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Assessment of Water Supply Coverage and Losses in The distribution system

(In Case Of Tilili 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 (Major in Water Supply and Environmental Engineering).

By: Simeneh Minwoyelet

Advisor

Dr. Agizew Nigussie

Addis Ababa

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This is to certify that the thesis prepared by Simeneh Minwuyelet, entitled: Assessment of Water Supply Coverage and Losses (In Case Of Tilili Town) and Submitted in partial Fulfillment of the Requirement for the Degree of Master Science in Civil Engineering (Major in Water Supply and Environmental Engineering) complies with the regulations of the University and meets the accepted standards with respects to originality and quality.

Signed by Examining Committee:

External Examiner Dr Geremeu Sehalu Signature ------------ Date ----------------

Internal Examiner Dr Mebruk Muhamed Signature ------------ Date ----------------

Advisor Dr Agizewu Niguse Signature ------------ Date ----------------

Dedication

The thesis is dedicated to my late father Minwuyelet Terefe and my mother Amahush Sinshaw.

Abstract

The main objective of this research is to assess the town water supply coverage and loss situation in the distribution system of Tilil town and to recommended measurements for improvement. Assessing the current water supply coverage by using percentage of mode of service and compare with different guidelines put for what could be the coverage in percent today and estimating the total water loss of the town then have made analysis by using different performance indicators and identify the cause of loss

The methodology was carried out by collecting the primary and secondary data from the TWSB and other focused areas. The water supply coverage was evaluated by using the total consumption data and aggregated with the total population of the town and compare how much water was produced and distribute, how much water was billed or consumed and how much water was loosed for the town.

After evaluated the water supply coverage and the total water loss of the town population for casting and break down of demand have been made and the data was assembled in Water CAD and simulated was takes place in the skeletonized network. Pressure measurement and water meter test was carried out in the town based on the following criteria. Pipe and meter age, elevation difference, relatively high medium and low water consumption, Hydraulically easily desecrate area for 24hr water availability

Based on the methodology the current water supply coverage of the town is very low relative to the recommended value and the value of NRW is higher too much compare with the coverage. There are several reasons for the low coverage and high level of water loss in the town, and some advisory solutions were briefly proposed for the major effect of the water loss and coverage

The main reason for the decline of average consumption per connection is the increasing of connections was not supported with proportional increase in supply and another factor increasing NRW are all the aged water supply infrastructure component was do not replaced before burst or leakage is exist, customer Meter Servicing Problem, lack of real time field information, inadequate training of personnel. To increasing water supply coverage or average consumption per connection is that the increase of connections should be supported with proportional increase in supply. Operation and Maintenance of network is very essential to increase the life of network, therefore, operation and maintenance policy needs to be develop

Keywords: Water coverage, NRW,ILI Real losses, apparent losses, meter accuracy and Performance indicators.

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.I would like to thank the Tilili Town Water Service bureau and Amhara Design and Supervision Bureau staffs all for their kind responses and facilitation during data collection and assisting me in providing the necessary data and information for the study.

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Table of Contents

List of figure

List of Table

List of Abbreviations

AAWSA- Addis Ababa Water and Sewerage Authority

ADD -Average Day Demand

ADSB- Amhara Design and Super Vision Bureau

AWWA- American Water Work Association

BH- Bore Hole

CARL- Current Annual Real Los

CSA- Central Statistics Agency of Ethiopia

DMA- District Metering Area

DCI- Ducktail Cast Iron

EPA- Environmental Protection Authority

ETB- Ethiopian Birr

HGL- High Gradient Level

IBNET-International Benchmarking Network

ILI- Infrastructure Leakage Index

IWA- International Water Associations

FGD- Focused Group Discussion

HTU- House Tap User

KII -Key Informant Interview

MDF- Maximum Day Factor

MDD- Maximum Day Demand

NC- Number of Connection

NFPA- National Fire Protection Association

NRW- Non Revenue Water

NTU- Neighbor Tap User

PTU- Public Tap User

PHD- Peak Hour DEMAN

PHF- Peak Hour Factor

SIV-System input volume

TTWSB- Tilili Town Water Service Bureau

TDD Total Domestic Demand

UARL- Un Avoidable Real Los

UFW- Unaccounted for Water

USEPA- United State Environmental Protection Agency

WDS- Water Distribution System

WSDS- Water Supply Distribution System

WHO- World Health Organization.

YTU- Yard Tap User

ANNEX

1.0 Background.

1.1Introduction

Water is the precious gift of nature and that it is the source of prosperity, life and wealth for the people. It is most crucial for sustaining life; and is required for all human activities. The available water sources throughout the world are getting depleted; and this problem is further aggravated by climatic change and the rate at which populations are increasing especially in developing countries. Urban settlements in the developing countries are, at present, growing five times as fast as those in the developed countries. Based on the Global water supply and sanitation assessment (2000)report in developing countries urban population is expected to grow from 1.9 billion in 2000 to 3.9 billion in 2030, averaging 2.3% per year. On the other hand, in developed countries, the urban population is expected to increase, from 0.9 billion in 2000 to 1 billion in 2030 overall growth rate 1%. This has brought into focus the urgent need for planned action to manage water resources effectively. Therefore the development, conservation and use of water, one of the main elements in country‘s overall development.

Intermittent supply leads to many problems including, severe supply pressure losses and great inequities in the distribution of water. Another serious problem arising from intermittent supplies, which is generally ignored, is the associated high levels of contamination. This occurs in networks where there are prolonged periods of interruption of supply due to negligible or zero pressures in the system (Vairavamoorthy et al., 2007a).

Water loss represents inefficiency in water delivery and measurement operations in transmission and distribution networks.

In pursuing a continuous water supply, cities in the developing world must ensure that their water systems become more efficient and effective by reducing water losses, gradually increasing water tariffs, improving revenue collection, increasing staff productivity, and securing safe and reliable water supplies. When efficiency gains are ensured, investments in new infrastructure will lead to more effective and efficient water services. The most municipalities need to manage leakage in their pipe networks. During this century, water will be a scarce resource and therefore, needs to be harnessed in a scientific and efficient manner.

Provision of reliable and safe water supply services to urban habitat is an essential contribution to overall economic and welfare advancement. Improving of adequate and potable water supply and reduction of loss in urban areas in both developed and developing Countries is essential any types of human moment. For instance, in developing countries the provision of adequate potable water supply in addition to drinking, cleaning etc. improves health by reducing incidence of water-related illness such as diarrhea, cholera etc. Reducing the incidence of illness will help to reduce demand for imported medicine and thereby easing balance of payment problem facing least developed countries.

According to WHO and UNICEF Joint Monitoring Programmed for Water Supply and Sanitation (JMP,2010)report Water lose is not only represent economic loss and wastage of a precious scarce resource but also pose public health risks. Every leak is a potential intrusion point for contaminants in case of a drop in network pressures. Leakage also often leads to service interruption and customer complaints, is costly in terms of energy losses and increases the carbon footprint of the service provider. These problems are likely to be compounded in the future as a result of the widening gap between ageing water supply infrastructure and investment, rapid population growth, poor management practices, poor governance, and more extreme events as a consequence of climate change. The high water losses in water distribution systems present an excellent opportunity of un-tapped water resources that have already been treated to drinking water standards and could be recovered cost effectively. To recover water losses requires understanding why, where and how much water is lost, and developing appropriate intervention measures. Water supply systems 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 to the problem. The Water Loss Task Force (WLTF) of the International Water Association (IWA) (2006) report moreover, managing and reducing losses of water at all levels of a distribution system remains one of the major challenges facing many water utilities in most developing countries including Ethiopia. As a result of the overall shortage of water many water utilities are faced a challenge in distributing the available water impartially among the residents. Beside to this poor management of the existing infrastructural asset increases the level of water losses in water supply.

The rapid increase in population, economic development and awareness of health benefits of improved water and sanitation have been proven by WWDSE (2010) to cause rise in water demand, necessity of improved system infrastructure management and strategies to deliver clean and safe drinking water to customers. Even though distributing the available water and

water loss from a utility‘s distribution system is a growing management problem in Ethiopia, there are few studies conducted on the existing water utilities in the country related to water loss and coverage. Although the Tilili Town water utility distribution system components were built decades ago and are currently in need of attention, issues related to the overall coverage of water supply and water loss from the utility are not investigated yet. Therefore, assessing the water supply coverage and water loss using statistical and water audit methods in order to develop strategies for the future is more urgent than ever. For that reason this study mainly deals with water supply coverage and loss assessment and developing strategies for the water loss reduction in Tilili town water utility.

Due attention has been given by the Ethiopian government to alleviate these shortcomings and to achieve rapid socioeconomic development through better health care and productivity of its people.

The sustainability of water supply facilities depends on a timely and regular maintenance and operation of the system. In Ethiopia, it has been found out that operation and maintenance (O&M) of water supply facilities are poor. There is poor technical and financial capacity among the urban service providers that leads to high levels of water losses mainly through leakages (MoWR, 2004) Ministry of Water Resources.

The Ethiopian water resources management policy (framed in 1999 with Proclamation No. 197/2000) focused on conserving, protecting, enhancing water resources and increasing the coverage, quantity, reliability, efficiency and acceptable quality, taking the existing and future realities of the country into consideration. The policy envisaged supplying improved potable water service based on full cost recovery and self-reliance‖ tariff, it provided incentive for proper use; reduces waste and excessive consumption of water resources.

1.2 Statement of the problem

One of the major problem affecting Tilili town water users is increasing the demand of the community and considerable difference between the amount of water put into the distribution system and the amount of water billed by consumers called “Non-Revenue Water” (NRW). Trends of Tilili town for different year and statistical surveys indicated that NRW in the town is increased on the other hand the water supply coverage of the town is below the recommended value.

Water shortage and frequent service interruption is not only as a consequence of the shortfall between demand and supply but also as result of lack of controlling NRW and managing network systems. Depending on the context of the existing system both or one of the factors may be found as a root cause for the shortfall between demand and supply.

The water supply coverage of Tilili town is very low compared with the minimum standard set by AAWSA – Business Plan Report(2011) (29l/c/d) and UN-Habitat at 2000 as a basic need for drinking and sanitation alone (25l/per/day). Nearly 51% of the entire town population is getting water less than this basic service level. Although there is overall shortage of water in the town, predominantly the existing amount of water is fairly distributed among the different customer.

The average water supply coverage of Tilil town is found to be 49% and the per-capita demand is estimated to be18.29liter/person/day and it is below the minimum required water set to urban town This average per capital consumption is lower compare with other developing town even it is lower than that of the standard set by UN-Habitat as a basic need. . The main reason for the decline of average consumption per connection is that the increase of connections was not supported with proportional increase in supply.

On the other hand despite the fact that overall pipe network seems to be of a young age, the total water loss is higher. This signifies that besides to the loss caused as a result of leakage, other non-physical losses may also be expected to be higher. To this effect, as illegal connection is not noticed as major problem, loss due to meter errors especially under recording of meters is expected to be higher. It has been observed that lack of attention to the important aspect of Operation and Maintenance (O&M) of water supply schemes in the towns often leads to deterioration of the useful life of the systems necessitating premature replacement of many system components .

The water, meter reading , data handling and billing errors and illegal use control policy has a big influence on the level of apparent loss. The frequency of sampling, testing, repair and replacement of customer meters will determine the level of metering inaccuracies.

Due to the above problem currently the water supply system in Tilili town does not meet the need of people and small industries. Tilili town also faced high levels of water losses.

People have to travel long distance and fetch unsafe and unreliable water from rivers, ponds and other unprotected source one of the main problem for insufficient NRW reduction efforts of the town is lack of understanding the magnitude, sources, and cost of NRW. Due to this the town faces a huge challenge to provide adequate and improved water supply system & to reduce NRW .All these in fact are manifested in deprived poverty status of the town

And it is becoming the adverse effect on urban development and public health and poor sanitation. In order to minimize these challenges, an enormous evaluation should be performed those are indicative of poor governance and poor physical condition of the WSDS.

1.3Objectives of the study

1.3.1 General objective:

The general objective of the research is to assess the performance of the existing water supply distribution system for Tilili Town and recommended measures for improvement

1.3.2 Specific Objectives:

The main objectives of the research are

To assess the current water supply coverage situation and compare with standards

To estimate the amount of water loss and its causes

To identify causes of water losses by using performance indicators

To suggest the appropriate measure for improvement

1.4 Research Questions

How much water is produced and distributed in to the community?

What is the current water supply coverage?

What is the level of water connection in the town?

What is the total demand and supply of the current condition?

What are the causes of water loss?

How much water is lost in the entire town while compared with the water produced with billed water to evaluate and compare the total loss?

How is the water availability, production, demand and consumption trend over a period of years?

What is the common practice of the town water service bureau with regard to water loss management?

What are the challenges and possible solutions for assessing town water coverage and loss?

1.5 Significance of the Study

The study indicate to provide relevant information for customer, government and non – government organizations which helps them for taking appropriate decision making and designing appropriate intervention to minimize shortage of information about the water supply of town.`

1.6The Study Area

1.6.1 Location

Tilili is located in the area bounded between the geographic coordinates UTM (Adindan) 37p 278503 East 1203386North. It lies mainly within Fetam River catchment at an average elevation of 2440 m,a.s.l. The topography of the town area is flat to gently rising mild slopes towards North.

1.6.2 Topography

It is situated at 2440 m,a.s.l meters above sea level. The town has 1380 ml average annual rainfall and a minimum and maximum temperature of 15oC and 22oC. Tilili town is one of the high land areas of the country

1.6.3 Population

According to the 2015 (CSA) data, the projected population of Tilili Town for the year 2015 E.C was estimated to be about 17518. . According to the municipality's report, 97% of them are from the Amhara, and the rest are from Oromo, Tigre and other nation‘s nationalities. Most of the town's residents are Orthodox Christians. The rest are Muslims and Protestants. According to a research by conducting Tilili town will be in shortage of water supply unless new infrastructure will be constructed or the current water management efficiency improved.

Tilili is one of the rapidly urbanizing towns of the Amara, region suffering from the shortage of water and high water loss. The existing situation of water supply system in Tilili town not meets the demand. And it is becoming the adverse effect on urban development and public health and poor sanitation. In contrary with the high coverage of the water supply system, adequate amount of water is not supplied to the community and this is supported by the variations in water supply..

To address the water supply coverage and losses problems and to give appropriate solutions, it requires relevant data and information as well as different research which could be used as an evaluation of water supply coverage for taking appropriate decision making.

2.0LITRATURE REVIEW

2.1Introduction

Cities and Town are increasingly facing acute water crisis in terms of imbalance between supply and demand. The demand for urban water supply and allied services is increasing rapidly as globalization accelerates economic development and brings about improvements in living standards in the country due to the dynamics of demography, i.e., the interactive effects of demo- graphic growth and migration to town under its push and pull effect. 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 systems 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 to the problem. The developing cities and town have great difficulty both financial and technical to develop and expand water supply projects and one of the difficulties among the others is managing and reducing loses of water at all levels of a distribution system. As a result of the overall shortage of water many town are faced a problem in distributing the available water impartially among the residents. Beside to this poor management of the existing infra structural as set 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 losses will be reviewed in this chapter.

A recent study made by the African Development Bank (ADB, 2006) concluded that the average level of water loss is 30% of the water produced, with wide variations among individual suppliers ranging from 4% to 65%. The World Bank (WB, 2006) study also found that more than 900 utilities in 44 developing countries around the world have approximately 35% in water loss level. The actual figure for overall water loss levels in the developing countries is more in the range of 40-50% of the water produced. As a result of increased concern about water losses, a variety of technical guidelines and technical reports describe the procedures and methodologies for detecting and repairing leaks. Guidelines for analyzing the feasibility of technology renovation have also been formulated. The American Water

2.2 Urban Water Coverage and Loss

2.2.1 Urban Water Coverage

According to the researcher there are different definitions towards of water supply coverage in the world. According to (International Program of UNICEF and WHO 2000) water supply coverage is defined as the population of people who have access to protected water supply. According to this definition in 2007 the water supply coverage was 98.6%.

According to coverage calculations made by AAWSA(2011) are usually based on the individual per-capita demand from the total production of water this done by the following approach.

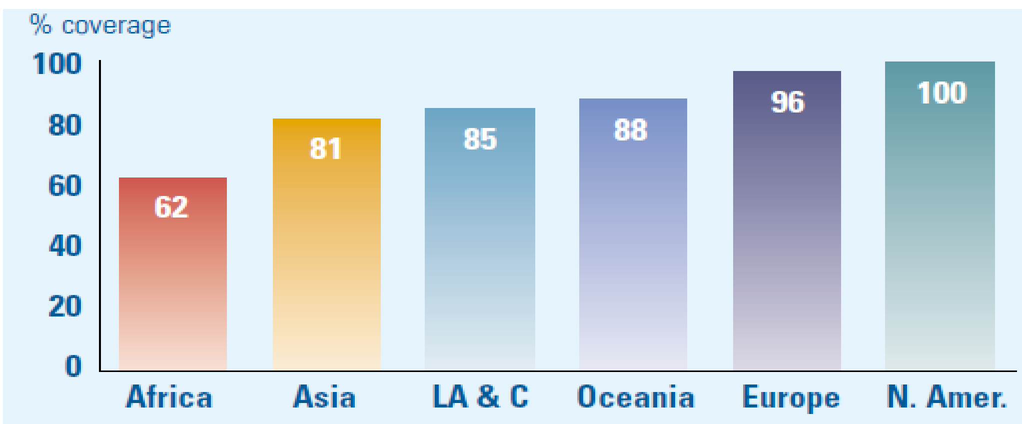
The mean per capita domestic demand of the town is estimated to be 66 l/c/day at the beginning of the planning horizon (2011) and will reach 110l/c/day by the end of the planning period (2020) exhibiting 6.6% annual increment. The proportion of domestic demand will increase from 63% to 74% during the planning horizon exhibiting a shift of 11%.

The coverage is calculated through the formula: COV % = Total production x (1 - % of physical losses) / (Population x 110 l/c/d).According to this definition the water supply coverage in 2010 is 81%. Such an approach is rather appropriate to analyze the supply capacity of the Utility and not the actual water used by the residents.

Another calculation, more indicative for distribution issues, and considering only one mode of service (connection), is to compare the number of domestic connections with the total number of households.

According to this definition the connection rate in 2009 was 45%. The limitation of this approach is that it does not consider the demand level of those households who would like to use yard connections by sharing from the houses in the same compound. Housing Units- Another factor that complicates the coverage rate is the definition of housing units. AAWSA defines housing unit based on compound. It does not consider households that do rent part or the whole of a service quarter not as housing units but as part of the main house. However, CSA considers the units with in the service quarter and other premises in the compound as individual housing units. For projection of housing units CSA's housing unit definition has been considered.

Water supply coverage is a key tools used to evaluate the existing water supply condition of one specific country or town and helps to compare one country with others globally to locally and used to compare the degree of water loss from town to town and town to cities in the distribution system of one specific town. The percentages of population with or without piped water connection are a relevant indicator to compare the coverage and level 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 and town of developing countries in general and African cities in particular is very low while compared to the demand (WHO, 2000).



Source: Global Water Supply and Sanitation Assessment 2000 Report
Figure -2.1 Urban water supply coverage

2.2.2Water loss

Water loss is defined as the quantity of water different between supply and consumption through the distribution system wasted in different ways. The global volume of non-revenue water (NRW) or water losses is staggering. Each year more than 32 billion m3 of treated water are lost through leakage from distribution networks. An additional 16 billion m3 per year are delivered to customers but not invoiced because of theft, poor metering, or corruption. In some low-income countries this loss represents 50-60% of water supplied. The World Bank database on water utility performance (IBNET), the International Benchmarking

Network for Water and Sanitation Utilities, at (www.ib-net.org) includes data from more than 900 utilities in 44 developing countries.

The world‘s water resources are rapidly deteriorating due to the combined effects of Climate change, population growth and fast development. Hence, Water loss management is becoming even more vital. It has long been recognized that fresh water supplies are a finite resource that require careful and sound proactive strategic management (water conservation) to ensure that adequate supplies are available to meet the demands (Miya, 2008).

Not understanding the magnitude, sources, and cost of NRW is one of the main reasons for insufficient NRW reduction efforts around the world. Only by quantifying NRW and its components, calculating appropriate performance indicators, and turning volumes of lost water into monetary values can the NRW situation be properly understood and the required action taken. It is noteworthy that despite the fact that many utilities in the developing world have implemented NRW reduction programs.

2.2.3Water Losses In Developed and Developing Countries

2.2.3.1Water losses in some developed countries

The magnitude of Water loss is highly different between developed and developing country for instance in the Netherlands, low loss levels in the range 3-7% of distribution input have been reported (Beuken et al. 2006). The water distribution system in the Netherlands is perhaps the most efficient in the world. In the USA about 22 million m^3 of water is lost per day or characterized as public use/loss. The usual NRW in the USA is 15% but range from 7.5% to 20% (Beecher 2002). In the USA, losses due to main breaks are on the same order of magnitude as annual flood losses, which are estimated at more than $2 billion of property damage (Grigg 2007). In the UK, often perceived to be leaders in leakage management, about 3 million m3 per day is lost as leakage and has remained relatively stable at about 20-23% of water delivered in the past decade (OFWAT 2010). Most companies in the UK are operating at economic levels of leakage based on current tools, techniques and technologies. In Portugal, NRW averages 34.9% but varies from less than 20% to more than 50% (Marques and Monteiro 2003). In Greece‘s Larissa city, NRW has been estimated at 6 million m3/year (or 34% of SIV) (Kanakoudis and Tsitsifli 2010). In Australia, for a data set of 10 water systems, NRW varies from 9.5 to 22%, with an average of 13.8% (Carpenter et al. 2003). In Canada‘s Ontario province, as much as $1 billion worth of drinking water disappears into the ground every year from leaky municipal water pipes, and leakage varies from 7% to 34% of water distribution input (Zechner 2007). The fact that water utilities and municipalities are losing such large amounts of water from WDSs undermines their efforts in promoting water conservation and efficient use of water with negative environmental, economic and social impacts.

2.2.3.2Water losses in some developing countries

A recent year to improve water distribution system efficiency via reduction of water loss, progress in the developing countries is painfully slow. In Asian cities, the Asian Development Bank reports water loss in the ranges of 4.4% of total water supply (PUB, Singapore) to 63.8% (Maynilad, Manila) (ADB 2010) and 50-65% of NRW is due to apparent losses (McIntosh 2003). In Africa NRW figures ranging from 5% (Saldanha Bay, South Africa) to 70% have been reported (WSP 2009). In Latin American water utilities NRW of 40-55% of water delivered have been reported (Corton and Berg 2007). In Brazil, water losses average 39.1% of water supplied, equivalent to almost 5 billion m3 of water lost every year (Cheung and Girol,2009).

Based on the World Bank study, around 45 million m3 of water is wasted daily by leakage, enough to serve nearly 200 million people and nearly 30 million m3 of water is reach every day to customers but not paid for due to the following reason metering inaccuracies, theft and corrupt utility employees, costing water utilities about US $6 billion every year (Kingdom et al. 2006). Clearly, this is unacceptable, that where water utilities are starving for additional revenue to expand services to the poor and where water is heavily rationed that it is also highly loosed.

This is likely to be compounded by the high rate of infrastructure deterioration which will result in greater loss of treated and energized drinking water. The impact of poorly managed urban WDSs joined with increasing global change pressures (urbanization, climate change, population growth) is likely to result in extreme scarcity scenarios. In the Middle East and

2.2.4 Factors Influencing Losses

Real or Physical Losses are accelerated by many factors including soil conditions, quality of pipe materials, proximity to electrical currents, pressure regime , the managing staff and community behaviors etc. There are different crucial factors which can influence system performance and result in excessive real losses, including, continuity of supply, length of mains, number of service connections, location of customer meters on service connections, and average operating pressure. Since operating pressures are often constrained by local topography and the specified minimum standards of service (to customers or for fire-fighting) they can vary significantly between systems from 30 meters to over 120 meters and for this reason it is difficult to recommend an appropriate pressure that should be maintained. This said, however, from a leakage viewpoint it is fair to say that the minimum permissible pressure should be used in all cases for as much of the time as possible. Many countries already recognize that pressure control is one of the most important tools of proper leakage management. Pressure reduction not only reduces the water lost through existing leaks but also reduces the frequency with which new leaks occur. In addition, effective pressure management can greatly extend the life of the water reticulation system an important and often overlooked benefit.

2.2.4.1Intermittent supply system

The distribution system is usually designed as a continuous system based on the assumption of continuous supply.

However, in most Ethiopian town water supply is not continuous but intermittent. A severe problem arising from intermittent supplies, which is generally ignored, is the associated high levels of contamination. This occurs in networks where there are long periods of interruption of supply due to shortage or zero pressures in the system.

Intermittent supply creates uncertainties in the minds of the consumers about the reliability of water supply. During the supply period the water is stored in all parts of vessels for use in non-supply hours and when the supply is resumed, the stored water is pushed and drained and fresh water again stored. During non-supply hours polluted water may enter the supply mains through leaking joints and pollute the supplies and create the health related problem. Further, this practice prompts the consumers to always keep open the taps of both public stand posts and house connections leading to wastage of water whenever the supply is resumed. Intermittent systems which require frequent valve operations are likely to affect equitable distribution of water mostly due to operator negligence. 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, increased levels of leakage are experienced due to stress being inflicted on the pipes causing them to rupture. Due to high levels of water loss, a continuous supply is not available resulting in water schedules. The elimination of water schedules becomes the desired goal.

2.2.4.2Poor Infrastructure

In developing country, it has been observed that the distribution system is very aged which is laid many years ago. With age there is considerable reduction in carrying capacity of the pipelines due to corrosion. In most of the places the consumer pipes get corroded and leaks occur resulting in loss of water and reduced pressure. All these materials suffer from degradation over time due to operational measures, environmental conditions and general wear and tear and result in increased leakage in the network. It is, therefore, necessary to replace older mains so that less leakage occurs.

2.2.4.3Supply hours and system pressure

In intermittent water supply system most water supply agencies provide water to its consumers at restricted hours per day to 70% of the population in its area of responsibility. Shortage of water sources and frequently failure of power supply may cause reduced supply hours and hence reduction in quantity of water supplied. The pressure in distribution system is very poor varying from 0.3 to 1.0 kg/cm2. Most of the consumers have two storage tanks for storing the water, one below the ground level and one overhead tank due to less residual pressure.

2.2.4.4Poor record keeping

In developing country, it has been observed that pipe network is very old which is laid many years ago. The pipe materials becoming aged there is considerable reduction in carrying capacity of the pipelines and increasing NRW due to corrosion. In most of the places the consumer pipes get corroded and leaks occur resulting in loss of water and reduced pressure. All these materials suffer from degradation over time due to operational measures, environmental conditions and general wear and tear and result in increased leakage in the network. It is, therefore, necessary to replace older mains so that less leakage occurs.

