
Pump-1		----	2495	2374	121	1186
RV-1		----	2484	2480	4	40
RV-2		----	2453	2417	36	350
RV-3		0.00	2475	2402	73	716
RV-4		0.00	2475	2400	75	735

Node Name	Node Title	External Demand	Hydraulic Grade	Node Elevation	Pressure Head	Node Pressure
RV-5		0.00	2484	2401	83	809
T-1		----	2500	2480	20	192
T-2		----	2476	2473	3	29
T-3		----	2500	2499	1	10
Pump-2		----	2509	2440	69	677
RV-1		0.00	2500	2480	20	192
RV-2		0.00	2484	2417	67	652
RV-3		----	2443	2402	41	400
RV-4		----	2443	2400	43	420
RV-5		----	2470	2401	69	680

Appendix -F

Calibration of the Pressure junction

Time steps	Pressure junction	O_i	$\bar{O}_i$	P_i	$\bar{P}_i$	$(O_i - \bar{O}_i)$	$(P_i - \bar{P}_i)$	$(O_i - \bar{O}_i)$	$(O_i - \bar{O}_i)^2$	$(P_i - \bar{P}_i)^2$
8:00-12:00 (earlymidnoon)	J-120	44.50	43	37.61	43	1.43	(5.39)	(7.71)	2.05	29.05
	J-46	23.05	43	26.11	43	(20.02)	(16.89)	338.13	400.78	285.27
	J-16	45.90	43	44.30	43	2.83	1.30	3.68	8.01	1.69
	J-131	36.65	43	40.80	43	(6.42)	(2.20)	14.12	41.21	4.84
	J-137	34.80	43	33.60	43	(8.27)	(9.40)	77.73	68.39	88.36
	J-116	32.60	43	35.30	43	(10.47)	(7.70)	80.62	109.61	59.29
	J-98	37.32	43	44.36	43	(5.75)	1.36	(7.82)	33.06	1.85
	J-114	31.64	43	34.65	43	(11.43)	(8.35)	95.44	130.63	69.72
2:00-6:00 (afternoon)	J-120	44.00	43	46.61	43	0.93	3.61	3.36	0.87	13.03
	J-46	36.80	43	39.11	43	(6.27)	(3.89)	24.39	39.31	15.13
	J-16	40.60	43	48.30	43	(2.47)	5.30	(13.09)	6.10	28.09
	J-131	48.70	43	52.80	43	5.63	9.80	55.18	31.70	96.04
	J-137	23.20	43	26.60	43	(19.87)	(16.40)	325.86	394.80	268.96
	J-116	46.30	43	44.30	43	3.23	1.30	4.20	10.44	1.69
	J-98	48.32	43	50.36	43	5.25	7.36	38.64	27.57	54.17
	J-114	43.64	43	46.65	43	0.57	3.65	2.08	0.33	13.32
8:00-12:00 (early midnight)	J-120	60.71	43	52.11	43	8.60	9.11	78.35	73.96	82.99
	J-46	58.70	43	57.61	43	15.63	14.61	228.36	244.31	213.45
	J-16	77.59	43	88.15	43	34.52	45.15	1558.60	1191.66	2038.52
	J-131	63.09	43	64.30	43	20.02	21.30	426.33	400.62	453.69
	J-137	70.12	43	71.10	43	27.05	28.10	760.12	731.73	789.61
	J-116	66.38	43	68.80	43	23.31	25.80	601.41	543.38	665.64
	J-98	59.11	43	64.86	43	16.04	21.86	350.64	257.30	477.86
	J-114	75.36	43	72.95	43	32.29	29.95	967.10	1042.67	897.00
2:00-4:00 (early morning)	J-120	24.34	43	25.61	43	(18.73)	(17.39)	325.76	350.91	302.41
	J-46	23.15	43	25.11	43	(19.92)	(17.89)	356.38	396.83	320.05
	J-16	37.67	43	18.30	43	(5.40)	(24.70)	133.37	29.16	610.09
	J-131	27.34	43	29.80	43	(15.73)	(13.20)	207.67	247.51	174.24
	J-137	32.80	43	28.60	43	(10.27)	(14.40)	147.88	105.46	207.36
	J-116	16.97	43	17.30	43	(26.10)	(25.70)	670.81	681.29	660.49
	J-98	32.25	43	23.36	43	(10.82)	(19.64)	212.50	117.06	385.73
	J-114	34.64	43	23.65	43	(8.43)	(19.35)	163.11	71.06	374.42

$$R^2 = \frac{\sum (x-\bar{x})(y-\bar{y})}{\sqrt{\sum (x-\bar{x})^2 (y-\bar{y})^2}}$$

R^2=0.95

Appendix –G

Summary of water demand

Table:- Population and Domestic Demand Projection

	Unit	2003	2005	2010	2015	2020
1. Population						
Population growth rate (town)		4.4%	4.4%	4.5%	4.3%	4.1%
Population (town)	No	75,480	75,480	75,480	75,480	75,480
Coverage (town)	%	69.0%	69.0%	77.7%	86.3%	95.0%
Total population	No	52,086	56,877	80,172	110,491	149,242
Coverage by service type						
HTU	%	3.0%	4.2%	7.2%	10.2%	12.0%
YTU	%	19.0%	23.1%	33.5%	43.8%	50.0%
NTU	%	8.0%	8.9%	11.3%	13.6%	15.0%
PTU	%	39.0%	36.2%	29.2%	22.2%	18.0%
Person per connection		4.9	4.9	4.9	4.9	4.9
Persons per PF		1,051	1,148	1,054	980	735
No Connections						
HT		462	505	1,264	2,351	3,847
YT		2,928	3,197	6,180	10,361	16,030
PT (urban)		28	28	31	35	38
Population served by						
HT		2,264	2,473	6,193	11,518	18,852
YT		14,346	15,666	30,282	50,768	78,549
NT		6,038	6,594	10,666	16,211	23,565
PT (urban)		29,437	32,145	33,030	31,995	28,277
Total Population Sevred		52,085	56,878	80,171	110,492	149,243
2. Demand						
2.1 Domestic						
Per capita demand						
HTU	l/c/d	40	40	53	67	80
YTU	l/c/d	30	30	35	39	44
NTU	l/c/d	20	20	24	28	32
PTU (town)	l/c/d	15	15	18	20	23
PTU (rural)	l/c/d	5	5	6	6	7
Adjustment factor		1.0	1.0	1.0	1.0	1.0
Consumption (daily)						
HTU	m3/d	91	99	330	768	1,508
YTU	m3/d	430	470	1,050	1,997	3,456
NTU	m3/d	121	132	256	454	754
PTU (town)	m3/d	442	482	584	651	650
Total Domestic demand	**m3/d**	**1,083**	**1,183**	**2,220**	**3,869**	**6,369**