Preventive maintenance of distribution system assures the twin objectives of preserving the bacteriological quality of water in the distribution system and providing conditions for adequate flow through the pipelines. Incidentally, this will prolong the effective life of the pipeline and restore its carrying capacity. Some of the main functions in the management of preventive maintenance of pipelines are assessment, detection and prevention of wastage of water from pipelines through leaks, maintaining the capacity of pipelines, cleaning of pipelines and relining.

2.2.4.5Community Behaviour

Water loss is not just an engineering or designing problem but also reflects a socio cultural condition that needs making awareness or changes in community behavior and attitudes toward water usage and its benefit. Many efficacies that have been successful in lecturing NRW have gone beyond technical measures to address community behavior that drives illegal connections and pilferage.

Addressing these problems, some Asian cities are also binding communities to reduce NRW. Technical measures have been complemented by efforts to address illegal connections by walk-through surveys and authorizing illegal connections by legitimizing them and adding them to the network.

2.2.5Types of Water Losses

2.2.5.1 Leakage in Water Distribution Systems

From one municipality to another and even from one location to another, the causes of leaks will vary depending on the nature of the soil , the quality of construction , the materials used, the pressure levels and the utilities operating and maintenance practice (AWWA, 1987) .

Leakage is often a large part of unaccounted for water (UFW) and is a result of either lack of maintenance or failure to renew ageing systems. Leakage may also because 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 leakage s 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 differentiate 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 w ater lost in a network, and comprises the physic al losses from pipes , joints and fittings and also from over flowing service reservoirs (WHO , 2001).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 etal .,2000) .

2.2.5.2 Pressure and leakage

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 ‘developed 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 below (wallingford HR.,2003) .owever, 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 second peaking is at evening.

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 variation s 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 building s 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 area s ; pressure above 90 meters , tend to result in excessive leakage losses (Twort A.c.et al .1994) ..

2.2.5.3 Effects of Corrosions on leakage

Corrosion is the main 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 oils must be protected by coating s with corrosive resistant materials.. Recent estimates indicate that the cost of water main breaks in Canada is about $ 8 0 million per year. One year on that this cost is so high is that most water mains in Canada are made from either cast or ductile iron. As thise pipes age, they are weakened by corrosion, causing an increased number of breaks (IRc1996) .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 such as clays and other highly organic soils can be extremely corrosive, though corrosive condition can exist in non-corrosive s oils too .Soil condition s are responsible for the exterior corrosion of metal structures under or in contact with the ground.(Laikeselassie,2004)

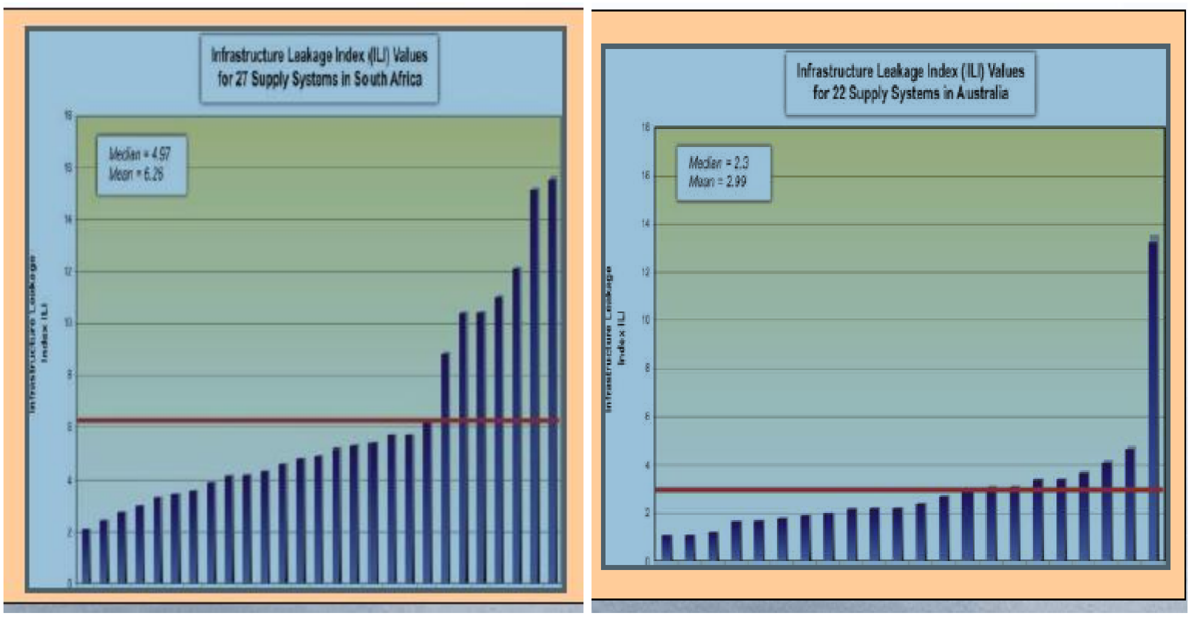
Source:2010@American water work association

Figfure2.2Effects of correctional leakage

2.2.5.4Infrastructure Leakage Index (ILI)

It is the ratio of the CARL to the value of UARL calculated for current pressures and continuity of supply. As this is a ratio and hence has no units and it can be said that this is a non-dimensional Performance Indicator of the current overall management of the infrastructure for leakage control purposes which can be used as a comparison between countries of different measurement. The greater the amount by which the ILI exceeds 1.0, the greater the potential opportunity for further management of real losses by infrastructure management and maintenance, more intensive active leakage control, or speed and quality of repairs. If the ILI for a particular system is calculated, and is, say, 3.0, this means that:

• The current annual Real Losses are assessed as being around three 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 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 is scarce, or both



Source: IWA Water Losses Task Force (Lambert et al, 1999)

Figur-2.3: Comparison of ILI Values Australia with South Africa

2.2.5.5Measuring water 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 an important indicator of probable losses, but it may over estimate them because supply meters tend to under-record consumption. 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, rate s of occupancy, age of mains , length of mains per 1000 population served, proportion of trade and bulk supplies , ground condition , etc . (Twort A .c. etal .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 Ac . 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) .

2.2.5. 6 Consequences of water loss and Leakage

The primary consequence of leaks in a distribution system is financial. Reduction in water loss enables water utilities to us e 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. 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.

2.2.6Apparentloss

Apparent Losses (AL) commonly known as commercial losses ,Water that is not physically lost but does not generate revenue because of inaccuracies metering inaccuracies, unauthorized use of water, meter reading errors, data handling and billing errors or any form of theft or illegal use. They occur as a result of inefficiencies in the measurement, recording, archiving, and operations used to pathway water volumes to a water utility (AWWA 2009). According to the World Bank, the volume of real losses occurring in developing countries alone is sufficient to supply approximately 200 million people. Water theft Complete illegal, by passes, disconnected reconnect) Free Water Supply (Stand posts, Bath taps, Toilet Taps, Fire demand, Bower Supply, Line Flashing) administration errors (Meter errors, Billing mistakes, estimated bills).

2.2.6.1 Meter error and water loss.

The purpose of a water meter is to measure and track the amount of water delivered through a distribution system. More importantly a water meter registers the amount of water delivered to a customer so an appropriate bill for that water can be charged. That charge determines the amount of revenue a system receives. The potential for revenue loss can be staggering if the system has a large number of meters significantly under-registering. Loss of revenue is more significant particularly for systems that have a high water production cost or high purchase water costs.

A water meter like any mechanical device is subject to wear and deterioration over time. The deterioration would be accelerated by poor water quality such as corrosive or abrasive water. Water meters may over register but this rarely occurs because wear on internal meter parts generally cause lower measurements. It can be assumed that after a certain age the inaccuracy of the meter due to deterioration becomes 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 meters that many of them put meter replacement policies to alleviate the problem. When should a water meter be replaced? There is no study that can show or recommend the exact age when a water meters accuracy is diminished to such a degree that replacing is economical.

How long water meters retain their overall accuracy depends on many factors, such as the quality of the water being passed through the meter, the rate of flow and the total quantity of water that has been measured. Various conditions that water meters are exposed to prevent any exact time frame for water meter decay due to differences in water chemical composition, temperature and humidity. Economic replacement policies for residential meters based on selective testing programs in the National Reports generally indicate that meters should be repaired or replaced after 5 or 15 years. At this age the accuracy would have diminished to the point that the cost of meter replacement is less than loss of revenues with continued use of the meter. The residential meters is majority for any system generally cost between $30 and $40 which translates to less than $3 a year for a 15 year service period? 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) .

Source:2010@American water work association

Figure2.4 meter registration error due to age.

2.2.6.2 Comparing water losses

The amount of water loss differs from country to country, city to city and even from network to another network within one city .Different countries use different indicators to evaluate their status in comparison with others and to compare the distribution of water loss from one location to other location of a distribution system in order to take action based on the level of loss. As stated above 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 comparison s (WHO,2001).

Depending upon the consumption per service connection, the same volume of real losses/ service 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 area s 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 Table 2-1 below .

Table 2-2 Standard International Water Balance

<table>
<tr><td rowspan="9">System Input Volume</td><td rowspan="4">Authorised Consumption</td><td rowspan="2">Billed Authorised Consumption</td><td>Billed Metered Consumption</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="7">Non Revenue Water</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water Losses</td><td rowspan="2">Apparent Losses</td><td>Unauthorised Consumption</td></tr>
<tr><td>Customer Meter Inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission & Distribution Mains</td></tr>
<tr><td>Leakage and Overflows at Reservoirs</td></tr>
<tr><td>Leakage on Service Connections up to metering point</td></tr>
</table>

(Source:Farley&Stuart)

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 others who are implicitly or explicitly 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 volume s and billed authorized consumption.

2.2.6.3Global overview of water loss management (WLM)

Urban water supply distribution systems are often buried and forgotten until when they visible into leaks and bursts causing significant economic, environmental and social costs. The efficiency of water supply distribution systems is measured by the difference between SIV and water delivered to customers and billed (revenue water) commonly referred to as non-revenue water (NRW) (Lambert and Hirner 2000). NRW is made up of water losses (RL and AL) and authorized unbilled consumption such as water for firefighting and flushing mains. The amount of water lost is a measure of the operational efficiency of a water supply distribution system (Wallace 1987). High levels of water losses are telling of poor governance and poor physical condition of the WDS (Male et al. 1985; McIntosh 2003) and the costs of system inadequacy are transferred to customers via high water tariffs (Park 2006). According to WHO/UNICEF (2010), 884 million people in the world do not have access to improved water supply, almost all of them in the developing regions. This challenge is likely to be exacerbated by the rapidly increasing urban population in that region. Half of the world‛s population 3 billion people live in urban areas and are projected to reach 5 billion by 2030 (Feyen et al. 2009). The stark reality is that the World‛s water resources are finite and limited and can no longer sustain this rate of growth unless used wisely. The high water losses in WDSs present un-tapped‖ water resources that can be recovered cost effectively.

2.2.6.4Real Loss Management

Tools and methods are required to reduce these water losses. Although a number of methodologies and tools have been developed, their application to water supply distribution system in most developing countries is generally still limited due to the unique conditions that exist such as intermittent water supply, limited resources and high levels of apparent losses. The study also reveals the gap between developed methods and their applications in practice. Future research needs identified include developing methods and tools for uncertainty in flow measurements and their propagation into NRW, apparent loss intervention measures, appropriate performance indicators and benchmarking techniques for water loss management in developing countries, planning and refining economic models for pressure management, online monitoring and optimal sensor placement for leaks/bursts detection, and exploring further discrete multi criteria analysis involving stakeholders in planning and management of water losses.

Source: Global Water Supply and Sanitation Assessment 2000(WHO-UNCEF)**Figure-2.5 Four Basic Methods of Managing Real Losses**

2.2.6.5Apparent loss management

The water, meter reading , data handling and billing errors and illegal use control policy has a big influence on the level of apparent loss. The frequency of sampling, testing, repair and replacement of customer meters will determine the level of metering inaccuracies.

Illegal consumption will never be known unless there is a proactive utility strategy to investigate suspicious accounts.

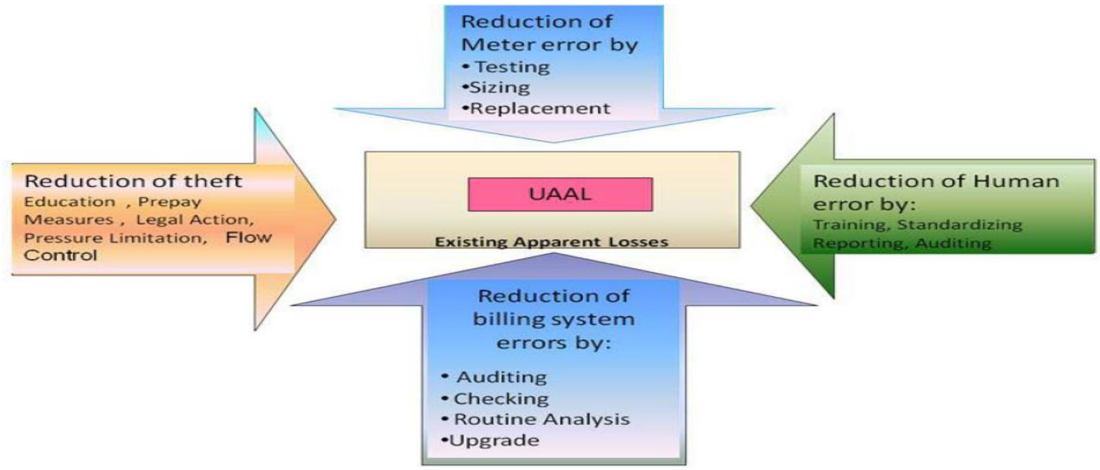
Socio-economic aspects: The great amount of illegal use cases in Colombian cities is contributed to poverty and unplanned settlements (slums)(Medeiros et al. 2010).To the contrary, in Africa city, the records indicate that 12.5% of confirmed illegal cases were commercial users (Mutikanga et al. 2011) who will always do anything to downfall the system including corrupting utility employees in order to minimize their operating costs. Poor people striving to survive on less than a dollar per day cannot afford to bribe utility employees for illegal connections or meter by passes. However, the high number of illegal use cases were found to be also attributed to low pay of utility field plumbers who actually carryout the illegal connections and reconnections to earn extra income.

Cultural aspects: The cultural honesty of people also plays an important role for the reduction of NRW. It is not only the poor who use water illegally but also the very rich star hotels trying to minimize operating costs. Some individuals have habits of stealing for the sake of it a habit known as kleptomania. Studies on electricity theft on102 countries do confirm these behavioral characters (Smith 2004). In some societies, people will always find ways of evading bills not only for water but for example tax. Some people still perceive water as a social good and not an economic good and will always find ways of using it illegally. Illegal use of water is a complex social-technical problem that needs social-cultural approaches.

Governance issues: According to McIntosh (2003), poor governance is the root cause of the NRW problem. Both water theft and electricity theft are identical, and it is likely that customers who steal water are likely to steal electricity as well. The only difference is that it is levels of electricity theft have been reported in countries with high levels of corruption, low government effectiveness, political instability and ineffective accountability (Smith 2004). The punishments charged in courts of law are often too low compared to the value of water stolen. In addition, compiling evidence for illegal use cases is not always trivial and can be expensive. This is particularly common in the developing countries where water service coverage is low, supply is irregular, and salaries are low.

2.2.6.6 Apparent loss reduction strategies

Many utilities prefer to address AL to earn quick revenues needed before controlling real losses and infrastructure rehabilitation. However, apparent loss plans go beyond conventional engineering approaches to managerial solutions that include social and behavioral sciences. Whereas, AL cannot be totally eradicated, the following interventions have proved to be very helpful in minimizing the losses. Revenue protection: Establishing revenue protection structures within the utility organizational mainstream have been reported to minimize illegal use in both the water and energy sectors (AWWA 2009; Mutikanga et al. 2009a; Smith 2004). The focus of the revenue protection unit (RPU) is to investigate and monitor suspected accounts more proactively (zero consumption, negative billing, low consumptions etc.).



Source: Global Water Supply and Sanitation Assessment 2000(WHO-UNCEF)

Figure2.6 -Four Basic Methods of Managing Apparent Losses.

2.2.6.7 Numerical Model Calibration

According to (Haestad Method 2003) first edition transient flow problems are usually solved using hydraulic computer model analysis. Given descriptions of the system and the event triggering an information on the boundary conditions, the model can determine fluid velocity V (and flow Q) and pressure p (and head H). Comparisons of computed results from the analysis program with results measured during laboratory experiments or field tests from actual systems have been used to validate the analysis programs. These comparisons require that well-defined flow control operations be used to allow proper simulation and calibration of the computer model. For systems having free gas in the liquid and the potential for liquid column separation, the theoretical analysis is more complex, and the computed results have more uncertainty. It is impossible to develop a theoretical model that accurately simulates every physical phenomenon that can occur in an actual hydraulic system. Therefore, all modeling of involves some approximation and simplification of the real problem.

The differences between computer model results and actual system measurements are caused by several factors, some of which are highlighted in the following list:

A calibrated model can also be used as part of an outflow‖ detection system. Model simulation results from a specific flow control operation can be compared to actual system results at particular nodes in the system. Significant differences between computed and measured values may indicate an outflow (such as an open valve, leak, or pipeline break) that can be rapidly located, evaluated, and corrected.

In general, if model peaks arrive at the wrong time, the wave speed needs to be adjusted. If model peaks have the wrong shape, the description of the control event (pump shutdown or valve closure) should be adjusted. If the transient dies off too quickly or slowly in the model, the friction losses need to be adjusted. If there are secondary peaks, important loops and diversions may need to be included in the model.

However model is a generalized, simplified representation of a complex system or task. In addition to this models used to understand a process and effectively make decisions. Models are essential in that they are used to:-develop insights and understand the governing principles and fundamental mechanisms of a system, determine the ‗what if?‘ questions and predict outcome of a system, assess impacts of a system. But it may be garbage in garbage out so it should be calibrate and validate with the standard and actual or observed data

.2.2.10 Conclusions

The literature review was focusing on the issues related to the water supply coverage and losses of water in a distribution system, causes of water losses, the consequence of water loss, methods of evaluating water loss, etc. The literatures were reviewed considering expected methods and approaches needed in the following analysis chapters. Although it was tried to assess relevant literatures, some additional inputs from literatures will be referred while discussing relevant issues in each analysis chapters.

In several developing countries on the one hand the level of water c overage is very low while compared to the developed world on the other hand water loss is comparatively very high and difficult to manage both technically and financially. Many countries use unaccounted for water expressed in percentage terms to compare the level of water losses among different countries or different cities. Nevertheless, some literatures do not recommend using it for compares on among different areas as it is highly affected by the magnitude of production. Moreover, evaluating water loss using a night flow monitoring that is usually expressed as per length of mains or number of service connections is a good indicator especially for evaluating the physical loss, but it is not an easy task especially in developing countries where data is usually scarce. The three methods of measuring water loss will be used in analyzing the water loss in this study and the appropriate method will be recommended considering the local condition on .Many water utilities have been developing new strategies to reduce losses to an economic and acceptable level in order to preserve valuable water resources. Thus water utility benefits by

(a) Saving the production costs of the water,

(b) Used to reducing NRW and increasing revenues through sales of water saved

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

(d) Reducing NRW and increases in utility rates, and thus maintaining better consumer relations. NRW Management Strategy in Western Central Region of the following countries Afric, Asia Latin, America, the Caribbean and North Americam

Source: Global Water Supply and Sanitation Assessment 2000(WHO-UNCEF)

Figure-2.7 NRW Management Strategy in Western Central Region of the above country

3.0 DISCRETION OF THE STUDY AREA

3.1Introduction

Tilili is a town of GuagusaShigudad woreda in West Gojam Zone of Amhara region. It is located at about 429 kms North west of Addis Ababa and about 138 km South of Bahir Dar town. It lies at an average elevation of 2440 m,a.s.l and the area bounded between the geographic coordinates UTM (Adindan) 37p 278503 East 1203386North. within the head waters of Fetam River catchment.The topography of the town area is flat to gently rising mild slopes towards North**.**

3.1.2 Climate

It is situated at 2440 m,a.s.l meters above sea level. The town has 1380 ml average annual rainfall and a minimum and maximum temperature of 15oC and 22oC. Tilili town is one of the high land areas of the country. The town including the rural kebeles in its environs has over 1230 residents

3.1.2.1 Rainfall

The average annual rainfall in the area is determined to be 1942mm. The area receives high rainfall distribution with a mono modal rainfall regime (i.e. single maxima rainfall pattern). As it can be noticed from Fig 3.1 below, the “big rain” in the area is from June to September. Months of April and May before the big rains and October and November after are months of moderate rains. Records show that about 44 % of the annual rain falls in the month of July andAugust.

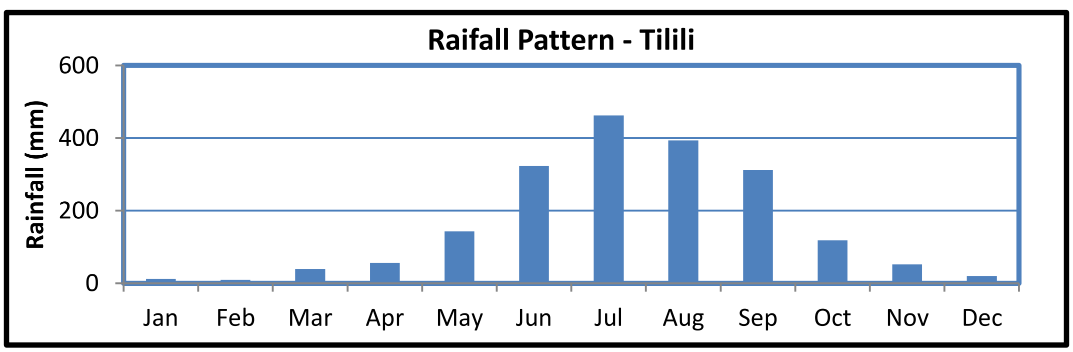


Figure3.1Rainfall Pattern of Tilili Station

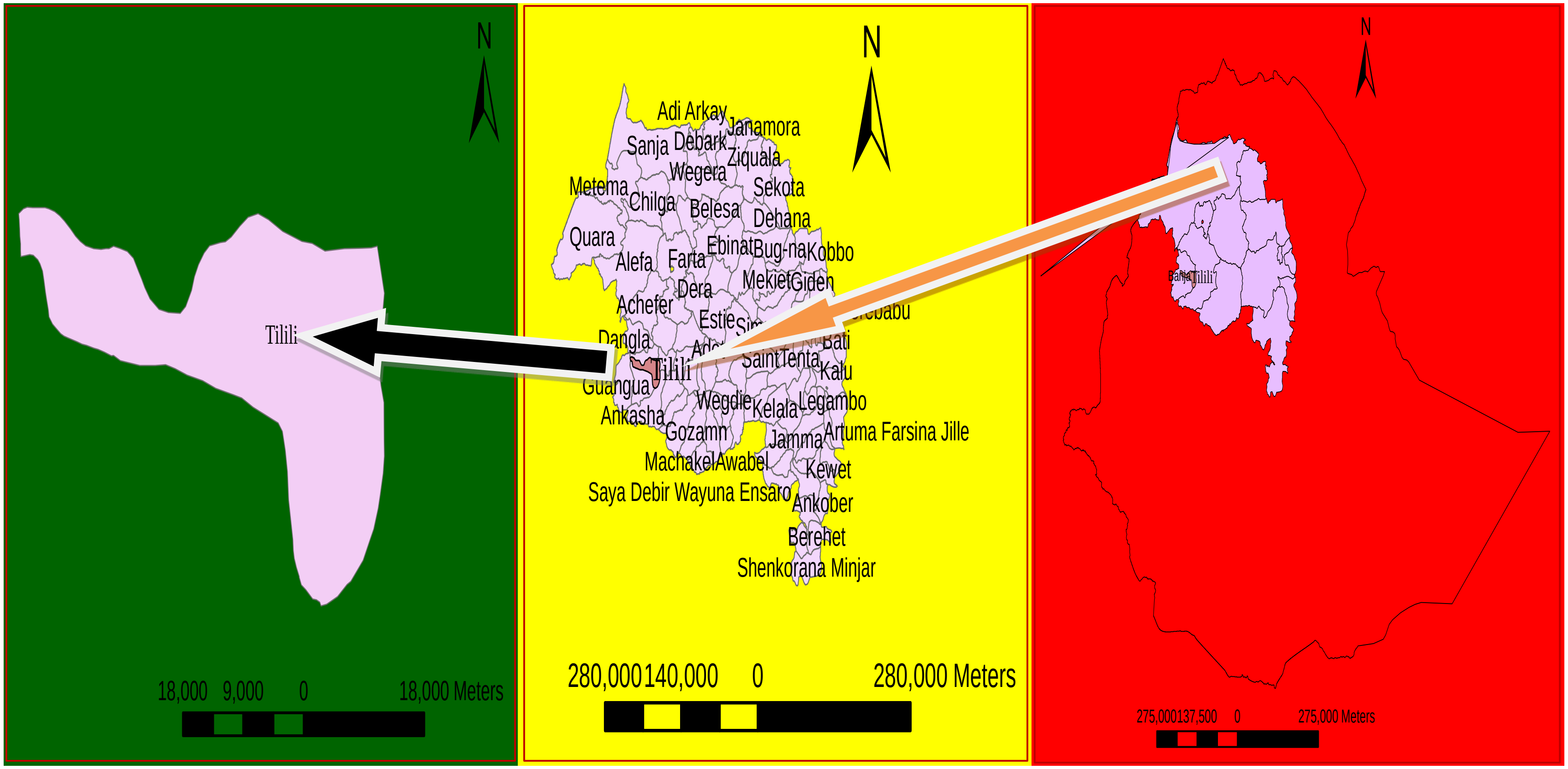


Figure 3.2Location of Tilili Town

3.1.3 Geology and Hydrogeology

3.1.3.1 Geology

The regional geology of the study area is made of dominantly by thick Tertiary volcanic rocks of the Trap Series. These are the Ashange formations, which are known to represent the earliest fissure basalt volcanism in the Northern Ethiopian plateau including the study area. According to the descriptions of previous geological works, they are characterized by thick flows, deeply weathered, crushed, tilted basalts. The rock consists of predominantly mildly alkaline basalts with interceded pyroclastic and rare rhyolites.

It has been explained in the previous geological works, such as Tefera et al. (1996) and others, that the main features of the Cenozoic volcanic geology of the region is that volcanism in these areas began in Eocene, with the outpouring of fissured flood basalts of the Ashange Formation and ended with the eruption of alkaline basalts and trachyte's along the preexisting structures (as recent quaternary formations) which is also mapped around Tilili area featured by its vesicular characteristics.

The distinctive regional geological structures traced in the region are that exist south of Tilili and extending in the South west. Regionally west and north west of Tilili area is dominantly marked by lined scoria cones and spatter cones.

3.1.2 Physiographic

Regionally, the town is surrounded by chain of mountains and drained by several Rivers. The topography of the town area is dissected by several rivers emerging from the surrounding hills along the gentle slopes towards the plain area. Towards the road to Bahir Dar, the land is flat and swampy in the area, where the rivers intersect at the locality of adjacent south of the town. In the eastern side, it is bounded by Afafucha and Kakust Rivers and in the West by Tugjal and Fetam. Geomorphological, in the East, the Tilili town is bounded by the Kurb hills and in the West by Ambera hills that extend further to Sangeb mountain chains in the direction along Bahir Dar road as shown in figure 3-1. The catchment above Tilili has a semicircular shape created as a result of the drainage systems of the tributaries of the main Fetam River contributing to it as head waters. This has given the catchment a feature like closed system, with narrow stretch of low area at the immediate south of the town across the locality of the existing Kakust well. This is expected to create convergence of both surface and groundwater flows and formation of marshy area, as well as groundwater out flows creating artesian conditions at shallower depths. The rivers are sources of groundwater recharge in the area together with aerial precipitation.

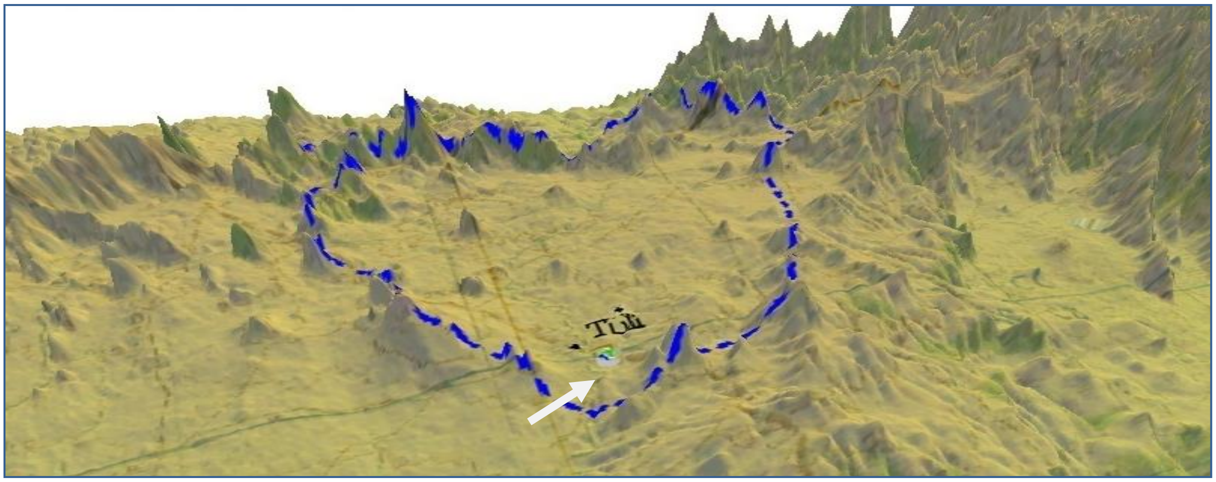


Figure3.3Tilil Town Geomorphological Area

3.1.3Hydrogeology

The two types of rock units in which their contacts and extents have been obscured by their overburden i.e. the older basalts regionally considered to be equivalent to Tetramer formation and the younger quaternary basalts found overlying the older formation have different hydro geological properties and so for possibilities of groundwater occurrence. In situations of Tilili area, except at some localities where joints are tight and the rock is relatively fresh, in most places the rock indicates high degree of weathering and fracturing indicating moderate to good permeability to hold and transmit groundwater. .

3.4Existing Water Supply System

The water source of Tilili is Kakist borehole-1 drilled in 1986 EC borehole-2, 2002respectively by Water Works of the region. The water source ofTilili is from two boreholes with a yield of more than 5 l/s and 9l/s. The BH is located at 1199630 N and 283822 E. The elevation of boreholes is 2417 and 2423 respectively meters above sea level and is in good operational condition. BH-1 has 45 meter depth which is shallow depth and diameter of the well is 6 inch. The type of the pump installed was a submersible pump (Grundfos) Denmark made type SP 17-15 weight 23 kg model A12A0015 063/0004CE with frequency of 50 HZ and RPM 2,900/min as shown in table 3-1. The discharging capacity of the pump is 17m3/hr with 123m head.At present there is shortage of water in the town and people are highly complaining about the shortage. The same transmission main conveying water from the borehole to the service reservoir is being used as gravity main. The line supplies water to the town while it conveys water to the service reservoir. This situation has created a situation whereby the reservoir doesn't get filled up leaving those public taps and household consumers without water. At times the water supply service is trying to shut valves at critical places to allow the filling of the reservoir in order to provide water to those living on the higher parts of the town. The problem arises from the low pump capacity, less pumping hours and lack of standby pump and generator. The characteristics of the water source are shown in table below.

According to the data from the business and economy bureau, the population of the town in 2016 is about 18251Population projection figure will reach 32272 by 2030 and 41190 by 2037. According to the corresponding water demand projection the maximum demand of Tilili that includes domestic demand, commercial and public demands is 11.2 l/s for 2016, 24.3l/s for 2030 and 35.1l/s for 2037. The existing have a safe yield of 7l/s each borehole individually produced 5l/sec and 9l/sec there are located an elevation of 2423m and 2430m respectively. This situation calls for drilling of two new boreholes with a safe yield of 9 l/s considering 20 hours pumping for phase 1 and phase-2 to meeting the demand until the end of the design period

4. METHOLOGY

The research methodology applied in this study was on different literature review based on articles published in academic journals, reports, conference proceedings and text books on methods and tools applied to evaluate water supply coverage and water loss and its management for water supply distribution system.

The main objective of the research was evaluate the water supply coverage and loses by assessing the distribution system in the town and to recommend mitigation measure for the causes of decreasing the water supply coverage and factors which increasing NRW of the town. The water supply coverage of the town was first evaluated before analyzing the water loss by using the percentage of mode of service with current customer to determine the level of connection or per - capita demand of the town .Total water production and total billed data are basic instrument to evaluating the existing water supply coverage and losses of the town distribution system. The total water produced and the actual water consumption as aggregated from the individual customer meters were used as an input for town level analysis, while for the town level analysis, data on water production and consumption that has been previously recorded for different year by town for monitoring purpose will be use and by comparing the coverage of the town with the current guide line of (MOWR). 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 supply coverage in the town, the water loss from the distribution system of the utility was analyzed. The total water produced and the actual water consumption as aggregated from the individual

After evaluating the distribution of water supply coverage in the town, the total water loss was analyzed sub-divided into two levels, the entire town and sub- system level. After calculated the loss at all levels, comparison has been made using different methods of measurement in order to choose a suitable method fit for the local condition. 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 con-junction 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 get water from the same reservoir and located at different locations, time of installation and higher pressure areas has been also analyzed for possible losses of water through customer meters. Contracts (customer meters) were used as an input for the water loss analysis. Water meter accuracy test was conducted and the result was used as an input in the analysis of the total water loss components. The water loss analysis, both apparent and real, is carried out by using the top down water balance approach. Finally, based on the calculated performance indicators and key statistics comparisons has been made, and strategies for loss reduction are developed from international experiences.

4.1Water Supply Situation in Tilili town

4.1.2Existing Source

Currently, Tilili town is supplied from a two boreholes source with a reported water production of 700m^3/day. The main characteristics of the existing well are presented below. Kakust Borehole. According to the water service office technician, the well is being pumped at a rate of 20m^3/hr (5.5 l/s) for about 13 to 14 hrs per day. Records indicate that the well was drilled to a depth of 45m in 1986 EC. The yield was reported as 4.7 l/s. The well has characteristics of artesian flow during rainy season and during no pumping time. The site has swampy characteristics and groundwater has been seen to flow under the wells head during the field visit period. The well is located within the town and the overflow pipe provided is not adequate to accommodate the excess artesian flow that causes leakage under the big well head construction. Hence, it needs critical sanitary cares especially where the water is gushing outside the well casing, and poor sanitary seal and well head construction that lead to possible interference with the surface water body of the marshy area, which also has possible contacts with contaminant sources from the town zone.

4.2Data Sources and Data collection techniques

4.3.1 Data Sources

4.3.1.1 Primary Data of the Town

Primary data were collected from water customers (household survey), interviews of management officials and experts of Ministry of Water Resources, Amahara Water Resources Bureau,Enjibara Water Supply and Sanitation Enterprise ,like water billing, water tariff GPS reading for easting ,northing and elevation and design manual were collected.

4.3.2Pressure Measurement

Pressure Measurement was carried out in different time interval throughout 24 hour in the distribution system at a critical time because the level of pressure is different depending up on the different parameters like demand variation elevation difference or head and pumping system. Pressure sample was taken from customer connection direct from the main line from higher elevation ,lower elevation and pressure reading gauges were installed within critical time when the demand is to be high medium and to be very low those are morning(7:00 to 9:00),noon that is midday(12:00 to2:00) ,after noon (3:00 to 5:30) and mid night at low demand.

Table 4.1Pressure measurement at different location in the distribution system

		Location			Observed pressure			Reservoir
S.N$_O$	Gauging ZONE	X	Y	Z	Meter	Bar	Psi	Elevation (m)
1	J-1	3081.61	3009.49	2430	78.6	7.86	95.9	252.4
2	J-5	3,033.91	3,030.05	2421	68.4	68.4	83.4	2512.2
3	J-22	2,980.86	3,000.38	2420	53	5.3	64.6	2511.5
4	J-30	2,988.43	3,103.41	2425	58	5.8	70.7	2511.6
5	J-40	3,072.34	3,078.65	2445	54.1	5.41	66	2511.5
6	J-15	3,079.38	3,084.97	2445	63	6.3	76.8	2511.9
7	J-10	3,053.84	3,037.80	237	62.5	6.25	76.2	2512.1
8	J-17	3,013.09	3,055.32	2433	63	6.3	76.8	2511.8
9	J-57	3,107.94	3,089.73	2,458	37	3.7	45.1	2511.2
10	J-58	3,095.59	3,083.69	2,460	33	3.3	40.2	2511
11	J-25	3,063.93	3,063.93	2440	49	4.9	59.8	2511.4
12	J-45	3,031.85	3,084.61	2435	64.6	6.46	78.8	2511.3
13	J-48	3,031.18	3,132.62	2452	46.6	4.66	56.8	2511.7

4.3.3 Secondary Data of the Town

Secondary data were collected from Water Supply Network Map of Tilili Town and Amhara design and Supervision bureau. The other most important secondary data source were the water production and water consumption at different year(2008 to2016) from Tilli Town water service bureau current population number 17251 from finance and economy bureau, number of boreholes and their average daily discharge were collected

Table4.2Annual consumption, production and loss of Tilili Town

Year(E.C)	Total Population	customer number	Production(m^3)	Billed(m^3)
2000	13200	753	37281	23114
2001	13808	797	41083	29690
2002	14443	822	49610	30528
2003	15108	926	67477	43594
2004	15646	1200	73988	52677
2005	16334	1300	80985	53115
2006	17051	1461	84428	59866
2007	17518	1715	91231	57222
		Average	65760.38	43725.75

Source :(TTWSB)

According to the information of the Town water service bureau, in the town there are two service reservoirs which have different capacity, location and constructed in different years. Reservoir one(R-1) was constructed in 1986 E.C and located at an elevation of 2505 m.a.s.land its capacity is75m3 and reservoir two(R-2) was constructed in 2001 E.C and located at of 2510m.a.s.land 300m^{3}respectivelyfor the whole distribution systems that makes possible distribution with gravity for all distribution systems that is water directly pumped from the boreholes entered to those reservoirs.

Source :(TTWSB)

Figure 4.3Tilili Town Existing Water Distribution Net work

4.3.5The existing distribution network

The data collected from existing distribution network of the town like Locations of pipelines, Pipe materials, node numbers length, reservoir location 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 the network cover the total 16.5 km distribution pipe line, 3.8 km is pressure main.

Table 4.4Diameter type and Size of the pipe of the distribution

Item No.	Pipe Material	Diameter(mm)	Length (m)	Remarks
2.1	GI	25	757.50	Distribution Line
2.2	HDPE	50	4019.00	Distribution Line
2.3	HDPE	80	720.00	Distribution Line
2.4	HDPE	110	7,980.00	Pressure main
2.5	PVC	150	2544.5	Pressure main
2.6	PVC	200	2,249.50	Pressure main
Sum			18270.5	

Source :(ANRWRB)

4.3.6 Customers Meter

The water meter is an essential utility tool for effective revenue generation equitable customer billing water demand management and generation data for network planning and management. When metering is inefficient; all benefits associated with metering are lost. Revenue losses caused by metering inefficiencies can be reduced by assessing water meter performance by different pressure, identifying the main causes of losses related with customer water meter and applying appropriate intervention measures. Using data from the Tilili town water distribution system, this study examined water meter performance and water meter management framework to help water utilities minimize revenue losses due to metering inaccuracies.

The different metrological class's flow rates testing were indicated that meters with equal capacity and Manufacturer Company will have different minimum and high flow rates depending on their age category. It is important to note that since meters was testing by dividing the customer meter based on their age that is old and new in service meters, by putting master meter as a bench mark. The number of samples subjected to flow tests was limited due to the long flow times required to measure the extremely small flow rates and minimize uncertainty in the measurements (Arregui et al. 2006).

4.3.7Burst Frequency

Types of burst and there frequency has been collected from the town water service bureau they are yearly recorded. It include the location of burst happen and awareness and time of repaired those data are very important parameter to estimate the amount of leakage in the system. The frequency of burst is determined from the run time report and the average run time is fixed by the town water service bureau expert by average in addition to this the level of bursting is identified and the degree or and the quantity is measured by the replaced pipe diameter and the average value has been taken.

Table 4.6 Yearly reported burst frequency

year	Yearly reported pipe burst
2000	20
2001	27
2002	30
2003	29
2004	33
2005	35
2006	37
2007	40
Average Burst	31

Source:TililiTown water service bureau (TTWSB)

4.3.8.Data collection techniques

Both qualitative and quantitative data were collected through Survey (Questionnaire), Key informant interview (KII), Focused group discussion (FGD), personal observation and Global Positioning System (GPS). The questionnaires were prepared for sample water meter customers, the structured KII for the managers and experts in different organizations and the service delivering sections.

4.3.8.1 Sampling meters and properties

There are about 1700 installed customer meter and over 17518 users in Tilili. Testing all of these meters and measuring consumption patterns can only be realized by defining and establishing representative samples. In this study only meters of size 15 mm were considered as they constitute about 92% of all customer meters installed in the TTWDS. There are different types of customers meter according to the town which are the meter manufactured by quality, size and type with their customers 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. The average age of the meters is about 10 years with some meters being more than 20 years old.

$$\varepsilon = \frac{(Vm - Va)x100}{Va} \quad \text{Eqn(4.1)}$$

Where:-ε = %age error V_m=measured value &V_a =actual value (master meter)

4.3.8.1 Meter sampling for accuracy estimation

4.3.7.1 Sample size and sampling techniques

The methodology used in this study for sampling in individual customer meter by taking testing has been used by many researchers. The method for estimating water meter accuracy was adapted from (Arregui,etal.2006) is used for small customer meters and is also applicable to large customers with each user's demand profile assessed separately.

To gather the primary data up on water meter customers survey random sample were selected out of the total in Tilili Town in the entire distribution system by selecting the area according to customer settlement that means relatively old and new settlement customers water meter were tested and, KII ,FGD and survey questionnaire for water meter customers were used in the villages. According to Amhara design and super vision consultant different type of water meters manufacturer 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 (ADSB, 2009)"Tilili Town 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 conditions, one under high pressure medium pressure and low pressure. Under high pressure testing bench where a series of calibrating meters were arranged. Meters to be tested were installed next to the calibrating meter or master meter in parallel. 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 is lost in such cases as most customers are aware of this mechanism and adjust the tap in such a way as not the meter to start register as the water flowing through it. Volume of water is collected to determine how much water lost in this way.

4.3.8.2Improper Metering

During water meter customers survey one of the most critical problem observed was improper metering or false data reporting were a common problem for customer and also town water service bureau:-Reporting over the actual total amount of water consumed by customer

Reporting under the actual water consumed by the customer (NRW) of government

4.3.8.3 Illegal connection

According to town water service bureau some peoples use illegally to fill tankers (normally at night) or to provide water supply to construction house. The utility staff can detect these flows, often high volume over a short period of time; through appropriate flow measurements at DMA meters in Tilili town some customer used illegal connection during construction new house for mudding purpose and for masonry works.

4.3.9Water Consumption

To quantify the magnitude of water loss through the distribution system the distributed (production) and consumption (billed) data were basic instrument and those data were collected from the town water service bureau there were annually around 65760.38m^{3}production 43725.75m^{3}consumption and 22034.625m^3 loss by average were collected.

4.4 Method of Data Analysis

The water supply coverage of Tilili Town is first evaluated before analyzing the water loss. The focus was by evaluating the current water supply coverage with in the town and comparing what seems like the average per capita consumption and level of water connection according standards put towards the Towns. These are highly related to the issue of water loss. After evaluating the distribution of water coverage in the town, the total water loss was analyzed. The total water produced and the actual water consumed as aggregated from the individual customer meters for the recent consecutive eight years were used as an input for the town water loss analysis. After calculating the water losses, comparison was made using performance indicators. Then after, the possible causes of the losses were identified by comparing the losses in conjunction with factors like age of pipe, potential pressure, customer meters etc.

4.4.1 Town Water Loss Analysis

To estimate the total loss of water in the Town was first the total volume of water supplied to the network distribution system (total production) and actual water consumption (billed) for different year were collected and then by subtracted consumption from production and compared with each other. In this case, the data on consumption were aggregated to the town. The water consumption and water production was found from the Town that records on water distributed to each eight subsequent year was found for the Town water service bureau. The other factor considered in the town water loss analysis should be evaluate seasonal changes of water loss particularly the summer (rainy season) and winter (dry season) to compare in which season higher water consumption and loss were happened by used the results of the two seasons for characterize the possible causes of the water losses but the town did not have monthly recorded data due to this lack of monthly data I was forced to evaluate annually water losses.

The reduction and control of water loss is becoming even more vital in this age of increasing demand and changing weather patterns that bring droughts to a considerable number of locations in the world. Many water utilities have been developing new strategies to reduce losses to an economic and acceptable level in order to preserve valuable water resources. In Tilili, water is supplied by a region municipal, Amhara water and sewerage authority, and this is usually the best assurance of an uninterrupted supply of economical and safe water to our people in the to town of the region and country level. The water may lose from different components of water demand those are Domestic like residential, Non Domestic like commercial, industrial, institutional and Fountains like public water uses all this components are generate revenue to the utility, while unaccounted system losses and leakages, are components of water supply demand but they are not associated with total cost revenues, and are a source of wasted production costs. With today's high water production costs and rates, the expense of detecting and mitigating the unaccounted for water and leakages is an attractive option for minimizing operating expenditures. The water utility benefits by:

- Saving the production costs of the water,
- Increasing revenues through sales of water saved,
- Deferring the system expansion and capital expenditures through the capture of lost water,
- Reducing increases in utility rates, and thus maintaining better consumer relations.

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

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

NRW=Real los (Physical loss) +Apparent…………………………………………………..…Eqn(4.2)

4.4.2Performance indicator Assessment

4.4.3Non-Revenue Water: as % of system input volume

Percentage by volume is still recommended as a basic financial PI for non-revenue water but a basic PI for real loss from a water resources viewpoint, it should not be used for assessing any aspect of operational performance management of water losses (Liemberger& Farley, 2004). It is given by the Expression: Where:- Vapparent = volume of apparent loss per day (m3/day)

N_c = Number of service connections

4.4.3.1 Real Loss Analysis Methods

4.4.3.2 Real Losses per service connect

This is the most basic and widely used method. The volume of Real Losses is assessed as the component remaining after Authorized Consumption and Apparent Losses volumes have been calculated, and deducted from System Input Volume. Volumes of Real Losses calculated from a 'Top-Down' Water Balance are always subject to some calculation error, because of errors in the individual components. Some of the most recent international applications of the IWA Standard Water Balance include the facility to calculate the95% confidence limits for the assessed volume of Real Losses, given 95% confidence limits for the other volumes entered in the water balance.

The disadvantages for relying only on Water Balance for assessment of Real Losses are:

- Water Balance gives no indication of the individual components of Real Losses, or how they are influenced by Utility policies.
- Water Balance normally covers a 12-month retrospective period, so it has limited value as an 'early warning' system for the occurrence of new unreported leaks and bursts for these reasons, Real Losses should preferably also be assessed by additional methods, namely:
- Component Analysis of Real Losses
- Analysis of night flows

4.4.3.4Component Analysis of Real Losses

The general principle of assessing some components of Real Losses from repair statistics is well known. Annual numbers of repairs are assumed to represent the annual number of new leaks and bursts; these are then classified into different categories, with different typical flow rates. If average duration of each category of leak or burst is logically assessed, then the annual volume lost from different categories can be assessed. In 1993, an internationally applicable overview concept known as 'Background and Bursts Estimates' (BABE) was developed from this basic building block for calculating components of Real Losses based on the parameters which influence them.

In BABE analyses, components of Real Losses are considered to consist of:

- Background leakage at joints and fittings, flow rates too low for sound detection if non-visible
- Reported leaks and bursts – typically high flow rates but short duration
- Unreported leaks and bursts – moderate flow rates, average duration depends on method of active leakage control

The objective of this performance indicator is to measure the efficiency of the water supply system (Mutikanga*et al.*, 2010). It is given by the expression:

Real loss (l/cnon/day) = $\frac{Vreal}{Nc}$……………………………………………………Eqn(4.3)

Where:- V_{real}.is the volume of real loss per day (m3/day)

4.4.5 Infrastructure leakage index

It is the ratio of the current annual real losses (CARL) to the unavoidable (technical minimum) annual real losses (UARL). It is calculated as follows (Liemberger& Farey,2004).

ILI=$\frac{CARL}{UARL}$……………………………………………………………Eqn(54.4)

Where: - CARL- Is current annual real los

4.4.6UARL-Un avoidable real los

5.1.5.1 Unavoidable Annual Real Losses (URL)

Real Losses cannot be eliminated totally. The lowest technically achievable annual volume of Real Losses for well-maintained and well-managed systems is known as Unavoidable Annual Real Losses (UARL).The relationship between Current Annual Real Losses (CARL)and UARL from an IWA water balance represented by the large rectangle. Using the four methods of leakage management (the four arrows), Real Losses can be controlled, but (at the current operating pressure) cannot be reduced any further than the UARL. System-specific values of UARL can be assessed using a formula developed by the IWA Water Losses Task Force. (Lambert et al,1999). Data required for this assessment are the number of service connections Nc, the length of mains Lm (km), the length of private pipes (Lp in km) between the street: property boundary and customer meters, and the average operating pressure (P meters).

The general equation for UARL is

1) UARL (litters/day) = (18 x Lm + 0.8 x + 25 x Lp) x P
2) Or UARL (liters/service connection per day) =$(\frac{18}{Dc} + 0.8 + 25x\frac{Lp}{Nc})xP$
3) Or UARL (l/service/day/m pressure) =$(\frac{18}{Dc} + 0.8 + 25x\frac{Lp}{Nc})$
4) Or UARL (liters/km mains/day) =$(\frac{18}{Dc} + 0.8x25x\frac{Lp}{Lm})xP$
5) Or UARL (litters/km mains/day/m pressure = $(18 + 0.8Dc) + 25x\frac{Lp}{Lm})$

Where: -Lm = mains length (km);

Nc = number of service connections

Lp = total length of private pipe, property line to customer meter (km)

P = average pressure (meters);

DC = density of connections/km mains

Based on AAWSA business plan phase-II (2011draft report From the above different equation the most applicable equation is the first equation so we can estimate UARL by using this equation.

UARL (litters/day) =$(18xLm + 0.8xNc + 25xLp)xP$…………………………Eqn (4.5**)**

If different measurement units are required (e.g. US Gallons, miles, psi etc as in Lambert et al, 2000), the three coefficients in the equation (18, 0.8, 25) can easily be recalculated from first principles to suit the alternative units. However, when the ILI is calculated, because it is non-dimensional, it is always internationally comparable, whatever initial measurement units have been used to calculate the(UARL).This equation, based on component analysis of Real Losses for well-managed systems with good infrastructure, has proved to be robust in diverse international situations (AAWSA – Business Plan Version 01 Draft Report May 2011, Lambert and McKenzie, 2002), and is the most reliable predictor yet of 'how low could you go' with real losses for systems with more than 5000 service connections, connection density (Nc/Lm) more than 20 per km, and average pressure more than 25 meters.

Town yearly total water loss is computed by using water production &consumption in the table below. 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:

Q = $30.39375 * A * \sqrt{P}$ ………………………………………………………………Eqn(4.6)

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 section1992).

4.5 Analysis Of Town Water Supply Coverage

4.5.1 Introduction

Problems in provision of adequate water supply to the rapidly growing urban population are increasing dramatically. Water demand in the domestic sector of developing cities and town including Tilili increases through time that as a result demands for additional water sources and infrastructure. Despite to these, the financial capacity of the town is low to satisfy the growing demand. Financial constraint is one of the major factors for the low water coverage of the water supply but poor management of the existing water supply system also has a great impact for the low coverage. Beside to the overall low supply coverage, supply disparity exists among different localities. Therefore evaluating the intra-town distribution of the water supply is important in order to identify the problematic areas and intervene accordingly.

Water supply coverage is usually evaluated based on different aspects like the quantity, quality, paying capacity of the people, distance, etc., but the aim of this research is not to evaluate all these but related to the quantity of supply and level of connection that are related to the water loss. In this part of the analysis, the number of domestic connections per family and the average daily per capita consumption is used to analyze the domestic water supply coverage for the entire town. The level of coverage has been also compared with other town or cities of developing countries. Beside to the statistical analysis for the entire town, the distribution of the average daily per capita consumption and connection per family has been evaluated using a map for some part of the town.

4.5.2Level of Connection

Access 4to water supply may be evaluated using the amount of water consumed and the level of connection. For evaluating the amount of water consumption, the annual water consumption is converted to average daily per capita consumption using the population data of the Town. The number of domestic connections per family has been also used for analyzing the level of connection as elaborated below. Coverage calculations made by (AAWSA – Business Plan Draft Report May 2011) are usually based on the following approach;

The average need per capita is globally estimated to 110 lcd.

The coverage is calculated through the following formula:

$$\text{Coverage (\%)}=\frac{(Total\ production\ x\ (1-\%\ of\ physical\ losses))}{Total\ population for\ casted x 110 lcd}\text{..........................Eqn(4.7)}$$

$$\text{Coverage (\%)}=\frac{(number\ of connection*average\ family size)}{Total\ population for\ casted}\text{..........................Eqn(4.8)}$$

4.5.3 Average Daily per-capita Demand

The volume of water consumed for domestic purpose has been aggregated to the Town to analyze the distribution of the water coverage to the town .Evaluating the domestic water supply coverage using volume of consumption may not allow realizing the distribution comparison among the standards.

To estimate the per capita demand of the town the annual consumption should be converted to average daily per capita demand using the total population number. Average daily per-capita demand was estimated by the following expression:

$$\text{Per-capita demand }=\frac{Annual\ consumption(m3)*{}^{1000l}/_{m3}}{population number or\ user*365}\text{...............................Eqn (4.9)}$$

4.5.4The Existing Distribution system

Types of distribution System for efficient distribution it is required that water should reach to every consumer with required rate of flow depending up on the methods of distribution. The distribution system is classified as gravity system, pumping system and dual system.Tilili town water supply designs adopt a dual system, since both methods are available. The distribution systems are generally, supplied by gravity from the associated reservoirs into the town and pumping system from the borehole into service reservoir by submersible pump.

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

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

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

As defined in water distribution system models, reservoirs are nodes that represent infinite sources or sinks of water, such as lakes. Tanks are nodes with fixed storage capacity and varying volumes during distribution. Pumps are devices that impart energy to water thereby increasing its head. Valves limit the pressure or flow at points in the system. A loop is a sub-component of a distribution system: it consists of an entity made of nodes all connected through links. It is an important component of a model because mass in and out of a loop can be accounted for and used to solve for flows.

4.5.5 Water Distribution System simulation

The term simulation generally refers to the process of imitating the behavior of one system through the functions of another. The term simulation refers to the process of using mathematical representation of the real system, called a model. Network simulations, which replicate the dynamics of an existing or proposed system, are commonly performed when it is not practical for the real system to be directly subjected to experimentation, or for the purpose of evaluating a system before it is actually built.

4.5.6Applications of Water Distribution Model

Water distributions models (WDM) can be used to junction to pipe analyze variety components of the system of the network of the distribution system of the town. Water distribution network simulations are used for a variety of purposes, such as

- Long-range master planning, including both new development and rehabilitation
- Fire protection studies

- Water quality investigations
- Energy management
- System design Computer Applications in Hydraulic Engineering. Haestad Press, and Daily operational uses including operator training, emergency response, pressure and leakage in the piping Water bury Connecticut (Haestad Methods, Inc. 1997).

4.5.7Types of Simulations

After the basic elements and the network topology are defined, further refinement of the model can be done depending on its intended purpose. There are various types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. The two most basic types are Modeling, Analysis, and Design of Water Distribution Systems. American Water works Association, (Cesario, A. L and Denver, Colorado1995.)

4.5.7.1 Steady-state simulation

As the term implies, steady-state refers to a state of a system that is unchanging in time, essentially the long-term behavior of a system that has achieved equilibrium. Tank and reservoir levels, hydraulic demands, and pump and valve operation remain constant and define the boundary conditions of the simulation. 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. Computes the states of the system (flows, pressures, pump operating attributes, valve position, and so on) are assuming that hydraulic demands and boundary conditions do not change with respect to time.

4.5.7.2 Extended-period simulation (EPS):

Determines the quasi-dynamic behavior of a system over a period of time, computing the state of the system as a series of steady-state simulations in which hydraulic demands and boundary conditions do change with respect to time.

The results provided by a steady-state analysis can be extremely useful for a wide range of applications in hydraulic modeling. There are many cases, however, for which assumptions of a steady-state simulation are not valid, or a simulation is required that allows the system to change over time. For example, to understand the effects of changing water usage over time, fill and drain cycles of tanks, or the response of pumps and valves to system changes, an extended-period simulation(EPS) is needed. It is important to note that there are many inputs required for an extended-period simulation. Due to the volume of data and the number of possible actions that a modeler can take during calibration, analysis, and design, it is highly recommended that model be examined under steady-state situations prior to working with extended period simulations. Once satisfactory steady-state performance is achieved, it is much easier to proceed into EPSs.

4.5.8 Hydraulic Modelling Software

Software is recommended that the up to date Water CAD6.5 software for an unlimited number of pipes . 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 understands network operation to identify higher and lower pressure zones. The use of the above appropriate for the development of the skeletal and all mains models of Tilili town water supply network.

4.5.9 Population

The base population used for projection of population of the town is the 1994 census population data to determine the existing water supply coverage by mode of service and to design future water demand of the distribution system.

Table 4.6 Population Number and Growth Rates at 2015

Year	Growth rate urban	Population
2015	4.5%	17518

Source:-Tilili Town finance and economy bureau

4.5.10Nodal Demand

The average day, commercial and institutional demand, including non-revenue water, were distributed to the nodes of the network model according to the land use plan as per the according to the following method (ADSB, 2001):-

From the land use plan and site the spatial distribution of the existing and proposed infrastructure were identified;

In order to simplify the analysis, the land is reclassified into two major categories; domestic and commercial and institutional; Using a digitized map the area supplied from each node is delineated, measure and tabulated for each category;

The demand area ratio for each category is computed

Nodal demand is computed by multiplying the area at each node with the demand area ratio for the specific consumer group

$$\frac{Qi}{Qt} = \frac{Ai}{At} \qquad Qi = \frac{QTxAi}{At} \qquad \text{Eqn (4.9)}$$

Where: Qi -is average day demand of the junction

Qt - is average day demand of the end of phase

Ai - is area of the junction

At - is total area of phase

$$ND = \frac{PDxPi}{Pt} \qquad \text{Eqn (4.10)}$$

Where:-Nd = Nodal demand

Pi = population in each district of the service area of the junction

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

4.5.11 Domestic Water Demand Projection

In this section deals with the estimation of the present and future water requirements for Tilili town using the prevailing modes of services and commonly known demand categories, which are relevant to urban centers and compare with the current water coverage of the town by forecasting the population in CSA method.

Water demand adjustment factors as well as water demand growth rates, which are presumed to be essential in defining phased design capacities of the major physical unities of the proposed water supply schemes are also discussed and estimated from different sources & data.

4.5.12Design Period

Design period is the number of years for which a provision is made in designing the capacity of the various components of the water supply schemes. The design period should neither be too long nor should it be too short and it is guided by the following considerations. Useful life of the component structure and their chance of being old or obsolete; the design period cannot exceed the useful life of the component structure.

The design criteria guide line adopted for Tilili Town is based on Urban Water Supply Design Criteria (Ministry of Water Resources, January 2006) and Design Guide Line for Water Supply

Projects (Amhara Water Works Design and Supervision Enterprise). However, some adjustments were taken where appropriate. As per the Design Criteria the design period is 22 years and the water supply scheme will be implemented in two stages. Stage I will cater for demands the year 2015 to 2027 (Twelve years) and Stage II will cater for demands 2027 to 2037 (ten years). The lifetime of a water supply system component are usually estimated in terms of its technical and economical period.

Ease and difficulty that is likely to be faced in expansion; the water supply project of Tilili town includes different components such as intakes, service reservoirs, pumping units and treatment plants which can be easily expanded when there is a need and the mains which cannot be easily be considered expanded hence larger design period should.

Amount and availability of additional investment; according to the country condition, allocation of fund for water supply schemes is very difficult and costly. But developing such scheme is by itself an investment since it has a return.

Rate of interest: because of large cost of water supply schemes, such project cannot be financed by government only rather financed by loaners (donors) therefore increasing design period is a problem

Anticipated rate of population growth: from province population data, it has been seen that population of Tilili town is growing at an alarming (faster) rate hence high design period will have a problem.

Based on the above explained design period affecting factors and different guide lines of studies a medium design period, which is 21 years is selected, .The population projection growth rate is based on 1994 Population Household Census of CSA and the base year population is taken from CSA 2007.

For the projection of the population, the low variant growth rate given in the 1994 population and housing census of Ethiopia result for Amhara Region Volume II Analytical Report Dated December 1998 have been considered. For projection the exponential growth rate model has been used.

4.5.13 Population Forecasting

Since population is always a relevant factor in estimating future water use, it is necessary to forecast what the future population will be.

Generally, population growth rate of a town is affected by birth rates, death rates, migration, and economic activities etc. Tilili town grows quit remarkably in the past few decades when compared with other towns in the region particularly and in the country in general. In addition, it is the political center for the Amhara region. The new economic activities, land investment, and similar policy measures will increase the population rapidly.

4.5.13.1Population forecasting methods

Following the selection of proper design period, the design population shall be determined. The projected population growth during that time may be derived from available demographic data. Socio-economic factors should be taken into account when estimating the rate of population growth, such as family planning, migration and the level of economic prosperity.

According to Urban Water Supply Design Criteria report (January 31, 2006)Population figures are available from the 1994 population and housing census of Ethiopia, published by the Central Statistical Authority (CSA), Office of Population and Housing Census Commission. The CSA makes population projections for towns (urban) and rural areas by region. Population growth rates will vary from town to town, depending on numerous factors. The basic growth rates for domestic water demand calculation will be those available or implied from the above-mentioned CSA publications for the corresponding population.

In case of Ethiopia, the basic population data for the design has to be adopted from the census report of the Central Statistical Authority (CSA). Where possible the population data should incorporate classification based on the following issues (AAWSA – Business Plan Version 01 Draft Report May 2011) and Water Resources Administration Urban Water Supply and Sanitation Department (Urban Water Supply Design Criteria January31, 2006).

- Age and sex group
- Economic activity
- Religion
- Education characteristics

Further, the average household size of the settlement should properly be estimated in order to determine the possible number of connections that the water supply service is expecting for different service levels. However, demographic data are not always reliable and should be checked against the actual population in the area to be served by the new system. If necessary, additional data should be collected.

It is essential that the water supply system is designed to meet the requirements of the population expected to be living in the community at the end of the design period by taking into account the design period and the annual growth rate.

The various methods adopted for estimating future population are the following This method is based on the assumption that population increases at constant rate from decade to decade. It is suitable for old and saturated towns giving lower values.

Arithmetic increases method ,Incremental increase method, Geometric increase method,Decrease rate of growth method ,Simple graphical methods ,Master plan method ,Master plan method Logistic curve method and Exponential growth rate method:

Table 4.7 Urban Population Growth Rates by Region

Region	Urban Population Growth Rate						
	2000	2005	2010	2015	2020	2025	2030
Addis Ababa	2.95%	2.96%	2.88%	2.64%	2.29%	1.90%	1.75%
AFFAR	3.84%	4.27%	3.94%	3.99%	3.62%	3.63%	3.48%
AMHARA	5.09%	4.53%	4.67%	4.41%	4.25%	4.05%	3.85%
B/GUMUZ	4.34%	4.65%	4.35%	4.51%	4.05%	3.99%	3.59%
DIRE DAWA	4.93%	4.40%	4.15%	3.78%	3.51%	3.27%	3.12%
GAMBELLA	5.15%	4.56%	4.10%	4.64%	4.01%	4.16%	3.78%
HARARI	4.19%	4.00%	3.63%	3.44%	3.25%	2.98%	2.99%
OROMIYA	5.29%	4.88%	4.74%	4.53%	4.32%	4.08%	3.84%
SNNPRS	5.50%	4.94%	4.70%	4.46%	4.25%	4.02%	3.77%
SOMALI	4.61%	4.65%	4.52%	4.39%	4.16%	3.90%	3.68%
TIGRAY	5.06%	4.63%	4.56%	4.41%	4.22%	4.04%	3.81%

(Source: ESP Component 3 Urban Planning Model December, 2001 E.C)

Exponential growth rate method:

This method of population projection is preferred from other methods due to the reason that the other methods: Are based on the assumptions that do not represent the real situation of Tilili town.

Require a number of decade censuses data and

The exponential method is adopted by the CSA, as CSA is a specialized and authorized body, which among other responsibilities handles all demographic matters in the country.

$$P_t = P_o e^{rt} \quad \text{Eqn(4.11)}$$

Where: **P_t**-is projected population at time t

P_o-is initial population at time t_o

r-is annual growth rate and **t**-number of years

4.5.13.2 Base Population

The population projection growth rate is based on 1994 Population Household Census of CSA and the base year population is taken from CSA 2007 the total projected population from the census data at 2015was taken as the base population17518.Interpolation was used to obtain the growth rate profile for the year in between 2016 and 2027 and extrapolation for year in between 2030 and 2037

Table- 4.8Population Growth rates

Year	Growth rate urban (r)
2016	4.41%
2017	4.41%
2022	4.25%
2027	3.85%
2032	3.70%
2037	3.70%

Source- Growth rate " **r** " taken from CSA, 1994 population and housing census.

Applying the above growth rate in the exponential model, the urban population of Tilili is projected up to year 2037 and is presented in Table Appendix -A

4.5.14 Water demand Projection

Design of water system requires estimates of expected water demand applicable to the size of pumping equipment, transmission and distribution pipelines and storage facilities. Estimating the water demand for particular town depends on the size of population to be served, the availability of wastewater services and the purpose of demand. It varies according to the requirement of the domestic population, institution industrial and social establishment.

The per capital water demand for a varies demand categories depending on the size of the town and level of development, the type of water supply scheme, the socio economic condition of the town and the climate condition of the area.

The water demand of Tilili town has three main categories: Domestic demand, Institutional &Commercial demands.

4.5.14.1 Domestic Demand

As earlier, domestic water demand includes water used for basic needs such as drinking, cooking, ablution, washing clothes and utensils and cleaning houses. The average amount of water used per person per day varies from country to country as well as from place to place within a country. The major important factors for these variations are:

- Affordability and willingness of people to pay for water supply services
- Level of water supply services to be provided
- Cultural Practices
- Climatic conditions
- Level of socio economic development
- Water Quality Standard and etc.
- Level of Water Supply Services

The Table below shows the percentage of people using various connection types (connection profile) and Per-capita water demand based on population size in Tilili towns for a year 2000 and 2030. This has been based on the Average National (ESP) for the specific town category. Interpolation was used to obtain the per-capita water demand and connection profile for the year in between 2015 and 2027 and extrapolation for year in between 2030 and 2037.In order to project the domestic water demand of Tilili town the following procedures have been followed:

- Setting Population percentage distribution by mode of service
- Establishment of per capita water demand by purposes of each mode of service.
- Projection of services by mode of service.
- Setting adjustments for climate
- Setting adjustments for socio-economic conditions and.
- Projection of domestic water demand

Taking the above consideration into account, the proposed projection of modes of services is given in Table 4.9 Owing to the presence of relatively protected traditional water sources; it may not be possible to bring all such water source users to join the water supply service until the end of the design period. Furthermore the new master plan that is under preparation at the time of this feasibility study is expected to incorporate a considerable rural area as part of the town proper. As a result of the above mentioned reasons the total percent of served population is targeted at 90% at present and 100% at the end of the design period. Breakdown of water demand by purpose and corresponding projections are shown in subsequent tables.

Table-4.9 Population Percentage Distributions by Mode of Service

Coverage	Bas		2017	2022	2027	2032	2037
HCU	6%		7%	8%	8%	9%	9%
YCU	19%		22%	24%	27%	28%	31%
YSU	19%		21%	23%	25%	25%	25%
PTU	47%		45%	42%	40%	38%	35%
Total	91%		95%	97%	100%	100%	100%

Source: Towns Water Supply Feasibility Study and Engineering Design Report for (Amhara region)

According to MOWR Urban Water Supply Design Criteria January 31, 2006 report Domestic water demand for the following categories of consumer stage one for ten year and after ten year.

Table-4.10 Breakdown of per capita water demand by purpose (2006)

Stage-1	Stage-2
- House connection (HC). 50 l/c/day	70 l/c/day
Yard connection, own (YCO). 25 l/c/day	- 30 l/c/day
Yard connection, shared (YCS). 30 l/c/day	- 40 l/c/day
Public tap supplies (PT). 20 l/c/day	- 25 l/c/day

Table-4.11Breakdown of per capita water demand by purpose (2016)

Activity	HTU	YTU	NTU	PTU	PTU (rural)
Drinking	4	4	4	4	4
Cooking	8	5	6	3	3
Ablution	5	4	5	3	2
Washing Utensils	6	4	5	2	2
Washing Clothes	25	15	10	8	2
House cleaning	4	4	4	2	1

Activity	HTU	YTU	NTU	PTU	PTU (rural)
Bath or shower	25	15	10	2	2
Flushing Toilet	3	1	1	1	
Total	80	50	45	25	15

Source: Urban Water Supply Design Criteria Report (January,31-2006

Table-4.12 Projected per-capita demand by mode of service (2037)

	Mode of Service (2037)				
Purpose	HTU	YTU	NTU	PTU	PTU (rural)
Drinking	5	5	5	4	4
Cooking	10	7	7	5	4
Ablution	7	5	5	3	4
Washing Utensils	10	5	6	4	3
Washing Clothes	30	15	13	5	2
House Cleaning	8	6	5	4	3
Bath and shower	30	15	10	5	2
Flushing Toilet	10	3	2	1	1
Total (l/c/d)	110	60	53	31	23

Source: Urban Water Supply Design Criteria Report (January,31-2006)

Total domestic demand = **HC + YCO + YSC + PT**

Adjusted domestic demand = total domestic demand ∗ socio economic factor ∗ climate factor

Adjustment for climatic effects and socioeconomic conditions are shown in tables below

Table-4.13Adjustment Due to Climatic Effects

Group	Mean annual Precipitation (mm)	Factor
A	600 or less	1.10
B	601 – 900	1.05
C	901 or more (i.e. 1942 for Tilili)	1.00

Source: Towns Water Supply Feasibility Study and Engineering Design Report for (Amhara region Tilili having a mean annual precipitation of 1200 mm belongs to group C with a climate adjustment factor of 1.00.

Table-4.14 Adjustment due to socio-economic conditions

Group	Description	Factor
A	Towns enjoying high living standards and with very high potential development	1.1
B	Towns having a very high potential for development but lower living standards at present	1.05
C	Towns under normal Ethiopian conditions	1
D	Advanced rural towns	0.95

Source: Towns Water Supply Feasibility Study and Engineering Design Report for (Amhara region Tilili is considered as towns under normal Ethiopians conditions and therefore is categorized with the towns of group C with a factor of 1.00.

4.5.14.2 Non Domestic Water Demand

Non domestic water demand can be broadly classified in to the following major categories;-

- Institution water demand
- Commercial water demand
- Industrial water demand
- Animal water demand
- Firefighting water demand

4.5.14.2.1 Institutional and Commercial Water Demand

The water requires for school, hospital, health center, government office service religious institutions and other public facilities is classified as institutional water demand. Whereas the water reached for restaurant, shopping centers, hotels, local drinks and other commercial purpose is classified as commercial water demand. Both water demands are termed as public water demands. Such a demand category is expected to be relatively high in Tilili where there are many government offices, school and hotels. In a town where there is sample supply water, it is often the case that the public service giving institution likes hotels, restaurants, offices, college, school, etc. Tend to consume large amount of water merely because they are able to afford better than other demand group. Connection priority is also given to such institutions because of their economic and social

4.5.14.2.2 Industrial demand

The water requirements for this purpose defend up on the type and size of the industry

4.5.14.2.3Unaccounted for water

As computed from the past eight years production and consumption records of the water supply service, the average unaccounted for water of Tilili is about 34%of the total demand. This percentage is expected to be reduced to 20% by 2025 and stabilize until the end of the design period. Water Demand Factors

4.5.14.2.4 Average Water Demand

Urban average water demand is considered to be the combined total demand from domestic, commercial, institutional, industrial and non-revenue water. The average water demand and represent the daily demand of the town average over the year.

ADD=TDD+PD+UFW…………………...………………………………………………Eqn(4.12)

Where: ADD=Average day demand

TDD=Total day demand

PD=Public demand

UFW=Unaccounted for water

4.5.14.2.5 Maximum Daily Demand (MDD)

The ratio of the maximum daily consumption to the average annual daily consumption is the max day factor and the factor varies from 1.2 to 1.3 (MoWRD, 2006). For this specific research, 1.3 is taken to be economical for design purpose. This demand is used to design source capacity, riser mains components and pump capacity.

MDD=ADDxMDF…………………………………………………………………Eqn (4.13)

4.5.14.2.6 Peak hour Demand (PHD)

The peak hour demand is the highest demand of any one hour over the year. It represents diurnal variation in water demand resulting from behavior patterns of the local population. Usually the peak hour demand is significantly influenced by the size, mode of service and social activity patterns of the town. Further experience based studies show that the peak hour factor is greater for smaller population figures. The population of the town is about 17815 which is in the range of less than 20,000 Water Resources Administration Urban Water Supply and Sanitation Department Urban Water Supply Design Criteria report (January 31, 2006) . Therefore, depending on the information shown in table 6.9 below, the peak hour factor is taken as 2 and from the table the peak hour factor varies inversely with the size of the consumer.

Table -4.15The peak hour factor

Population range	Peak hour
<20000	2
20000-50000	1.9
50000-100000	1.8
>100000	1.6

Source; AWSA

PHD =MDDxPHF ……………………………………………………………………Eqn(4.14)

4.5.14.2.7 Water Source Assessment

After assessing the water demand, the next step in designing a water supply system is the processof choosing the most suitable water source or combination of source of water. The source mustbe capa ble of supplying enough water for the community throughout the design period.The water sources will be selected to meet the expected total demand (including unaccounted forwater) for the relevat design period.

Actual pumped discharge rate will be determined accordance with required maximum dayde mand and suitable pumping yield.

No of borehole $\frac{MDD\ @endofdesignphase}{Safeyield}$……………………..Eqn (4.15)

Safety factor is 1.2 (range) sources

Safe yield=7 l/s (the current average production from the existing boreholes is about 7 l/s)

This project includes two phases, phase 1 and phase 2.

Phase 1 starts from 2016 - 2026

Phase 2 starts from 2027 – 2037

Phase-1

MDD = 26.6 l/s (for the population27721**)**

No of borehole $=\frac{MDDat2026}{Safe\ yield}$ ……………………………………………………………………….Eqn (4.16)

Phase -2

The maximum day demand of the town in phase two (2037) is estimated to be 45.7 l/s while that of in phase 1 is26.6l/s. therefore an additional maximum day water demand is 19.1 l/s is required for phase two implementation.

Therefore No of borehole $=\frac{MDDat2037}{Safeyield}$ ……………………………………………………Eqn(4.17)

4.5.14.2.8 Design of Ground Water and Pump

Water well has to be designed to get the optimum quantity (safe yield) of water economicallyfrom a given geologic formation. The choice of open wells or boreholes and the method of welldesign depend up on topography, geological conditions of the underlying str ata, and depth ofground water table, rain fall climate and the quantity of water required.

4.5.14.2.8.1Pump

A pump may be defined as a mechanical device which converts the mechanical energy supplied to hydraulic energy & transfers the liquid through the pipe line there by increasing the energy ofthe flowing liquid**.**

4.5.14.2.8.2 Pump System Design

There are different types of pump systems are adopted for water supply like centrifugal, booster that will be used for zonal reservoir and submersible pump are common types. According to the design aspect of Tilili Town water supply adopted submersible pump

4.5.14.2.9Determination of Pipe Size

The size of the pipe can be determined by considering the discharge through the pipe and permissible velocity of the flow through the pipe.

4.5.14.2.9.1 Design of suction pipe

Generally velocity of flow in the suction pipe may vary 1.5 to 3m/s according to 9/25 Towns Water Supply Feasibility Study and Engineering Design Report (Amhara region) for Tilili Town assumed to be 1.9m/s

4.5.14.2.9.2 Design of Rising Main

For pumping a particular fixed discharge of water, it can be pumped through bigger diameterpipe at low velocity, or through lesser diameter pipe at very high velocity. But if the diameter of the pipe is increased, it will lead to the higher cost of the pipe line. On the other hand if thediameter of the pipe is reduced, the increased velocity will lead to higher friction head loss andwill require more horse power for the required pumping.

The following formula given by Lea is commonly used in determining the diameter of thepu mping mains.

$$D = 0.97 \text{ to} 1.22 \sqrt{Q} \quad \text{..........Eqn(4.18)}$$

Where D = Economical diameter of pipe in meters

Q = Required discharge of water to be pumped in m3/sec

4.5.14.2.9.3 Determination of total lift of the pump

The total head against which the pump should work includes static head against total loss of head due to friction, entrance, exits etc.

$$H_l = \frac{f*l*v2}{2*d*g} \quad \text{..........Eqn(4.19)}$$

Where: Hl=head against the pump(m) F=friction factors of the pipe

L= length of the pipe (m) g= gravity (m/s2)

V= velocity of the pump (m/s) d= diameter of the pipe (mm)

Cast iron is selected for the raising main; length of pipe from collecting chamber to servicereservoir and this formula gives optimum velocity of water flow between 0.8 to 1.35 m/sec

Head loss due to bends and valves, $He = \frac{1}{2} * \frac{V^2}{2g}$ ....Eqn(4.20)

Table-4.16 Expected Depth and Dynamic Water Level of the Boreholes Used in Design

Year E.C	Name	Location (m) X	y	Depth(m)	Casing Diameter (mm)	Safe yield(l/s)	Pump Position(m)	Elevation(m)	Remark
1986	BH-1	283822	1199639	45	200	5	2390	2435	Existing
2002	BH-2	283495	1199250.7	70	200	9	2388	2423	Existing
2018	BH-3	283865	1199131.	77	200	9	2388	2430	New phase-1
2029	BH-4	284009.	1202746.7	140	200	9	2365	2470	New Phase 2
2029	BH-5	283963	1202248.7	143	200	9	2365	2473	New Phase 2

Source Amhara Design and super vision bureau (AD&SB)

Total head = $H_L + h_L$=

Static head against which the pump lift(Hs)

Hs = (elevation of service reservoir − elevation of borehole)

Power Required For P = Yw*Q*HT ……………………………………………..........Eqn(4.21**)**

Whereby =unit weight of water

Power input $P_p = \frac{\gamma * Q * H}{\eta}$...Eqn(4.22)

4.5.14.2.10 Service Reservoir Sizing

Water storage requirements should take in to consideration of the peak daily water uses and maximum hourly demand, the capacity of the normal and stand by pumping equipment, the availability and capacity of auxiliary power, the probable duration of power failure and promptness with which repairs can be made, and to furnish water for such emergencies as fire fighting or accidental break downs.

4.5.14.2.1 Determination of Storage Capacity

The required reservoir capacity is determined by using analytical and mass-curve method to satisfy the peak hour demand of the population.

When water is supplied for balancing the variable demand against a constant rate of pumping for

20 hours, the analysis of storage capacity can be calculated as follows:

By considering daily maximum water demand and supply, 5% for miscellaneous losses and 10% for fire demand, the reservoirs capacity determined by mass curve.

Storage volume = maximum excess surplus +maximum excess demand

4.5.14.3Water Distribution System

4.5.14.3.1Types of Distribution System

For efficient distribution it is required that water should reach to every consumer with required rate of flow depending up on the methods of distribution. The distribution system is classified as gravity system, pumping system and dual system.

For Tilili town water supply designs adopted a dual system, since both methods are available. The distribution systems are generally, supplied by gravity from service reservoir into the customer and pumping system from the borehole direct into service reservoir by the submersible pump.

4.5.14. 3.2Lay Out Of Distribution System

For Tilili town the layouts of distribution system adopt loop system methods are available. The pipes have, where ever feasible, been laid along the routes of master plan roads or existing roads

And from methods of supplying water continuous system are selected for each user. Since water is supplied to the consumers for sixteen hours a day.

4.5.14.3.3 Nodal Demand

The average day, commercial and institutional demand, including non-revenue water, were distributed to the nodes of the network model according to the land use plan as per the following procedures: From the land use plan and site the spatial distribution of the existing and proposed infrastructure were identified;

In order to simplify the analysis, the land is reclassified into two major categories domestic and commercial and institutional Using a digitized map the area supplied from each node is delineated, measure and tabulated for each category;

The demand area ratio for each category is computed

Nodal demand is computed by using the above tow equation that I have mentioned equation (11&12) those are computed multiplied each junction demand by the population they are used from the junction with nodal demand the area at each node with the demand area ratio for the specific consumer group was $ND = \frac{PDxPi}{Pt}$ and $Qi = \frac{QTxAi}{At}$……………………Eqn(4.23)

4.5.14.3.4 Selection of Pipe Materials

Pipes must be strong, durable and cheap materials used for construction. Among different types of pipes cast-iron, wrought iron, GI steel, uPVC and cement lined are the most accessible and popular.

Ductile cast iron for main transmissions and uPVC pipes for distribution system were used. GISpipes are strong, durable, easy to joint, corrosion resistance; provide easy service connection anduPVC pipes are cheap and available in the market. Most of the town distribution system is uPVC and i was selecting the available pipes from the existing distribution systems.

4.5.14.3.5Design Pressure

According to Minster of water resource (MoWR)the pressure in the junction or node has from 10 - 70m H_2O if below or above develop as a result of pressure surge or water hammer in the pipe line in order to limit the maximum pressure in the pipe line and, thus, the cost of the pipes.

4.5.14.3.5.1Pipe diameter: - For the known design discharge the pipe diameter are assumed in such a way that the velocity of flow varies from 0.3 to 2m/s. Smaller velocity is assumed for pipes of smaller diameter and larger velocity is assumed for pipes of larger diameter.

4.5.14.3.6 Model Calibration and Validation

. A system-wide water balance approach with aim of separating real and apparent losses in order to establish appropriate water loss control strategies was applied during calibration of nodal demands in line with the IWA/AWWA water balance methodology. This calibration approach has been applied by various researchers (Almandoz et al. 2005; Cheung and Girol 2009; Nicolini et al. 2011).

Total hourly demands in the DMA are tracked while maintaining the overall system demand.

The model was calibrated under steady-state and EPS conditions using flow and pressure data.

Having estimated leakage rates, the next step is model calibration. The aim of calibration is to minimize the differences between the observed performance and model predictions. The model calibration procedure used in this study was the iterative trial-and-error approach (Walski 1983). Although manual calibration is tedious and has limitations on the number of parameters that can be handled effectively (Abe and Cheung 2009; Wu et al. 2011), it was preferred to advanced calibration tools based on optimization techniques due to its simplicity and the required level of accuracy was deemed to be less stringent for the purpose of the study. Generally, the use of advanced calibration tools is still limited in practice as indicated by recent studies (Savic et al. 2010; Speight and Khanal 2009), probably because they are computationally expensive. During the calibration process, pipe roughness coefficients pressure and nodal demands were adjusted as calibration parameters while pressure and flow data was collected for model calibration.

Although a large amount of “good” observation data is needed for estimating calibration parameters with sufficient confidence (Walski 2000), in this study data collection was limited to twelve junctions observed data was taken for pressure calibration. In such circumstances, trade-offs between sample design costs, level of model accuracy and real world constraints is necessary (Kapelan et al. 2003; Speight and Khanal 2009). The calibration procedure applied in this study is shown in Figure 6.5.However model is a generalized, simplified representation of a complex system or task. In addition to this models used to understand a process and effectively make decisions. Models are essential in that they are used to:-develop insights and understand the governing principles and fundamental mechanisms of a system, determine the ‘what if?’ questions and predict outcome of a system, assess impacts of a system. But it may be garbage in garbage out so it should be calibrate and validate with the standard and actual or observed data.

A calibrated model can also be used as part of an “outflow” detection system. Model simulation results from a specific flow control operation can be compared to actual system results at particular nodes in the system. Significant differences between computed and measured values may indicate an outflow (such as an open valve, leak, or pipeline break) that can be rapidly located, evaluated, and corrected. The relation between calibration and sampling location selection is a typical two face of one coin relationship. To calibrate a model, there should be measured or field test data; that is, sampling locations need to be defined. In addition to this to judge the success of calibration, one would like to calculate sensitivities for all potential measurement locations with respect to all possible calibration parameters.

The following key criteria have been considered in the selection of the sample nodes. 1Seletenizing the model by using WATER CAD6.5and Determine the estimated of model parameters of the models output(pressure and velocity)next to determination of those parameters field measurement was takes place at last model calibration and validation was performed by using Hazen wilians parameters and by considered higher and lower demand patterns and topographic condition of the town that means by considering different ground elevations were taken to evaluate the records of the pressure and meter readings (consumption) somehow in detail. 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).Thirteen representative sample measurements (twelve data sets observed data and twelve 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 pipes

5.0 RESULTS AND DISCUSSION

5.1Eixsting Water Supply Coverage and loss analysis

The main objective of the research was evaluate the water supply coverage and loses by assessing the distribution system in the town and to recommend mitigation measure for the above causes to increasing the water supply coverage and to reduce NRW of the town.

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

The average water supply coverage of the town was evaluated based on the daily per capital consumption and level of connection using the population data of the town.

Generally, in the old settlement areas of the town low level of connections per family and per capita water consumption was observed. Hence, it is concluded that the low financial capacity of the inhabitants, the topographic nature (higher elevation) of the areas and the available traditional pond as the main reasons for the low coverage in the areas

Three approaches were used to compare the loss among the systems:-

(i) The UFW expressed as a percentage.

(ii) Loss per-length of pipes and.

(iii) Loss per connection.

(iv) ILI

5.1.1 Existing Domestic Water Supply Coverage Analysis

The water supply coverage of the Town has been evaluated based on the average per capital consumption and level of connection per family. The average per capital consumption has been derived from the yearly consumption of the Town that has been aggregated from the individual domestic water meters. Beside to the average per capital water consumption, the distribution number of domestic's connection per family has been also evaluated Number of population as forecasted to the year 2015 has been used to evaluate the average per capital consumption.

5.1.2 Coverage Based On Per Capita Demand

The volume of water consumed for domestic purpose has been aggregated to the Town to analyze the distribution of the water coverage of the town .Evaluating the domestic water supply coverage using volume of consumption may not allow realizing the distribution comparison among the standards. To estimate the per capita demand of the town the annual consumption should be converted to average daily per capita demand using the total population number. Average daily per-capita demand was estimated is listed below table:-5.1by using the above equation (4.9)

Per-capita demand@2015 = $\frac{57222m3/yr}{8575} \approx 8.2m^3$/year

Per-capita demand = $\frac{57222(m3)*1000l/_{m3}}{8575*365} \approx$ 18.29(l/d)

Table5.1Coverage based on Yearly and daily per capita demand of the Town

year	Total Population	Total Domestic user	Total Customer	Total Production(m^3)	Billed (m^3)	Per-capita consumption(m3/Yr)	Consumption (l/c/d)
2015	17518	8575	1715	91231	57222	6.67	18.28

5.1.3 Coverage Based On Level of Connection

Access to water supply may be evaluated using the amount of water consumed and the level of connection. For evaluating the amount of water consumption, the annual water consumption is converted to average daily per capita consumption using the population data of the Town. The number of domestic connections per family has been also used for analyzing the level of the connection as elaborated below the table by using equation (4.9)

Number of connections in the town as end of year 2015 is as presented in Table 5-2.Tilili has a total of 1715 active connections as of December 2015. The proportions per type of customers are: 841 Domestic connections (49%), 386 Public fountains (22.5%) and 876 Nondomestic connections (5%).1715+876 =1715

Table5.2 customer Distribution by Mode Of Service

Year	2015		
User	No connection	%age	Users
HTU	21	1.22%	214
YTU (Private)	343	20.00%	3504
YTU (Neighboring)	91	5.30%	929
PTU (Urban)	386	22.50%	3942
Total (Urban)	841	49.0%	8588

Source taken from CSA 2007 the total projected population from the census data

The percentage population of Tilili that has access to safe water from the total population has been estimated by using total town active connection with relative to the total population of the town it was 49%. The remaining people use traditional and often unprotected sources

$$\frac{Total\ Active\ connection * Average\ Famly\ Size}{Total\ Population}$$

Coverage accessed safe water supply at 2015 = $\frac{1715*5}{17518}$ =48.95%≈49%

Table5.3 Domestic water supply coverage of the Town

year	Total Population	Average Family Size	Total Domestic user	Total Customer	Total Production(m3/year)	Billed (m3/year)	Coverage (%)
2015	17518	5.	8575	1715	91231	57222	48.95

5.3 Water Demand Projection

In this section deals with the estimation of the present and future water requirements for Tilili town using the prevailing modes of services and commonly known demand categories, which are relevant to urban centers and compare with the current water coverage of the town by forecasting the population in CSA method.

Water demand adjustment factors as well as water demand growth rates, which are presumed to be essential in defining phased design capacities of the major physical unities of the proposed water supply schemes are also discussed and estimated from different sources & data.

5.3.1 Population Projection.

The population projection growth rate is based on 1994 Population Household Census of CSA and the base year population is taken from CSA 2007 the total projected population from the census data at 2015was taken as the base population17518 .I used interpolation to obtain the growth rate profile for the year in between 2016 and 2027 and extrapolation for year in between 2030 and 2037 was projected until the end of the design period is listed below table-6.1

Table-5. 4 The projected population for the corresponding years

Year	Unit	2016	2017	2022	2027	2032	2037
1. Population		18251	19811	24148	28795	34233	41190
Population growth rate (town)		4.10%	4.10%	4.00%	3.80%	3.80%	3.70%

5.3.2Domestic Water Demand Projection

Domestic water demand is the amount of water needed for drinking, food preparation, washing, cleaning, bathing and other miscellaneous domestic purposes. The amount of water used for domestic purposes greatly depends on the lifestyle, living standard, and climate, mode of service and affordability of the users.

In projecting the domestic water demand the following procedures were followed: Determining population percentage distribution by mode of service and corresponding consumer category and its future projection. Establishment of per capita water demand by purpose for each mode of service or customer category; projected consumption by mode of service or customer category;

i. Modes of Services and Customer Category

According to the CSA 2007 statistical document and AAWSA records, there are five major modes of services for domestic water consumers that correspond to five costumer categories. These are:

Table5.5 Daily domestic demand by connection

		Year					
Daily consumption	Unit	2016	2017	2022	2027	2032	2037
HTU	m3/d	99.3	118.9	172.4	233	333.6	430
YTU	m3/d	197	241.5	355.7	503.5	699.6	1016.4
NTU	m3/d	171.7	210.5	307.9	403	522	700
PTU (town)	m3.d	203.5	218.4	301	395	491.6	626.5

Table 5.6 Summary total domestic demand

climate factor		1	1	1	1	1	1
socio economic factor		1	1	1	1	1	1
Total domestic demand(TDD)	m3/d	672	789	1136	1534.5	2046.8	2723

Table 5.7 Intuitional and Commercial Demand

year	2016	2017	2022	2027	2032	2037
Total l/day	134439	142145	168113	196034	226008	261894
Total l/sec	2	2	2	2	3	3
m3/day	134	142	168	196	226	262

Table 5.8Industrial Demand

Year	2016	2017	2022	2027	2032	2037
Total(l/day)	20,800	21,850	26,300	29,350	32,100	34,850
Total(l/sec)	0.24	0.25	0.30	0.34	0.37	0.40

Table 5.9 Domestic Non Domestic Demand (DD+C&IND+ID)

		2016	2017	2022	2027	2032	2037
TDD	m3/d	671.5	789.3	1136.8	1534.5	2046.8	2773
C&IND	m3/d	134	142	168	196	226	262
ID	m3/d	20.8	21.85	26.3	29.35	32.1	34.85
TDD+C&ID+ID		826.3	953.15	1331.1	1759.85	2304.9	3069.85

Therefore, in this case Maximum day demand and peak hour demand of Tilil town is summarized below the table depending on the size of population.

Table 5.10Maximum day demand and peak hour demand

Year		2016	2017	2022	2027	2032	2037
ADD	m3/d	971	1096	1509	1771	2310	3035
826.3	953.15	1331.1	1759.85	2304.9	3069.85	1.3	1.3
MDD	m3/d	m3/d	1261.79	1425.13	1961.27	2302.39	3003.03
PHF		2	2	2	2	2	2
PHD	m3/hr	80.9	91.4	125.7	147.6	192.5	252.9

5.4 Non-Revenue Water analysis

The gross non-revenue water as obtained from the water production and billed volume is averaged to 35.36%for the last eight years. The general trend indicates that NRW is increasing since 2015 and stabilized at 37% for the last two years.

5.4.1Total water loss expressed as Percentage (%)

Water loss expressed as a percentage can be an appropriate means to show the extent of the los within a given environment, but it may not be a good indicator for comparing the losses from one area to another but to identify loss by different causes firs it should be estimated the total water los of the town by using the total yaerly water production and total water consumption of the town . The total annual water produced and distributed to the system within the year 2015 had been 191231m^3/year from the total annual distributed water the consumption of year was 57222m^3/year and the total loss estimated was 34009m^3 /year (37.2%).

191231-57222=34009m^3/year $\longrightarrow$ $\frac{34009m3/year}{191231m3/year}$ =37.2%

5.4.2 Existing Tariff Rate

The current water tariff was set in response to the Ethiopian Water Resources Policy issued in 1999. The rate of the Water Supply and Sewerage Service implemented is an inverted block tariff rate that increases progressively for large volume of water used. That is, large volume of water user pays progressively more and more.The low income people using low volume of water (up to 5 m^3) pay less than 3 ETB per cubic meter of water. Other people who cannot afford house or yard connection pay at a flat tariff rate for any volume of water consumed 2.5 ETB / m^3). The highest tariff level is 4 ETB /m^3 for people consuming over 41 m^3 of water. In conclusion the existing tariff rate of Tilili WSS service charge is one of the lowest rates in the country and does not reflect the full cost recovery. By taking the 5 birr per cubic meter the total yearly Town nonrevenue water is 2.5*34009m3= 85022.5birr or 4251.125 USD American dollar.

Table 5.13 Non-Revenue Water (2008-2015) for Tilili Town

Year	Population	Customer number	Production(m^3/year)	Billed(m^3/year)	Yearly loss(m3)	Losses (%)
2008	13200	753	37281	23802	13479	36.1
2009	13808	797	41083	25529	15554	37.8
20010	14443	822	49610	30528	19082	38.5
2011	15108	926	67477	43594	23883	35.3
2012	15646	1200	73988	52677	21311	29
2013	16334	1300	80985	53289	27696	34.2
2014	17051	1461	84428	54776	29652	35.1
2015	17518	1715	91231	57222	34009	37.2

As shown in the figure 5.1 below the coefficient of determination (r^2) is 0.917, indicates that the regression model accounts for 92% of water loss is explained by production amount of water

The correlated graph also indicates that the direct relationship between water productions with water loss trend over years in the town .Water loss is increases dramatically in parallel when the water production is increases.

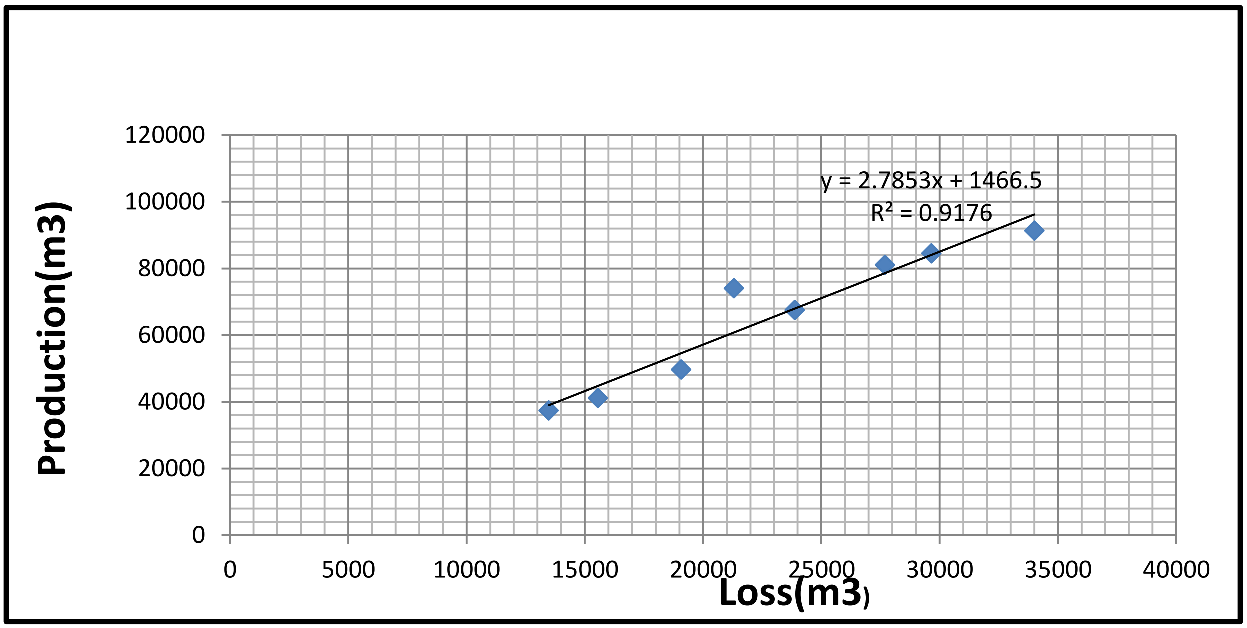


Figure 5.1correlated between yearly water production verses water loss of town

The figure 5.2 below indicated that the trend of water loss has a direct relationship with water consumption and water production of the town over a period of years. More water is produced and distributed to the system in parallel the loss is increased with it. So there should be an appropriate pressure management system is apply.

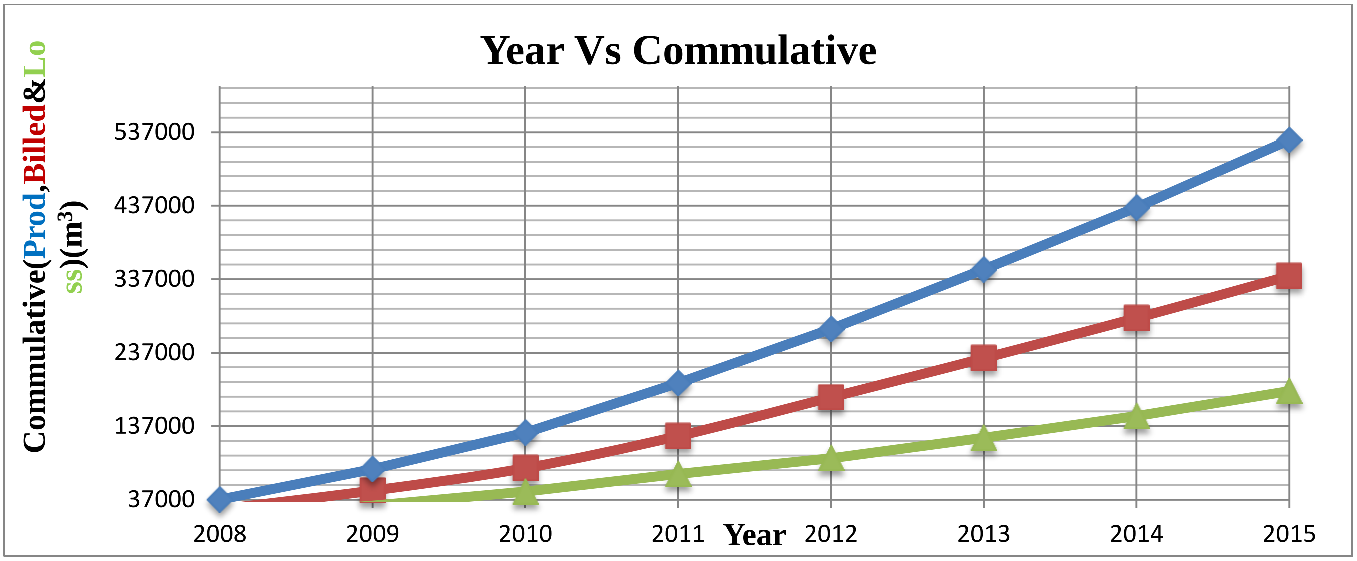


Figure-5.2 Yearly Total Water Los Distribution of Each Successive Eight Year

Even though the water loss of the town indicated below the figure 5.3 is increased for some year and decrees for some another year on average the general trend indicate that the amount of los was increased year to year in the town .it shows that problem of network management.

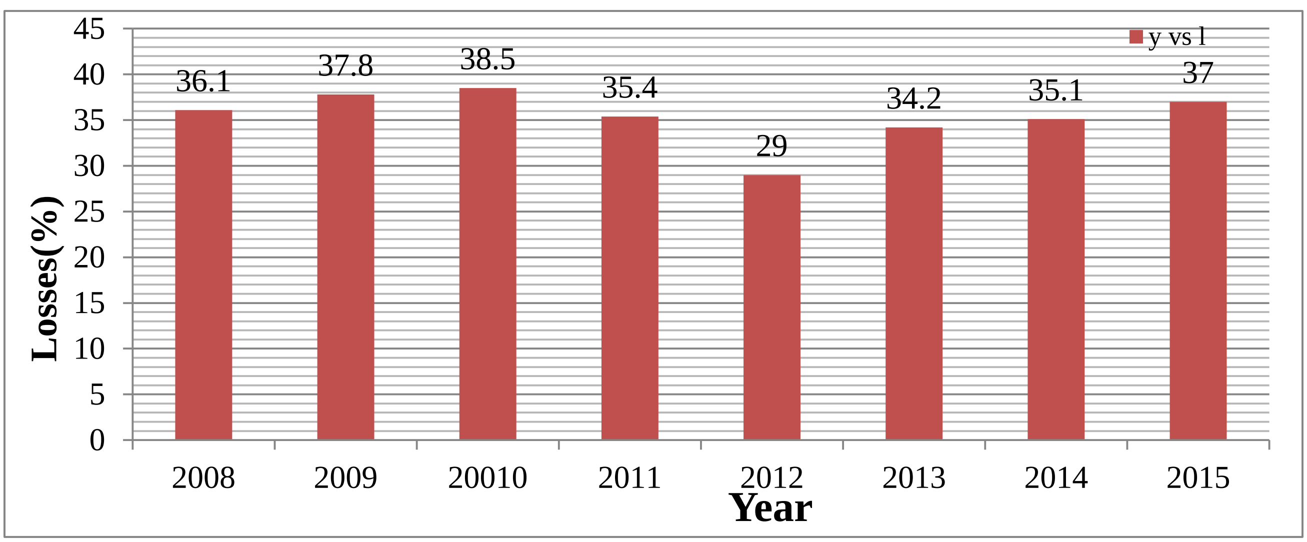


Figure 5.3 The town water loss trend over each successive eight year

5.4.3Total Water loss expressed as per number of connections

Water loss per connection (litters/connection/day) was a recommended performance indicator for areas with dense connections. In Tilili town, evaluating and comparing loss of the distribution using an indicator of loss per connection was more appropriate than the other indicators or approaches.

Comparison of water loss between different areas is recommended to be done using the water loss per service connection per day. The total number of connection in the town at 2015was 1715and the total annual water loss was 34009m^{3}therefore lost by litter per connection per day$\frac{34009x1000l}{1715x365}$ = 54.33l/d was estimated.

5.4.4 Water loss expressed as per length of main pipes

The second types of water los performance indicator were evaluating the total water loss of the specified year with the total length of main and sub-main pipes of the distribution system of the town. According to Tilili town there is around18.2kilometers.According to Tilili town water service bureau which have a diameter size of 50mm and above from those around5.5kilometers was distribution line and the remaining was pressure mains. But there are some pipes that their diameter not clearly identified; all the pipes greater or equal to 50 mm in diameter have been used to evaluate the water loss per length of pipes at2015 =$\frac{34009x1000l}{18.5x365km}$ =5.2m3/km/d=5120l/km/d

5.4.5 Estimating Real Losses

In the top down approach followed in this Report, real (physical) water losses are obtained as the difference between other water balance components, i.e. as the difference between System Input Volume on one hand and Authorized Consumption and Apparent losses on the other.

5.4.5.1 Calculating Infrastructure Leakage Index

The Infrastructure Leakage Index (ILI) being the ratio of Current Annual Real Losses to Unavoidable Annual Real Losses (UARL) is showing to be a most useful and practical performance indicator. It is being used for rapidly assessing efficiency in management of Real Losses, setting targets, and prioritising remedial activities (using a ‘twin track’ approach which considers pressure management in parallel with more traditional forms of leakage management).

The ILI approach was developed and tested over a period of several years by the IWA Water Losses Task Force; it was first published in December 1999 in “AQUA”, and included in the IWA ‘Best Practice’ Performance Indicators Manual (July 2000). The paper addresses some queries raised by practitioners applying the ILI approach. Some recent international applications are presented, including Utilities with exceptionally good performance (ILI less than 1.5). The introduction of 95% confidence limits into calculations of Water Losses and associated Performance Indicators is also discussed and recommended.

Table 5.14 Diameter type and Size of the pipe of the distribution

Item No.	Pipe Material	Diameter(mm)	Total Length (m)	Remarks
1	GI	25		
2	GI	30		
3	GI	35	757.5	Distribution Line
4	GI	25		
5	HDPE	50	4019	Distribution Line
6	HDPE	80	720	Distribution main
7	HDPE	110	7,980.00	main
8	PVC	150	2544.5	Distribution main
9	PVC	200	2,249.50	Pressure main
10	DCI	150	Collectors & Transmit ion 756&1631.5(meter)	Collector & transmit ion
11	DCI	200		
12	DCI	250		

Source-(TTWSB)

5.4.5.2Loss due to pressure and leakage

5.4.5.3Leaks in Water Distribution Systems

From one municipality to another and even from one location to another, the causes of leaks will vary depending on the nature of the soil, the quality of construction, the materials used, the pressure levels and the utilities operating and maintenance practice (AWWA, 1987) .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-joint 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. 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).

Town yearly total water loss is computed by using water production &consumption in the table below. 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 by using the above equation (4.6)

Loss from Distribution Mains Q = $30.39375 * A * \sqrt{P}$

Table5.15 Loss from the collectors and transition Mains

Item No.	Pipe Material	D(mm)	D(inch)	A(inch$^{2)}$	P	$\sqrt{P}$	Q(gal/min	Q(gal/day	Q(gal/year	Q(m3/yr
1	DCI	150	5.905512	34.87	60.43	7.774	8240.0	7910371.9	21672.3	82.1
2	DCI	200	7.874016	62.	60.4	7.774	14648.8	14062883.	38528.4	146.
3	DCI	250	9.84252	96.87	60.4	7.77	22888.8	21973255.	60200.7	228.2
		Total								456.3

Table5.16 Loss from the Distribution Mains

Item No.	Pipe Material	D(mm)	D(inch)	A(inch$^{2)}$	P	√P(inch)	Q(gal/min	Q(gal/day	Q(gal/year	Q(m3/yr
1	HDPE	100	3.9370	12.2	60.4	7.77	2881.1	2765899	7577.8	28.7198
2	HDPE	110	4.3307	14.7	60.4	7.77	3471.5	3332682	9130.6	34.60510
3	PVC	150	5.9055	27.3	60.4	7.77	6447.2	6189266.	16957	64.26663
4	PVC	200	7.8740	48.7	60.	7.77	11501	11040926	30249.	114.644
	Total									242.24

5.4.5.4Distribution 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 of0.008 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, 2015 about 4% of the total real leak in the distribution system is lost through distribution mains and its frequency is 6per 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 mentioned at equation 8and it is summarized below table

Table 5.17 Loss from the Distribution Sub- Mains

Item No.	Pipe Material	D(mm)	D(inch)	A(inch$^{2)}$	P	√P(inch)	Q(gal/min	Q(gal/day	Q(gal/year	Q(m3/yr
3	HDPE	25	0.984251	0.96875194	60.43	7.77	229	219629	601.7	2.281
4	HDPE	50	1.96850	3.87500	60.43	7.77	915	878515	2406.9	9.1221
5	PVC	80	3.149606	9.92001984	60.43	7.77	2343	2248998	6162	23.3526
6	PVC	85	3.34645	11.198	60.	7.77	264	2538908	6956	26.363
Total										61.118

5.4.5.5 Loss from Collectors and Transmission Main.

Loss from collectors and transmition is higher than other components due to higher pressure and diameter size of the pipes are designed to convey raw water from each borehole to a junction near the well field. Expecting 16 hours of pumping operation of the within a day flies from different component of the distribution system is depending on the pumping hour and pressure it is summarized below the following table**.**

Table -5.18 Loss from the connection.

Item No.	Pipe Material	D(mm)	D(inch)	A(inch2)	P	$\sqrt{P}$	Q(gal/min	Q(gal/day	Q(m3/yr
1	DCI	20	0.7874	0.62	60.4	7.773674	146.5	140628	1.46023
2	DCI	25	0.984	0.97	60.4	7.7736	228.9	219732.	2.28161
						Total			3.74183

The Infrastructure Leakage Index (ILI) – being the ratio of Current Annual Real Losses to Unavoidable Annual Real

Losses (UARL) – is proving to be a most useful and practical performance indicator. It is being used for rapidly assessing efficiency in management of Real Losses, setting targets, and prioritising remedial activities (using a ‘twin track’ approach which considers pressure management in parallel with more traditional forms of leakage management).

The ILI approach was developed and tested over a period of several years by the IWA Water Losses Task Force; it was first published in December 1999 in “AQUA”, and included in the IWA ‘Best Practice’ Performance Indicators Manual (July 2000).

UARL (litters/day) = (18 x Lm + 0.8 x N_C+ 25 x Lp) x P

UARL=(18X17+0.8X1715+25X0) X42. =70476Liter/Day = 25723m^3/Year

5.4.5.6 Billed authorized consumption

Billed metered consumption is the total Billed Metered Consumption of water from the distribution system of the town for year 2015 was57222m3/year.

5.4.5.7Billed unmetered consumption

This category of water consumption includes water lost as a result of pipes damaged by different factors like contractors or other parties. The responsible party is billed by Tilili Town for the volume of water lost, which is estimated based on the diameter of the damaged pipe and the duration of the incident. This category also includes water distributed by the Town to consumers by water tankers (trucks). The consumers are billed for this water but they are not metered150m3/year. Available data are summarised below table 7.22

5.4.5.8Unbilled Metered Consumption

This category of consumption includes water used by Tilili Town facilities, where water is metered but bills are not distributed. According to data provided by Town, Unbilled Metered consumption includes water used by the water supply service workers for reservoir cleaning main pipe flushing , firefighting and their domestic use in the in the administration office treatment plant and wells. This volume of water is considered non- revenue water as the water service office, 2015 report it is supplied free of charge though it is measured and it has volume of 250 m3/year.

5.4.5.9Calculating 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

$$\text{ILI}=\frac{CARL}{UARL} \quad =\frac{37009m3/year}{24723m3/year} \cong 2$$

This indicates that current annual real los is twice of unavoidable real loss of the town

Where, ILI= infrastructure leakage index (ILI)

NRW= Non -Revenue water (m3/year)

UARL =unavoidable annual real loss (m3/year)

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

Table5.19 International Data Set of ILIS

Country	Cyprus	New zealand	Canada	Uk	Carbibean	Wales	Tilili	England	Germany	France	D/markos	Japan
ILI	1	1.2	1.3	1.3	1.3	1.4	1.5	1.5	2.4	2.5	3.6	4.1

Ireland	Hungary	S/Africa	Itali	Greece	carebiab	Addis Abeba	Malesia	Poland	Bulgaria	Jordan	Sri Lanka
5	5.7	5.7	7.1	9.7	10.5	14.5	20.5	20.6	24.4	27	35.9

Source:-The IWA Water Losses Task Force (Lambert et al, 1999)

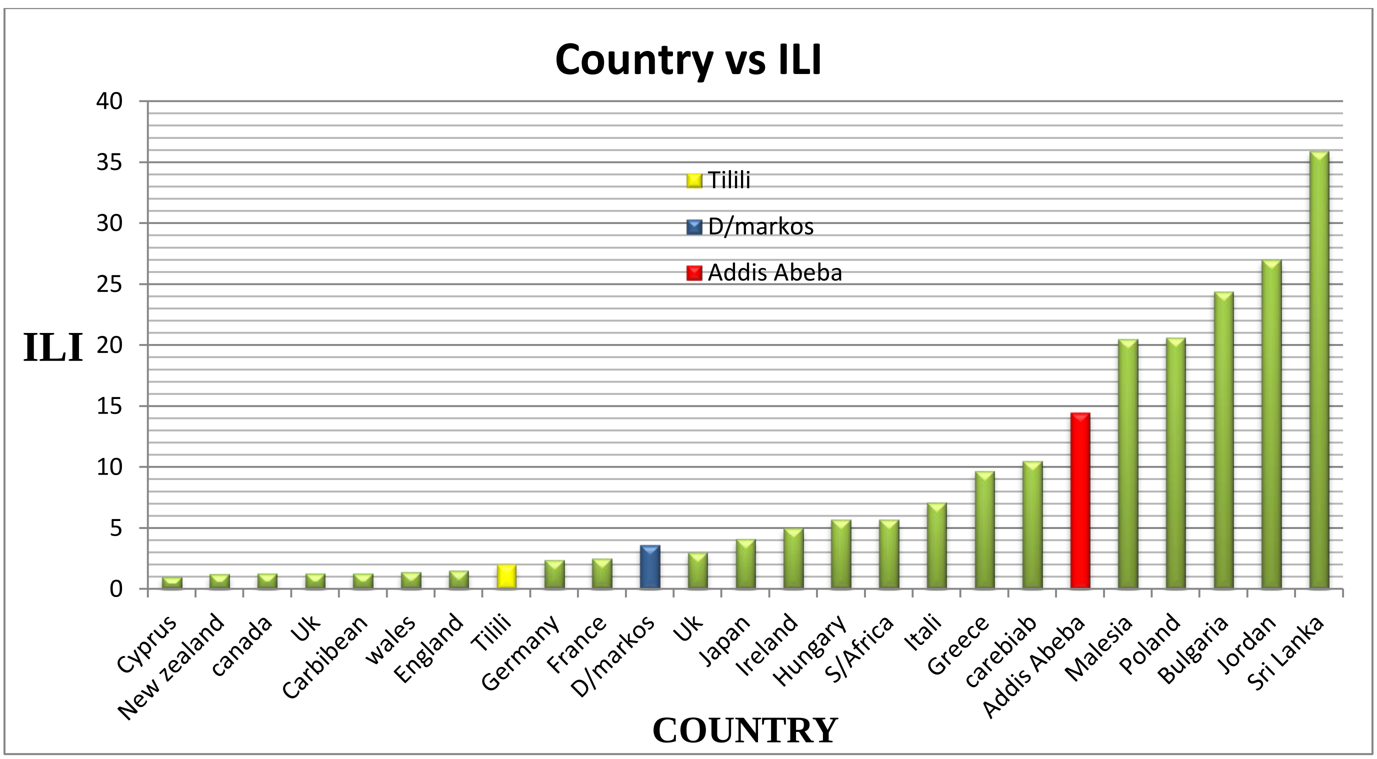


Figure5.4 International Data Set of ILIs

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. But from the above figure ILI value is 2 so it indicates that leakage management is not that much satisfactory in the town

5.4.5.10The apparent losses consist of two main components

- Unauthorized Consumption and
- Metering Inaccuracies.

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.

5.4.5.11 Unauthorized consumption

Unauthorized Consumption is the result of water theft through illegal connections and interference with the water meters. The most common types of water theft are meter reversal and connections before the meter.

5.4.5.12Customers Meter

The different metrological class's flow rates testing were indicated that meters with equal capacity and Manufacturer Company have different recording or measuring capacity during minimum midium and high flow rates depending on their age category. It is important to note that since it was testing meters by dividing the customer meter based on their age old in service meters, by putting master meter as a bench mark. The number of samples subjected to flow tests was limited due to the long flow times required to measure the extremely small flow rates and minimize uncertainty in the measurements (Arregui et al. 2006) .During customer meter testing the aged water meter were measuring low volume of water relatively new customer meter

$$\varepsilon = \frac{(Vm - Va)x100}{Va}$$

Table5.20Annual water loss from meters when the meter at different pressure flows

	Total Sub –meter Flow(l/d)							
	Meter type	Master meter reading(l/d)	Low flow	Medium flow	High flow	Difference		
5-10	15	200	120	160	185	80	40	15
	15	200	110	155	180	90	45	20
	15	200	100	150	175	100	50	25
	15	200	90	130	170	110	70	30
	15	200	80	117	165	120	83	35
10-15	15	200	75	110	160	125	90	40
	15	200	72	100	150	128	100	50
	15	200	71	95	142	129	105	58
	15	200	70	91	138	130	109	62
	15	200	68	90	135	132	110	65
15-20	15	200	67	86	130	133	114	70
	15	200	65	80	125	135	120	75
	15	200	63	75	120	137	125	80
	15	200	61	71	115	139	129	85
	15	200	60	70	110	140	130	90
	15	200	56	65	100	144	135	100
	15	200	54	60	90	146	140	110
	15	200	50	60	80	150	140	120
	15	200	40	50	80	160	150	120
	15	200	30	40	50	170	160	150

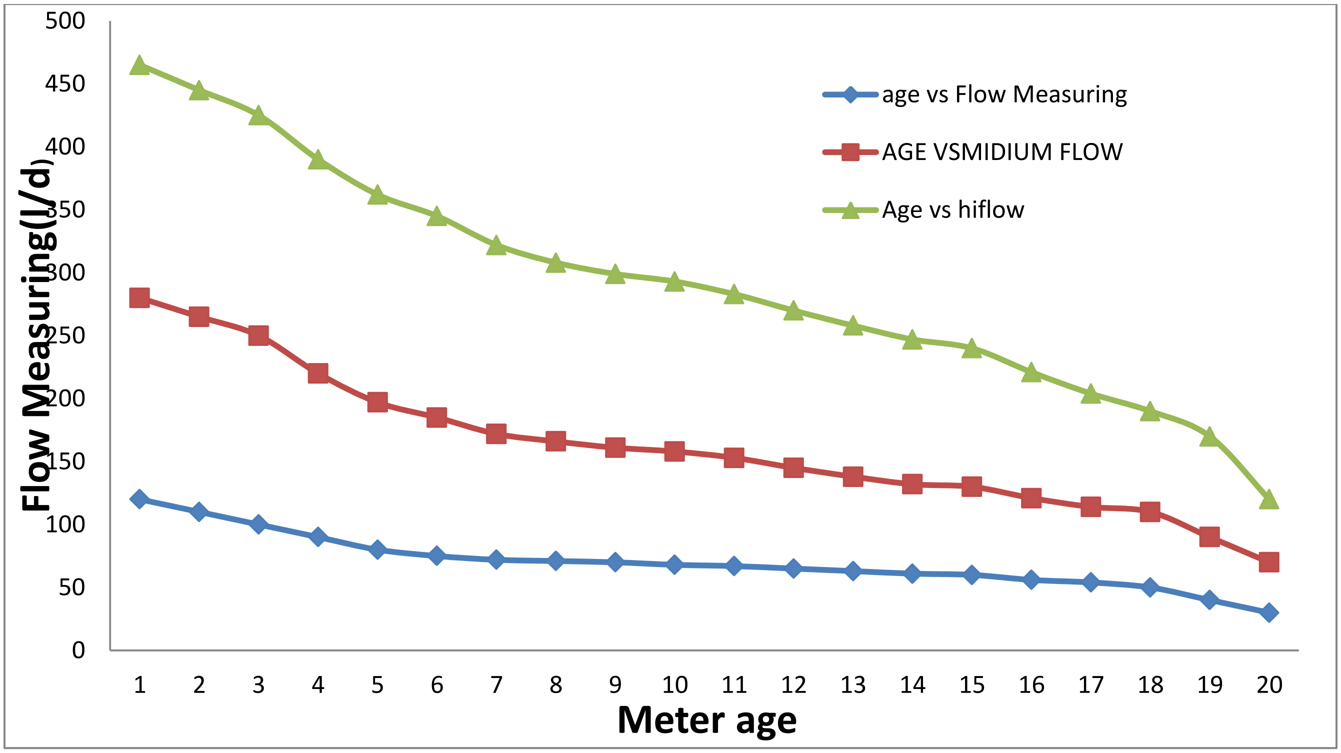


Figure 5.5 the water meter installed in different time category recording of consumption with different pressure or flow

5.4.5.13Metering inaccuracies

Metering inaccuracies include customer water meter inaccuracies (i.e. customer water Meter under registration and over registration) as well as errors in the meter reading data transfer and billing process. In the Town over registration was one of the critical problems of customer what were they paid over due the water service bureau servant irresponsibility just they are related the current consumption with the previous billed data without reading the water meter and also there were under registration problems. According to the water service office of Tilili Town, 2015 report metering inaccuracies was 560m3/year

5.4.5.14 Illegal connections

Although the only legal use of fire hydrants is for firefighting, but some peoples use illegally to fill tankers (normally at night) or to provide water supply to construction sites. The utility staff can detect these flows, often high volume over a short period of time; through appropriate flow measurements at DMA meters in Tilili town, there were some customers used illegal connection for different purposes but according to the town water service bureau information the most common was during construction new house for mudding purpose to watering the retaining wall concrete and mudding purpose. 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, 2015report unauthorized consumption amounts 450m3/year.

Table5.21 IWA 'Best Practice' Water Balance

<table>
<tr><td rowspan="7">System Input Volume of water 91631m3/year</td><td rowspan="4">Authorized Consumption 57622m3/year</td><td rowspan="2">Billed Authorized Consumption 57472</td><td>Billed metered consumption 57222</td><td rowspan="3">Revenue water 57622m3/year</td></tr>
<tr><td>Billed unmetered consumption150m3/year</td></tr>
<tr><td rowspan="2">Unbilled Authorized Consumption 150m3/year</td><td>Unbilled metered consumption250m3/year</td></tr>
<tr><td>Unbilled unmetered Consumption</td><td rowspan="4">Non-Revenue Water34009m3= 37.2%</td></tr>
<tr><td rowspan="3">Water losses 34009m3/year=37.2%</td><td rowspan="2">Apparent Losses 1010m3/year</td><td>Unauthorized use 450m3/year</td></tr>
<tr><td>Metering inaccuracies560m3/year</td></tr>
<tr><td colspan="2">Real Losses32999m3/year</td></tr>
</table>

5.5 Causes of water loses (NRW)

The major challenges(causes) of water loss in the town include meter reading and data handling errors ,ageing infrastructure, excessive pressure, inadequate resources, poor governance issues, inadequate asset management, poorly designed WDSS, insufficient reliable data for WDS performance evaluation, intermittent supply and kleptomania for water.

5.5.1 Leakage

The occurrence of leakage in the town is through physical losses from leaking and broken pipes, which are caused by poor operations and maintenance, the lack of active leakage control, and poor quality of underground assets. Some of the causes are natural leakage, land subsidence and Pipe deterioration and Human factor include Vibration and load by traffic, Poor construction and damage due to construction by other enterprise

Leakages occurring from transmission and distribution mains are usually large events, sometimes catastrophic, causing damage to highway infrastructure and vehicles. The majority of such bursts are usually not very severe although they cause supply disruptions according to the town water service bureau.

5.5.2 Water meter and data handling error

The customer meter reading and data handling error is can thought to be the main cause of AL of the town. These errors can be caused by picking wrong meter readings (intentionally or accidentally). The typical method of data handling and billing requires a meter reader to visit each property and read the customer meter. The data is then recorded by hand on a form, taken back to the office, given to the billing department, and typed into the billing system. And also like all mechanical devices, water meters typically decline in accuracy with usage over time causing substantial revenue losses to the utility and gives rise to unequal billing. One of the big problem encouraging AL of the town is lack of periodic water meter replacement based on meter testing and carelessness of data collectors for minimizing revenue losses due to customer metering inaccuracies and data handling error Meter reading and data handling errors arise during the process of meter readings (gathered) manually or automatically), data transfer to the billing system and archiving of customer

5.5.3 Meter bypassing

Some customers try to reduce their water bills by using a meter by pass, which is an additional pipe installed before the water meter. This bypass pipe is often buried and very difficult to detect. This type of unauthorized consumption is usually committed by industrial and commercial premises, where only a small volume of the consumption goes through the meter and the rest through the bypass pipe but this is not practical problems of the town of Tilili.

5.5.4 Illegal use connection

Although the only legal use of fire hydrants is for firefighting, but some peoples use illegally to fill tankers (normally at night) or to provide water supply to construction sites. The utility staff can detect these flows, often high volume over a short period of time; through appropriate flow measurements at DMA meters in Tilili town some customer used illegal connection during construction new house for mudding purpose.

5.5.5 Production meter accuracy

The accuracy of production flow meters is critical to calculate system NRW. Generally, the number of production flow meters is relatively small, meaning that a greater proportion of the flow is measured by each meter.

5.6Management of Non-Revenue Water

5.6.1 Management methods of physical /Real Losses

The four pillars of a leak management strategy include:-

5.6.1.1Pressure management

- Pressure level and pressure cycling strongly influence burst frequency.
- To assess the suitability of pressure management in a particular system, utilities should first carry out a series of tasks, including:
- Identify customer types and control limitations through demand analysis
- Gather field measurements of flow and pressure (the latter usually at inlet, average zone point, and critical node points)
- Model potential benefit using specialized models
- Identify correct control valves and control devices
- Model correct control regimes to provide desired results
- Analyze the costs and benefits

5.6.1.2Speed and Quality Repairs

- Efficient organization and procedures from the initial alert through to the repair itself
- Availability of equipment and materials
- Sufficient funding
- Appropriate standards for materials and workmanship
- Committed management and staff
- Good quality of service connections are often the weakest leakage

5.6.1.3Active leakage control

Active leakage control (ALC) is vital to cost-effective and efficient leakage management. The concept of monitoring flows into zones, or district meter areas (DMAs), where bursts and leaks are unreported is now an internationally accepted and well-established technique to determine where leak location activities should be undertaken.

5.6.1.4Asset management

Good asset management is a necessity for long-term economic leakage management, and the objective is to tackle leaks in the most cost-effective way. This requires priority setting and decisions on whether to repair, replace, rehabilitate, or leave the assets as they are, while simultaneously implementing pressure management and improving the operation and maintenance programmed.

5.6.2Management methods of commercial losses /Apparent Losses

5.6.2.1Installing Meters Properly

Meters should be installed properly according to the manufacturer‘s specifications. For example, some meters require a specific straight length of pipe upstream and downstream of the meter

- Installing Meters Properly
- Monitoring Water Quality
- Monitoring Intermittent Water Supply
- Sizing Meters Properly
- Using the Appropriate Class and Type of Meter
- Maintaining and Replacing Meters Properly
- Customer Meter Servicing
- Addressing Meter Tampering

5.6.2.2 How to address unauthorized consumption

- Finding and Reducing Illegal Connections
- Preventing Illegal Connections
- Tackling Meter Bypassing -
- Preventing Illegal Use of Fire Hydrants
- Actively Checking the Customer Billing System
- Avoiding Corrupt Meter Readers

5.7Distribution System Modeling

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

5.7.1Hydraulics Modeling

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

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

5.7.2Data Assembly

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

5.7.3Nodal demand allocation

The allocation of base demand is critical for accurate modeling system. The base demands were obtained from the utility customer billing database and manually allocated by aggregating consumptions to the nearest nodes and based on number of connection from similar nodes. This was enabled by integrating GIS block map information with unique customer account references that are also used in the customer billing database. It was assumed that nodal consumption followed the same pattern as the diurnal flow profile measured at DMA inlet. A system-wide water balance approach with aim of separating real and apparent losses in order to establish appropriate water loss control strategies was applied during calibration of nodal demands in line with the IWA/AWWA water balance methodology.

5.7.4 Types of Distribution System Modelling (Simulations**)**

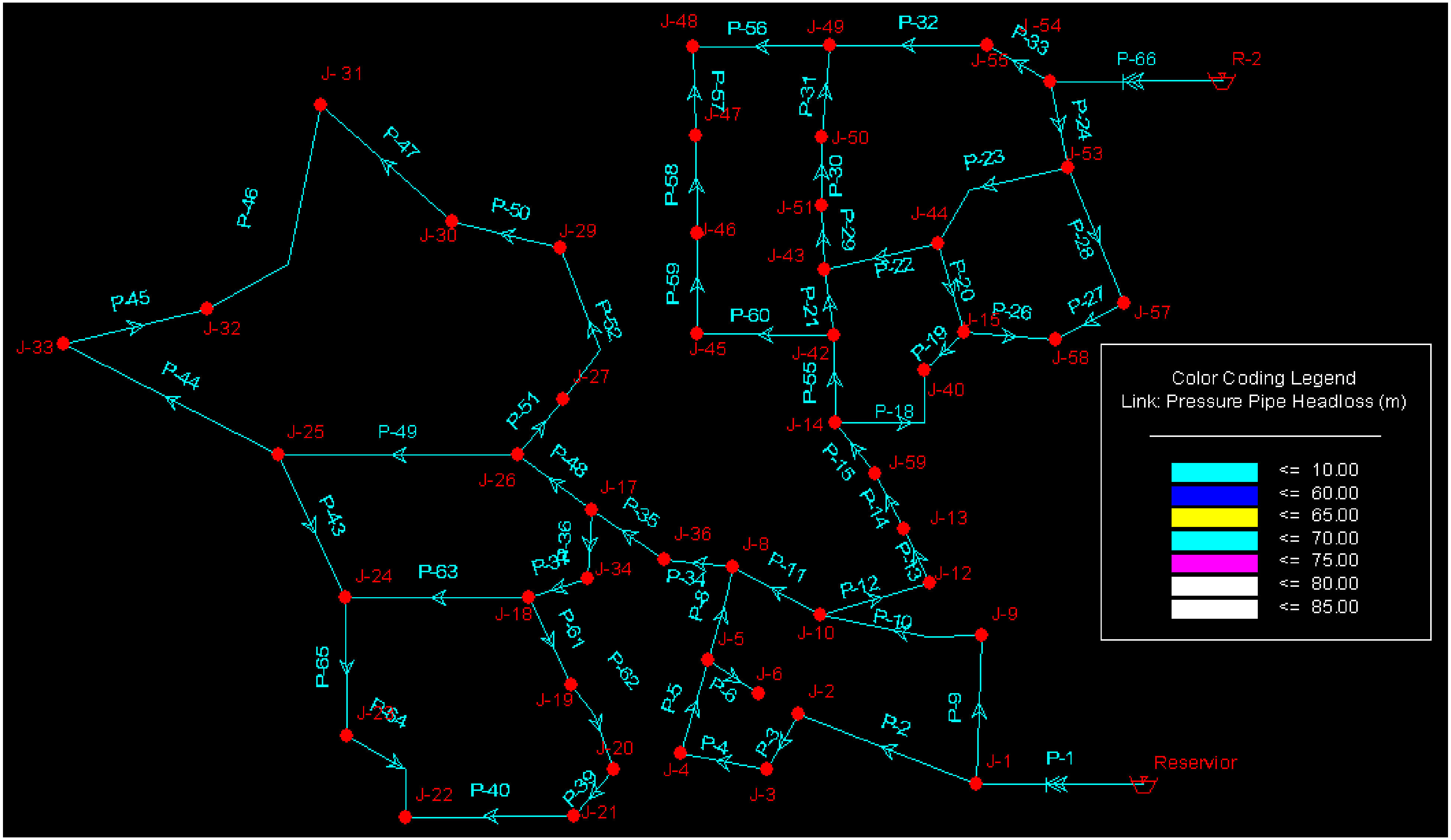
After the basic elements and the network topology are defined, further refinement of the model can be done depending on its intended purpose. There are various types of simulations that a model may perform, depending on what the modeler is trying to observe or predict. The two most basic types are

5.7.4.1 Steady-state simulation

Steady-state simulation 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.

5.7.4.2 Extended-period simulation (EPS)

Determines the quasi-dynamic behavior of a system over a period of time, computing the state of the system as a series of steady-state simulations in which hydraulic demands and boundary conditions do change with respect to time. The results provided by a steady-state analysis can be extremely useful for a wide range of applications in hydraulic modeling. There are many cases, however, for which assumptions of a steady-state simulation are not valid, or a simulation is required that allows the system to change over time. For example, to understand the effects of changing water usage over time, fill and drain cycles of tanks, or the response of pumps and valves to system changes, an extended-period simulation (EPS) is needed. It is important to note that there are many inputs required for an extended-period simulation. Due to the volume of data and the number of possible actions that a modeler can take during calibration, analysis, and design, it is highly recommended that a model be examined under steady-state situations prior to working with extended period simulations. Once satisfactory steady-state performance is achieved, it is much easier to proceed into EPSS can be accounted for and used to solve for flows.



Figur5.6 Extended simulation result

5.7.4.3 Pressure Location in the Distribution System

Pressure location in the distribution system at different junction assigned by color coding(pressure and velocity) of the network. The network of the system is shown in Figure 7.5 It is made of 100 pipes and 52 junctions. The different subsystems within the overall distribution system are shown.

WATER CAD can show pressures, demand, and water at different nodes as well as flows, velocities and head loss in pipes throughout the distribution network. Direction of flow is shown by arrows on Figure. The figure shows large flows (red arrows) going into the system which then re-distributes water to taps.

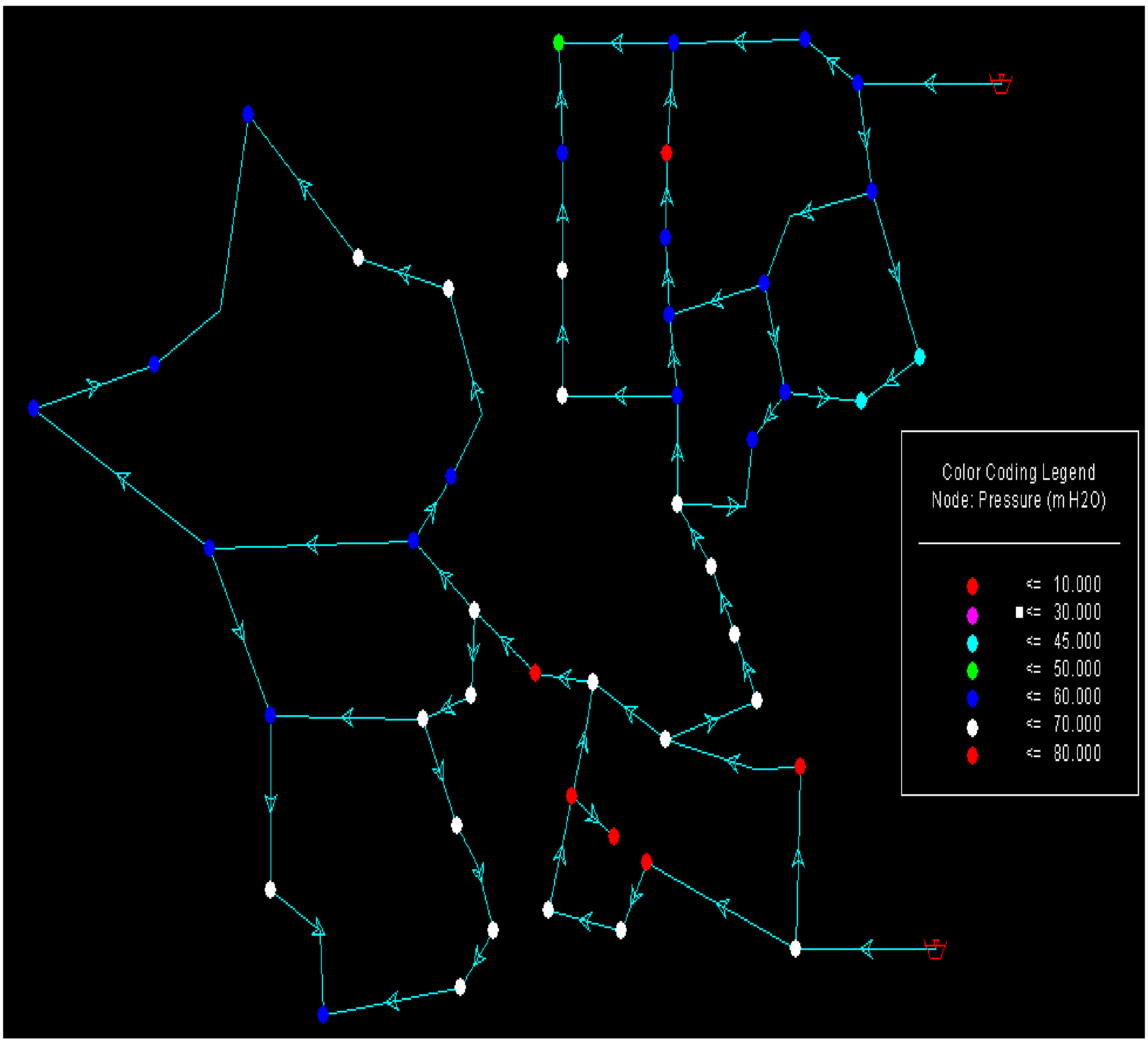


Figure5.7 -Location of pressure in the distribution system by diameter size

5.7.4.4 Pipe Diameter Location in the Distribution System

Pipes take flow as it moves from one junction node to another junction in a network. In the real world, single pipes are usually factory-made in lengths of around 10 to 20 feet, which are then collected in series as a pipeline. Real-world pipelines may also have various fittings, such as elbows, to handle abrupt changes in direction, or isolation valves to close off flow through a particular section of pipe. For modeling purposes, single segments of pipe and associated fittings can all be combined into a single pipe element. A model pipe must have the same characteristics (size, material, etc.) throughout its length.

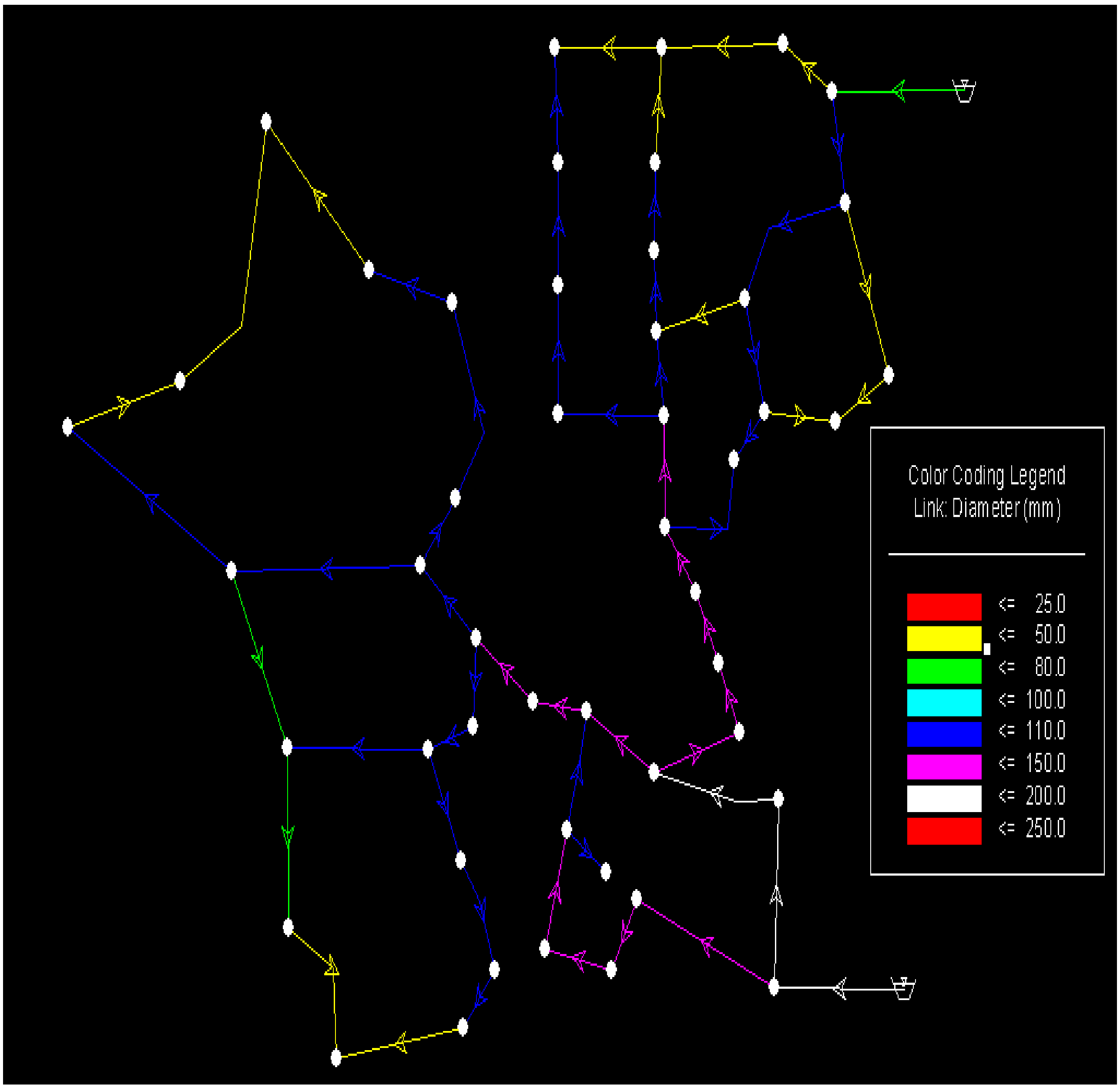


Figure 5.8-Location of pipes in the distribution system by diameter size

5.7.5 Model Calibration

The aim of calibration is to minimize the differences between the observed performance and model predictions. The model calibration procedure used in this study was the iterative trial-and-error approach (Walski 1983).

The credibility of a model is merely evident if a model result precisely reflects observed field values. This calibration approach has been applied by various researchers (Almandoz et al. 2005; Cheung and Girol 2009; Nicolini et al. 2011).Thus, to have a confidence on model result it needs to calibrate and validate a model. An effort to perform hydraulic and water pressure model calibration and validation for this case study is presented as follows

In general, if model peaks arrive at the wrong time, the wave speed needs to be adjusted. If model peaks have the wrong shape, the description of the control event (pump shutdown or valve closure) should be adjusted. If the transient dies off too quickly or slowly in the model, the friction losses need to be adjusted. If there are secondary peaks, important loops and diversions may need to be included in the model.

5.7.6 Hydraulic Model Calibration

Calibration is a procedure used to bring predicted and observed system parameters into closer agreement with one another. Flow problems are usually solved using hydraulic analysis of computer models. Given descriptions of the system and the event triggering the transient and information on the boundary conditions, the model can determine fluid velocity V (and flow Q) and pressure p (and head H).

Comparisons of computed results from a simulated result analysis program with results measured during laboratory experiments or field tests from actual systems have been used to validate transient analysis programs. These comparisons require that well-defined flow control operations be used to allow proper simulation and calibration of the computer model..

The differences between computer model simulated results and actual or measured system measurements are caused by several factors, some of which are highlighted in the following list:

• Precise determination of the celerity of the piping system is impossible. This is especially true for buried pipelines, which are influenced by bedding conditions and the compaction of the surrounding soil.

• Precise modeling of dynamic system elements (such as valves, pumps, and protection devices) is difficult because they are subject to deterioration with age and adjustments made during maintenance activities.

• Differences exist in the friction coefficients for a steady-state condition versus a transient condition. In a transient condition, flow direction changes, and changes in velocity gradients modify the shear stress in the pipeline. The friction coefficient used in the model should ideally account for the localized and transport accelerations.

• Prediction of the presence of free gases in the system liquid is sometimes impossible. These gases can significantly affect the celerity and propagation of waves. In addition, the occurrence of column separation and vapor cavity formation are difficult to accurately simulate.

The first two items can be eliminated as error factors by calibrating the model data and by carefully representing the operational characteristics of dynamic system elements.. .

The operational risks can be evaluated, flow control operations specified and optimized, and protection devices sized such that the extreme transient heads are controlled to acceptable limits for each particular system..

A calibrated model can also be used as part of an “outflow” detection system. Model simulation results from a specific flow control operation can be compared to actual system results at particular nodes in the system. Significant differences between computed and measured values may indicate an outflow (such as an open valve, leak, or pipeline break) that can be rapidly located, evaluated, and corrected**..**

However model is a generalized, simplified representation of a complex system or task. In addition to this models used to understand a process and effectively make decisions. Models are essential in that they are used to:-develop insights and understand the governing principles and fundamental mechanisms of a system, determine the ‘what if?’ questions and predict outcome of a system, assess impacts of a system. But it may be garbage in garbage out so it should be calibrate and validate with the standard and actual or observed data.

The most problem part of calibrating a model is making judgments concerning the adjustments that must be made to the model to bring it into agreement with field results. This section introduces methods for making these calibration judgments.

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. 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 calibration analysis should always be performed before a model is used for decision-making purposes.

The following is a seven-step approach that can be used as a guide to model calibration (Ormsbee and Lingireddy, 1997).

1. Identify the intended use of the model.
2. Determine the estimated of model parameters.
3. Correct each and everything calibration analysis
4. Evaluate model results based on initial estimates of model parameters.
5. Perform a rough calibration analysis
6. Perform a sensitivity analysis
7. Collect calibration data

5.7.7. Sampling Location

Data collection plays an important role in handling water distribution systems. The core objective of the field data gathering planning exercise is to determine what, when, under what conditions, and where to observe the characteristics of the system and collect data that, when used for calibration, will yield the best results. This is what calls as a sampling design problem is. The answers to what, when, and under what conditions are usually known and are described in the literature review, but the last question, where to locate measuring devices, has been the subject of numerous research studies. Walski (1983) suggests that pressure measuring devices should be located near points of high demand, near the perimeter of the skeletonized network, and generally distant from water sources

5.7.8Pressure Calibration

Pressures are measured throughout the water distribution system to monitor the level of service and to collect data for use in model calibration. Pressure readings are commonly taken at fire hydrants but can also be read at hose bibs (also called *spigots*); home faucets; pump stations (both suction and discharge sides); tanks; reservoirs; and blow-off, air release, and other types of valves.

If the measurements are taken at a location other than a direct connection to a water main (for example, at a house hose bib), the head loss between the supply main and the site where pressure is measured must be considered. Of course, the best solution is to have no flow (and hence no head loss) between the main and the gage for this thesis gaging was taken from different flow nodes

Table5.22Representative pressure simulated with observed

	Gauging	Measured time	Location				Pressure(mH_{20})	
S.N$_O$	Nod	Time	X	Y	Z	Simulated	Observed	difference
1	J-1	6:00				80.5	78.6	1.9
2	J.5	6:30	3,033.91	3,030.05	2430	72.5	68.4	4.1
3	J-22	7:00	2,980.86	3,000.38	2428	59.1	53	6.1
4	J-30	7:30	2,988.43	3,103.41	2425	58.9	58	0.9
5	J-40	8:00	3,072.34	3,078.65	2445	56.8	54.1	2.7
6	J-15	8:30	3,079.38	3,084.97	2435	66.7	63	3.7
7	J-10	9:00	3,053.84	3,037.80	2,437	65.3	62.5	2.8
8	J-17	9:30	3,013.09	3,055.32	2433	66.5	63	3.5
9	J-57	10:00	3,107.94	3,089.73	2,458	39.1	37	2.1
10	J-58	10:30	3,095.59	3,083.69	2,460	36.9	33	3.9
11	J-25	11:00	3,063.93	3,063.93	2440	53.4	49	4.4
12	J-45	11:30	3,031.85	3,084.61	2435	65.6	63	2.6
13	J-48	12:00	3,031.18	3,132.62	2452	48	46.6	1.4

$\frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})(y-\bar{y})^2}}$ = 0.9780437793 it indicated that the correlated betweensimulated with observed pressure is nearly 97% or there is 3% difference between the actual and simulated value

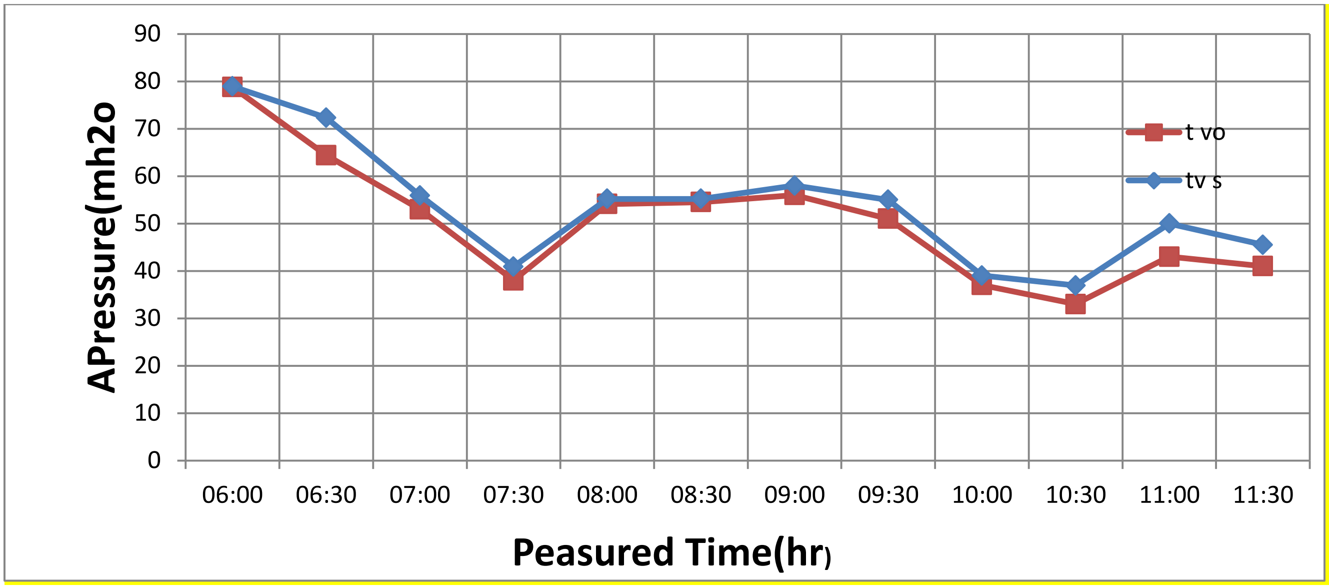


Figure5.9 Model calibration of by using observed and simulated data

The graph indicated that low water consumption and relatively high pressure will existing (5:00 to6:30) and high amount of water is consumed at(6:30 to7:30) morning and at the evening(4:00to6:300)

5.7.9Model validation

After a model is calibrated to a given set of test data, then correlated the model observed data it was around 98% can gain confidence in the model and/or identify its shortcomings by validating it with test data obtained under different conditions. In performing validation, system demands, initial conditions, and operational rules are adjusted to match the conditions at the time the test data were collected. For example, a model that was calibrated for a peak day may be validated by its capability to accurately predict average day conditions as well.

Consequently, perform a quick validation before applying the model to a new problem by changing the Haze wilams coefficient parameters and correlated the calibrated data and minimize the gap between observed and measured pressure data and the correlated was around 99.6%

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.

Table5.23Validated Representative Pressure of simulated with observed

	Gauging	Measured time	Location				Pressure(mH_{20})	
S.No	Nod	Time	X	Y	Z	Simulated	Observed	difference
1	J-1	6:00	3181.61	3009.49		78.9	78.6	0.3
2	J.5	6:30	3,033.91	3,030.05	2430	70.3	68.4	1.9
3	J-22	7:00	2,980.86	3,000.38	2422	55.9	53	2.9
4	J-30	7:30	2,988.43	3,103.41	2425	58.9	58	0.9
5	J-40	8:00	3,072.34	3,078.65	2445	55.2	54.1	1.1
6	J-15	8:30	3,079.38	3,084.97	2435	65	63	2
7	J-10	9:00	3,053.84	3,037.80	2,437	63.5	62.5	1
8	J-17	9:30	3,013.09	3,055.32	2433	55	51	4
9	J-57	10:00	3,107.94	3,089.73	2,458	37.6	37	0.6
10	J-58	10:30	3,095.59	3,083.69	2,460	33.6	33	0.6
11	J-25	11:00	3,063.93	3,063.93	2440	51	49	2
12	J-45	11:30	3,031.85	3,084.61	2435	64.6	63	1.6
13	J-48	12:00	3,031.18	3,132.62	2452	47	46.6	0.4

$$\sum \frac{(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})(y-\bar{y})^2}} = 0.9964647006$$

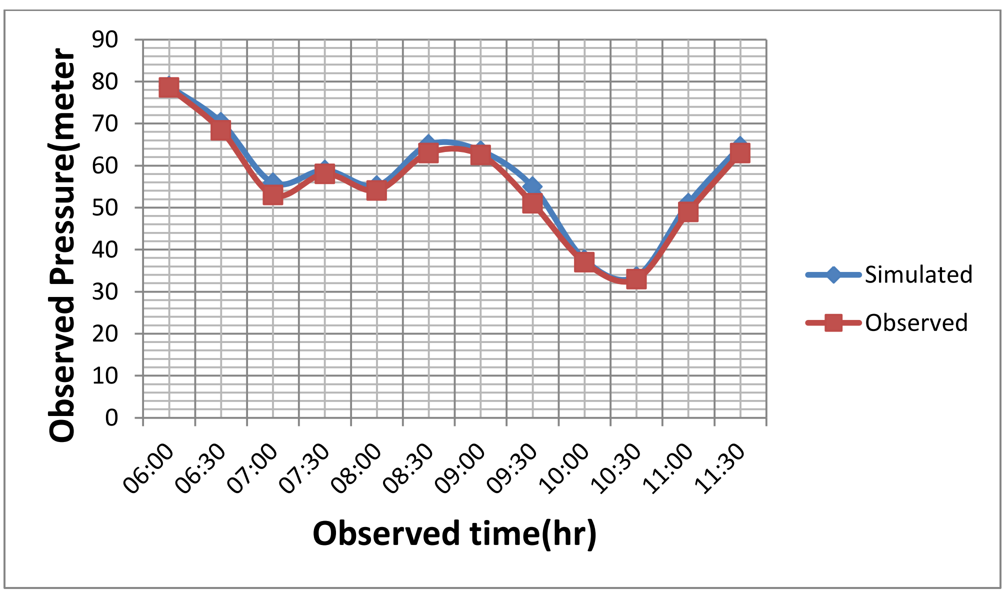


Figure5.10 Model calibration of by using observed and simulated data

This study has attempted to evaluate the situation with respect to the water supply coverage and water loss in Tilili town water supply distribution system system. The study has also proposed appropriate strategies and techniques for increasing water supply coverage and reduction and control of non-revenue water.

The average water supply coverage of the town was evaluated based on the daily per capital consumption and level of connection using the population data of the town.

The main reason for the decline of average consumption per connection is that the increase of connections was not supported with proportional increase in supply. When we look in to the average annual growth of connections between 2005 and 2010 it was 6% while production increased by 5% and billed amount by 4%. This shows that the growth of connection was not supported by sufficient supply. While the average consumption per domestic connection has declined the average non domestic consumption per connection has remained almost the same (average growth rate was -0.16%). The average non domestic consumption per connection for the same period was 59.14 m3/per month. This shows that in spite of their high share of consumption their average consumption did not decline even under difficult supply situations.

The water supply coverage of the town is very low compared with the minimum standard set by AAWSA – Business Plan Version 01 Draft Report 2011(29l/c/d) and UN-Habitat at 2000 as a basic need for drinking and sanitation alone (25l/per/day). Nearly 80% of the entire town population is getting water less than this basic service level. Although there is overall shortage of water in the town, predominantly the existing amount of water is fairly distributed among the different localities except few districts that consumed much water although their number of population is either low or moderate and vice versa.

Generally, in the old settlement areas of the town low level of connections per family and per capita water consumption was observed. Hence, it is concluded that the low financial capacity of the inhabitants, the topographic nature (higher elevation) of the areas and the available traditional pond as the main reasons for the low coverage in the areas.Three approaches were used to compare the loss among the systems:-

(v) The UFW expressed as a percentage.

(vi) Loss per-length of pipes and.

(vii) Loss per connection.

(viii) ILI

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, AAWSA and ADSB . Depending on the hierarchy of the network system, both the loss per kilometer length of main pipes (m3/km/day) and loss per connection (litres/connection/day) may be appropriate to measure the loss in the Tilili town.

Despite the low water supply coverage, the total water loss in the Tilili town is up to 37.2% of the total system input volume. It is found to be high; the total water loss was computed by subtracting the consumption (bill data) from the water production 91231m3/year (supplied)is 34009 m3/year at the town level. The approaches unaccounted for water expressed as a percentage and loss as per connection. The total water loss express as percentage is an important tool than water loss as per number of connection. In the town the loss of water as per number of connection, taking the total number of connection in the town as the water loss per connection for the similar duration was derived as; Water loss is 54.33 l/d liter/connection/day. The other issue addressed in the analysis was that of the impact of pipe age on water loss. On the one hand the loss was found to be higher in the sub-system where pipes of relatively older age are located. .

From the above explanations, the main causes of the water loss may be characterized as being caused by leakage and customer meter errors (under recording of meters).However, from the quality of the available data and the complex nature of the water loss and leakage, this conclusion could only be taken as suggestive findings to be a base for further studies. For better identification and characterization of the water losses and evaluation of its spatial distribution, a methodology is suggested in chapter six of this study

6 CONCLUSION AND RECOMENDATIION

6.1 Introduction

One of the major issues affecting water utilities in the developing world is increasing the demand of the community and considerable difference between the amount of water put into the distribution system and the amount of water billed by consumers called “Non-Revenue Water” (NRW). Current different research and statistical surveys indicated that NRW in developing countries is around 45 to 50% i.e. half of the total system input volume. Although, it is widely acknowledged that NRW levels in developing countries are very high, in fact, very few data are available in the literature regarding the actual figures. This is largely because most water utilities in the developing world do not have adequate monitoring systems for assessing water losses and many countries lack national reporting systems that collect and consolidate information on water utility performance.

Currently the water supply system in Tilili town does not meet the need of people and small industries. Tilili town also faced high levels of water losses those are indicative of poor governance and poor physical condition of the WDS. The amount of water loss in water distribution systems varies year to year in the town.

The main objective of the research was evaluate the water supply coverage and loses by assessing the distribution system in the town and to recommend mitigation measure for the above causes and to increasing the water supply coverage and decreasing NRW.

The major issue addressed in the analysis was the factors contributing to low coverage and high levels of water loss in town are:-

- ✓ The current water supply coverage is below the recommended value in addition to this the existing reservoir cannot feed above nine thousand population, but the current population number is around17 thousand so it indicated that design consideration problem for mode of service and small industries demand for parallel increasing of water demand and living standard of the community.
- ✓ The main reason for the decline of average consumption per connection is the increasing of connections was not supported with proportional increase in supply.
- ✓ Inappropriate system design and poor workmanship
- ✓ Problems of timely maintaining and replacing aged meters were one of the critical problem of increasing NRW because the concerned bodies are do not replacing aged water meter at the right time .

- ✓ Age of pipe network –All the water supply component was do not replaced before burst or leakage is exist
- ✓ Water meter and data handling error
- ✓ Customer Meter Servicing Problem
- ✓ Lack of real time field information.
- ✓ Inadequate training of personnel
- ✓ Inadequate emphasis on preventive maintenance
- ✓ Lack of operation manuals
- ✓ Poor maintenance of networks.
- ✓ Water scheduling
- ✓ Customer side leakage
- ✓ I legal connections-even if it was happed rarely it is not burning issue for the town

Generally due to the above problem what previously explained the town lost $5*34009m^3$=170045 Ethiopian birr (ETB) or 8502.25united state of American dollar (USD) revenue in a year this is by considering low price of water tariff ($5ETB/M^3$).

This paper has attempted to put forward the current situation of water coverage and loss in Tilili. Besides, it proposes appropriate solutions for the reduction and control of water loss. It is hoped that it will be a catalyst for increased and enhanced awareness and implementation of water loss solutions in the country. Therefore, there is a need for clear-cut sector policies, legal framework and a clear demarcation of responsibilities and mandates within the water supply sub-sector.

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

6.2. Recommendations

The study was focused the current water supply coverage and loss in Tilil town. By using different methodology application approach to distribution systems will result in better understanding and knowledge of the decrease of the average per-capita demand and components of uncontrolled flow rate, real losses and apparent losses. This paper has attempted to put forward the current situation of water supply coverage and loss in Tilili town. Besides, it proposes direction and strategy to arrive at appropriate solutions for the increasing water supply coverage and reduction and control of water loss in the distribution system.

- ✓ Based on MoWR guidelines put percentage of mode of service for rural town ,the current water supply coverage of Tilili town or the per-capita demand should be 32L/d and to increasing water supply coverage or average consumption per connection is that the increase of connections should be supported with proportional increase in supply.
- ✓ Reliable static and continuous data should be collected and mailed on priority basis which can be suitably used for performance evaluation purpose or all data should put in the form of soft copy rather hard copy .
- ✓ NRW in high levels typically indicate poor governance and a poorly managed water utility. For developing country, reducing NRW should be the first option to pursue while addressing increased demand for piped water supply. This can be achieved by conducting water audit.
- ✓ Attention must also be given for customer billing equity. Major differences are meter accuracy translates to unequal customer billing. A water system with a high unaccounted for water loss may have leaks or other distribution problems that must be addressed. Without accurate meters the degree of these problems cannot truly be known. The accuracy of a utilities water meters should be high priority and efforts made to maintain all meters at a high and uniform level of efficiency..
- ✓ It is observed that sensitivity of the water meter decreases with the age of meter increases and hence there should be appropriate policy for testing and replacement.
- ✓ An appropriate technical performance indicator for real losses shall be developed for developing countries so that one can understand the condition of infrastructure.
- ✓ In most of the intermittent supply system, operating pressure and supply hours are very poor. Due to scheduling of water, valves are operated frequently and therefore more leakages are concentrated at valves. This can be avoided by transforming intermittent to continuous water supply.
- ✓ Water loss is not only engineering problem but it can be reduced by creating awareness and changing behavior of people by educating them to be owner ness feeling.
- ✓ Operation and Maintenance of network is very essential to increase the life of network, therefore, operation and maintenance policy needs to be developed

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ANNEX

Annexes-A: Population Projection (20016-2037)

year			t	e	r	p_o	$P_t = P_o e^{rt}$
2016	4.10%	18251	1	2.7182	4.10%	18251	19015
2017	4.10%	19811	2	2.7182	4.10%	18251	19811
2018	4.10%	20640	3	2.7182	4.10%	18251	20640
2019	4.00%	21418	4	2.7182	4.00%	18251	21418
2020	4.00%	22292	5	2.7182	4.00%	18251	22292
2021	4.00%	23201	6	2.7182	4.00%	18251	23201
2022	4.00%	24148	7	2.7182	4.00%	18251	24148
2023	4.00%	25134	8	2.7182	4.00%	18251	25134
2024	3.80%	25693	9	2.7182	3.80%	18251	25693
2025	3.80%	26688	10	2.7182	3.80%	18251	26688
2026	3.80%	27721	11	2.7182	3.80%	18251	27721
2027	3.80%	28795	12	2.7182	3.80%	18251	28795
2028	3.80%	29910	13	2.7182	3.80%	18251	29910
2029	3.80%	31069	14	2.7182	3.80%	18251	31069
2030	3.80%	32272	15	2.7182	3.80%	18251	32272
2031	3.80%	33522	16	2.7182	3.80%	18251	33522
2032	3.70%	34233	17	2.7182	3.70%	18251	34233
2033	3.70%	35524	18	2.7182	3.70%	18251	35524
2034	3.70%	36863	19	2.7182	3.70%	18251	36863
2035	3.70%	38252	20	2.7182	3.70%	18251	38252
2036	3.70%	39694	21	2.7182	3.70%	18251	39694
2037	3.70%	41190	22	2.7182	3.70%	18251	41190

Annex-B Domestic demand projections for the corresponding years for phase-one.

Year	Unit	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026	2027
1. Population		18251	19811	20640	21418	22292	23201	24148	25134	25693	26688	27721	28795
Population growth rate (town)		4.10%	4.10%	4.10%	4.10%	4.10%	4.00%	4.00%	4.00%	4.00%	4.00%	3.80%	3.80%
domestic demand													
Coverage by service type													
HTU		6.80%	7.50%	7.66%	7.82%	7.98%	8.14%	8.30%	8.38%	8.46%	8.54%	8.62%	8.70%
YTU		21.60%	23.90%	24.38%	24.86%	25.34%	25.82%	26.30%	26.68%	27.06%	27.44%	27.82%	28.20%
NTU		20.90%	23.10%	23.48%	23.86%	24.24%	24.62%	25.00%	25.00%	25.00%	25.00%	25.00%	25.00%
PTU(urban)		44.60%	42.40%	41.96%	41.52%	41.08%	40.64%	40.20%	39.78%	39.36%	38.94%	38.52%	38.10%
Coverage total (town)		93.90%	96.90%	97.50%	98.10%	98.60%	99.20%	99.80%	99.80%	99.90%	99.90%	100.00%	100.00%
Total population to be served		17138	19197	20120	21002	21989	23020	24100	25094	25662	26667	27710	28795
Population served by													
HTU		1241	1486	1581	1675	1779	1889	2004	2106	2174	2279	2390	2505
YTU		3942	4735	5032	5324	5649	5991	6351	6706	6952	7323	7712	8120
NTU		3814	4576	4846	5110	5404	5712	6037	6283	6423	6672	6930	7199
PTU(urban)		8140	8400	8660	8893	9157	9429	9708	9998	10113	10392	10678	10971

Persons per PF		250	250	250	250	250	250	250	250	250	250	250	250
Person per connection		5	5	5	5	5	5	5	5	5	5	5	5
No Connections													
HTU		248	297	316	335	356	378	401	421	435	456	478	501
YTU		788	947	1006	1065	1130	1198	1270	1341	1390	1465	1542	1624
NTU		763	915	969	1022	1081	1142	1207	1257	1285	1334	1386	1440
PTU(urban)		33	34	35	36	37	38	39	40	40	42	43	44
Total Population Served		17138	19197	20120	21002	21989	23020	24100	25094	25662	26667	27710	28795
2. Demand													
2.1 Domestic													
Per capita demand													
HTU	l/c/d	46.8	47	46.6	49.2	50	51	51.4	53	54	55	56.2	57.4
YTU	l/c/d	42.8	43	42.6	45.2	46	47	48	48.6	55.8	51	52.2	53.4
NTU	l/c/d	39.8	40.6	40	42.2	43	43	44.6	45	46.2	47	48	49
PTU (town)	l/c/d	23.4	23.2	23.8	24.6	25	25.6	26.2	26.8	27.4	28	28.6	29.2
Total Consumption (daily)By connection													
	l/d	58082	69834	73675	82403	88944	96318	103021	111629	117375	125353	134295	143797
	l/d	168727	203598	214361	240664	259841	281558	304847	325896	387947	373480	402572	433620

	l/d	151815	185799	193848	215653	232351	245624	269253	282754	296751	313582	332658	352740
	l/d	190475	194877	206118	218758	228936	241384	254339	267952	277087	290983	305400	320352
Consumption (daily)													
HTU	m3/d	58	70	74	82	89	96	103	112	117	125	134	144
YTU	m3/d	169	204	214	241	260	282	305	326	388	373	403	434
NTU	m3/d	152	186	194	216	232	246	269	283	297	314	333	353
PTU (town)	m3/d	190	195	206	219	229	241	254	268	277	291	305	320
Total domestic demand(TDD)	m3/d	569	654	688	757	810	865	931	988	1079	1103	1175	1251

Annex-C: Summery projected water demand

	Unit	2016	2017	2018	2019	2020	2021	2022	2023	2024	2025	2026
1. Population		18251	19811	20640	21418	22292	23201	24148	25134	25693	26688	27721
Population growth rate (town)		4.10%	4.10%	4.10%	4.10%	4.10%	4.00%	4.00%	4.00%	4.00%	4.00%	3.80%
Coverage (town)	%	93.90 %	96.90 %	97.48%	98.06%	98.64%	99.22%	99.80%	99.84%	99.88%	99.92 %	100%
TDD	m3/d	569	654	688	757	810	865	931	988	1079	1103	1175
C&PD	m3/d	134	142	147	152	158	163	168	174	178	185	190
Industrial Demand	m3/d	20.8	21.85	22.9	23.95	25	25.65	26.3	26.95	27.6	28.25	28.8
Sub-Total		724	818	858	934	993	1053	1126	1189	1285	1317	1394
Unaccounted for water(%) of avgdd	%	34%	34%	34%	34%	34%	34%	34%	34%	34%	34%	20%
UDD (m3/d)	m3/d	246.3	278.2	291.8	317.5	337.5	358.1	382.8	404.2	436.8	447.7	278.8
A D D	m3/d	971	1096	1150	1251	1330	1412	1509	1593	1721	1765	1673

	l/sec	11.2	12.7	13.3	14.5	15.4	16.3	17.5	18.4	19.9	20.4	19.4
Ave.PD	l/c/day	53	55	56	58	60	61	62	63	67	66	60
M DF		1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
M. D D	m3/d	1261.8	1425.1	1495.06	1626.796	1729.06	1834.966	1961.2705	2071.1735	2237.913	2293.9	2174.753
	l/sec	14.6	16.5	17.3	18.8	20.0	21.2	22.7	24.0	25.9	26.6	25.2
PHF		1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9	1.9
PHD	m3/hr	76.8	86.8	91.0	99.1	105.3	111.7	119.4	126.1	136.3	139.7	132.4
	l/sec	21.3	24.1	25.3	27.5	29.2	31.0	33.2	35.0	37.9	38.8	36.8

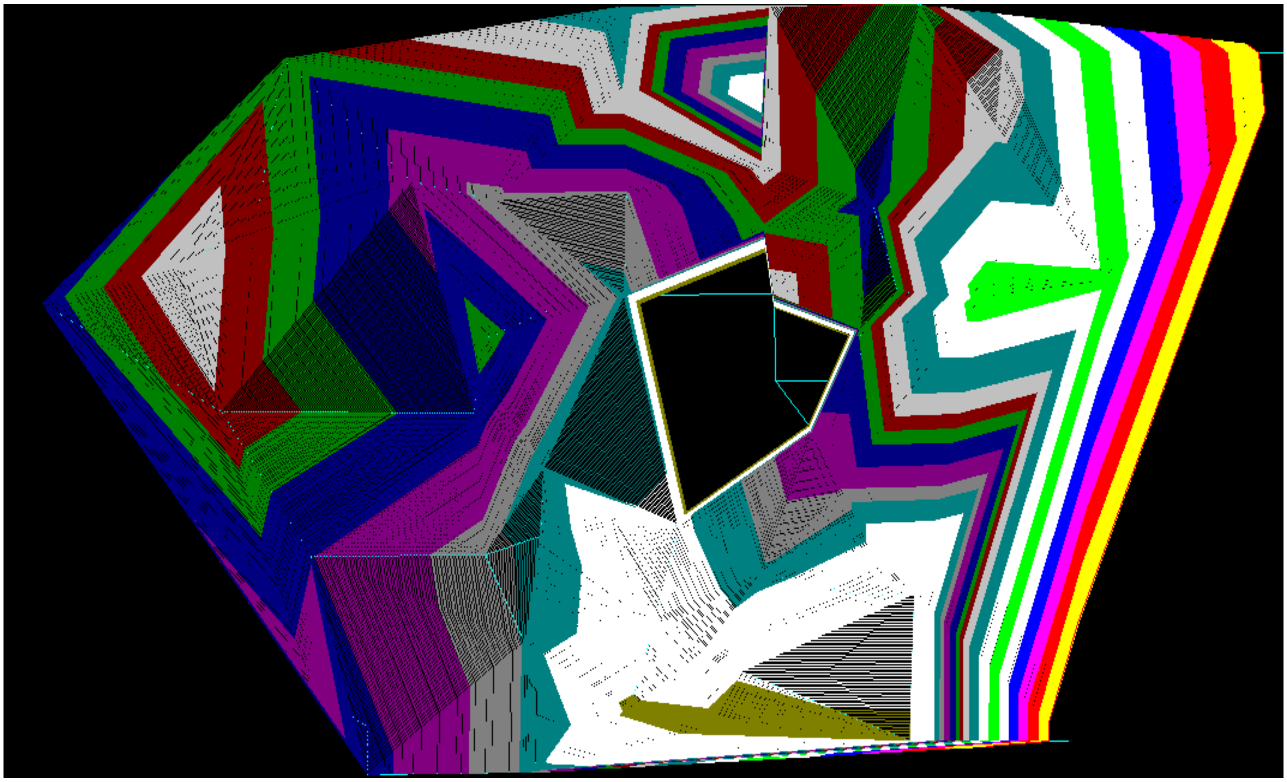
ANNEX-D:Reservoir Sizing For Phase–I at2027

Time	Hourly Factor	MDD(L)	HourlyDemand	HourlySupply	CummDemand	Cummusply	exces demand	Excesuply
1	0.3	2298240	28728	0	28728	0	28728	
2	0.3	2298240	28728	0	57456	0	57456	
3	0.3	2298240	28728	114912	86184	114912		28728
4	0.3	2298240	28728	114912	114912	229824		114912
5	0.3	2298240	28728	114912	143640	344736		201096
6	0.7	2298240	67032	114912	210672	459648		248976
7	1.1	2298240	105336	114912	316008	574560		258552
8	1.9	2298240	181944	114912	497952	689472		191520
9	1.7	2298240	162792	114912	660744	804384		143640
10	1.5	2298240	143640	114912	804384	919296		114912
11	1.5	2298240	143640	114912	948024	1034208		86184
12	1.4	2298240	134064	114912	1082088	1149120		67032
13	1.4	2298240	134064	0	1216152	1149120	67032	
14	1.3	2298240	124488	0	1340640	1149120	191520	
15	1.2	2298240	114912	114912	1455552	1264032	191520	
16	1.5	2298240	143640	114912	1599192	1378944	220248	
17	1.6	2298240	153216	114912	1752408	1493856	258552	
18	1.5	2298240	143640	114912	1896048	1608768	287280	
19	1.3	2298240	124488	114912	2020536	1723680	296856	
20	1	2298240	95760	114912	2116296	1838592	277704	
21	0.7	2298240	67032	114912	2183328	1953504	229824	
22	0.5	2298240	47880	114912	2231208	2068416	162792	
23	0.4	2298240	38304	114912	2269512	2183328	86184	
24	0.3	2298240	28728	114912	2298240	2298240	0	0
MAX(							29685	258552

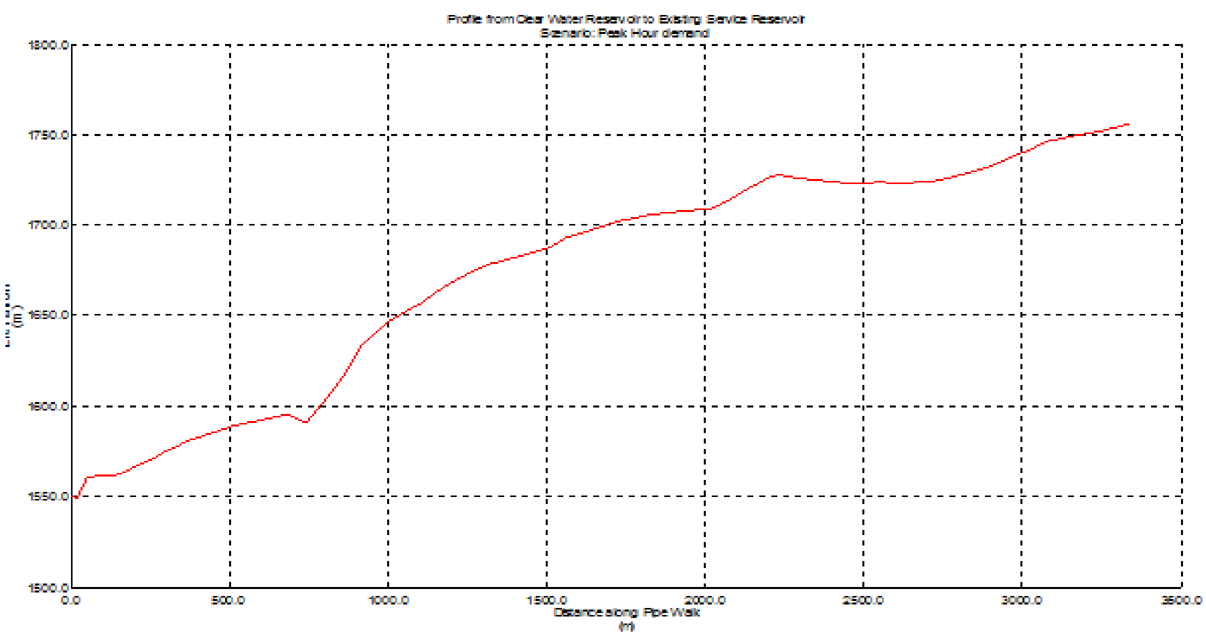
l/r)h							6	

ANNEX-E: Reservoir Sizing For Phase at 2037

Time	Hourly Factor	MDD(L)	Hourly Demand(L)	Hourly supply(l	Comm demand	CommuS upply	Execs Demand	execs supply
1	0.3	3948480	49356	0	49356	0	49356	
2	0.3	3948480	49356	0	98712	0	98712	
3	0.3	3948480	49356	197424	148068	197424		49356
4	0.3	3948480	49356	197424	197424	394848		197424
5	0.3	3948480	49356	197424	246780	592272		345492
6	0.7	3948480	115164	197424	361944	789696		427752
7	1.1	3948480	180972	197424	542916	987120		444204
8	1.9	3948480	312588	197424	855504	1184544		329040
9	1.7	3948480	279684	197424	1135188	1381968		246780
10	1.5	3948480	246780	197424	1381968	1579392		197424
11	1.5	3948480	246780	197424	1628748	1776816		148068
12	1.4	3948480	230328	197424	1859076	1974240		115164
13	1.4	3948480	230328	197424	2089404	2171664		82260
14	1.3	3948480	213876	197424	2303280	2369088		65808
15	1.2	3948480	197424	197424	2500704	2566512		65808
16	1.5	3948480	246780	197424	2747484	2763936		16452
17	1.6	3948480	263232	197424	3010716	2961360	49356	
18	1.5	3948480	246780	197424	3257496	3158784	98712	
19	1.3	3948480	213876	197424	3471372	3356208	115164	
20	1	3948480	164520	197424	3635892	3553632	82260	
21	0.7	3948480	115164	197424	3751056	3751056		
22	0.5	3948480	82260	197424	3833316	3948480		115164
23	0.4	3948480	65808	0	3899124	3948480		49356
24	0.3	3948480	49356	0	3948480	3948480	0	0
Max							115164	444204



ANNEX -F: Graph showing pressure contour maps at different consumption hour



ANNEX-G: Graph showing profile of elevation from main line pipes to the reservoir

As shown in above figure 4.133 shows that, as the distance increase the water loss also increase.

ANNEX-H: Pipe Report

Pipe	From Node	To Node	Length (m)	Diameter (mm)	Material	Hazen-Williams C	Headloss (m)
P-1	Reservoir	J-1	1,795.00	200	PVC	150	6.32
P-2	J-1	J-2	138	150	PVC	150	0.05
P-3	J-2	J-3	425	150	HDPE	70	0.51
P-4	J-3	J-4	221	150	HDPE	70	0.23
P-5	J-4	J-5	393.5	150	HDPE	70	0.3
P-8	J-5	J-8	456.5	110	HDPE	70	0.73
P-10	J-9	J-10	366	200	PVC	150	0.91
P-11	J-10	J-8	161.5	150	PVC	150	0.6
P-12	J-10	J-12	189.5	150	PVC	150	0.3
P-13	J-12	J-13	115.5	150	PVC	150	0.17
P-14	J-13	J-59	83	150	PVC	150	0.11
P-15	J-59	J-14	34	150	PVC	150	0.03
P-19	J-40	J-15	116.5	110	HDPE	70	0
P-20	J-15	J-44	203.5	110	HDPE	70	0.21
P-21	J-42	J-43	174	110	HDPE	70	0.31
P-22	J-43	J-44	179	50	HDPE	70	0.65
P-23	J-44	J-53	471	110	HDPE	70	1.29
P-24	J-53	J -54	220	110	HDPE	70	1.68
P-26	J-15	J-58	441.5	50	HDPE	70	4.87
P-27	J-58	J-57	602	50	HDPE	70	0.22
P-28	J-57	J-53	335	50	HDPE	70	6.15
P-29	J-43	J-51	131	110	HDPE	70	0.25
P-30	J-51	J-50	65	110	HDPE	70	0.03
P-31	J-50	J-49	221.5	50	HDPE	70	0.37
P-32	J-49	J-55	45	50	HDPE	70	0.3
P-33	J-55	J -54	237	50	HDPE	70	3.96
P-34	J-8	J-36	61.5	150	PVC	150	0.25
P-35	J-36	J-17	511	150	PVC	150	1.96
P-36	J-17	J-34	45.5	110	HDPE	70	1.01
P-37	J-34	J-18	273.5	110	HDPE	70	5.06
P-39	J-20	J-21	317.5	110	HDPE	70	0.53
P-40	J-21	J-22	363.5	50	HDPE	70	4.35
P-43	J-24	J-25	470.5	80	HDPE	70	0.41
P-44	J-25	J-33	311.5	110	HDPE	70	0.13
P-45	J-33	J-32	405.5	50	HDPE	70	2.16
P-46	J-32	J- 31	168.5	50	HDPE	70	0
P-47	J- 31	J-30	409	50	HDPE	70	2.93
P-48	J-17	J-26	370	110	HDPE	70	4.88

P-49	J-26	J-25	405	110	HDPE	70	1.32
P-50	J-30	J-29	171.5	110	HDPE	70	0.13
P-51	J-26	J-27	31.5	110	HDPE	70	0.08
P-52	J-27	J-29	446.5	110	HDPE	70	0.47
P-55	J-14	J-42	211	150	PVC	150	0.13
P-56	J-49	J-48	259.5	50	HDPE	70	0.66
P-57	J-48	J-47	221.5	110	HDPE	70	0.04
P-58	J-47	J-46	228	110	HDPE	70	0.21
P-59	J-46	J-45	147	110	HDPE	70	0.32
P-60	J-45	J-42	267.5	110	HDPE	70	1.06
P-61	J-18	J-19	42	110	HDPE	70	0.31
P-62	J-19	J-20	277	110	HDPE	70	1.24
P-63	J-18	J-24	298	110	HDPE	70	0.55
P-64	J-22	J-23	352	50	HDPE	70	4.62
P-65	J-23	J-24	212	80	HDPE	70	1.26
P-6	J-5	J-6	49.5	110	HDPE	70	0.01
P-66	R-2	J -54	200	80	Ductile Iron	130	4.86
P-18	J-14	J-40	226	110	Ductile Iron	130	0.01
P-9	J-1	J-9	88	200	Ductile Iron	130	0.3

ANNEX-I: Junction Report

	Elevation (m)	Base Flow (l/s)	Demand (Calculated) (l/s)	Calculated Hydraulic Grade (m)	Pressure (m H2O)	X (m)	Y (m)
J-1	2,423.00	0.42	0.26	2,503.68	80.522	3,081.61	3,009.49
J-2	2,424.00	0.57	0.36	2,503.64	79.476	3,049.94	3,021.11
J-3	2,420.00	0.45	0.28	2,503.12	72.956	3,044.24	3,011.83
J-4	2,421.00	0.81	0.51	2,502.90	69.73	3,028.95	3,014.68
J-5	2,430.00	0.49	0.31	2,502.60	67.452	3,033.91	3,030.05
J-6	2,431.00	0.99	0.62	2,502.59	68.443	3,042.91	3,024.51
J-8	2,433.00	1.66	1.04	2,501.87	68.732	3,038.32	3,045.69
J-9	2,425.00	1.22	0.76	2,503.38	66.224	3,082.54	3,034.19
J-10	2,437.00	1.74	1.09	2,502.47	58.34	3,053.84	3,037.80
J-12	2,434.00	0.86	0.54	2,502.17	68.03	3,073.42	3,043.01
J-13	2,439.00	0.83	0.52	2,502.00	62.875	3,068.65	3,052.08
J-14	2,242.00	2.03	1.27	2,501.86	59.339	3,056.43	3,069.77
J-15	2,445.00	0.94	0.59	2,501.86	56.745	3,079.38	3,084.97

J-17	2,433.00	1.47	0.92	2,499.66	55.529	3,013.09	3,055.32
J-18	2,429.00	1.07	0.67	2,493.60	64.472	3,002.07	3,040.70
J-19	2,428.00	1.6	1	2,493.29	65.157	3,009.51	3,026.04
J-20	2,425.00	2.09	1.31	2,492.04	66.91	3,017.19	3,011.90
J-21	2,426.00	1.9	1.19	2,491.51	65.38	3,010.09	3,004.22
J-22	2,428.00	2.22	1.39	2,487.16	59.046	2,980.86	3,000.38
J-23	2,430.00	1.42	0.89	2,491.79	61.661	2,969.71	3,017.43
J-24	2,433.00	1.5	0.94	2,493.05	59.928	2,969.66	3,041.02
J-25	2,440.00	1.95	1.22	2,493.46	50.355	2,956.81	3,063.93
J-26	2,436.00	1.2	0.75	2,494.78	58.66	3,000.13	3,064.77
J-27	2,439.00	1.3	0.81	2,494.70	55.59	3,008.17	3,073.67
J-29	2,430.00	0.4	0.25	2,494.23	64.102	3,007.56	3,099.16
J-30	2,433.00	1.1	0.69	2,494.10	60.981	2,988.43	3,103.41
J- 31	2,435.00	0.82	0.51	2,491.17	56.06	2,964.91	3,122.82
J-32	2,440.00	0.7	0.44	2,491.17	51.07	2,944.86	3,088.91
J-33	2,435.00	0.7	0.44	2,493.33	58.216	2,919.17	3,082.92
J-34	2,433.00	1.1	0.69	2,498.66	65.525	3,012.34	3,043.77
J-36	2,430.00	0.9	0.56	2,501.62	60.474	3,026.17	3,046.92
J-40	2,445.00	1	0.63	2,501.86	46.741	3,072.34	3,078.65
J-42	2,450.00	0.4	0.25	2,501.74	51.633	3,056.31	3,084.50
J-43	2,446.00	0.5	0.31	2,501.42	55.312	3,054.49	3,095.43
J-44	2,444.00	1	0.63	2,502.07	57.956	3,074.74	3,099.72
J-45	2,435.00	1.3	0.81	2,500.68	45.549	3,031.85	3,084.61
J-46	2,437.00	1.3	0.81	2,500.36	63.234	3,031.92	3,101.46
J-47	2,450.00	1.3	0.81	2,500.16	50.055	3,031.83	3,117.47
J-48	2,452.00	1.3	0.81	2,500.12	48.023	3,031.18	3,132.62
J-49	2,448.00	0.7	0.44	2,500.78	52.671	3,055.67	3,132.54
J-50	2,427.00	1.2	0.75	2,501.14	55.994	3,054.06	3,117.63
J-51	2,449.00	1.6	1	2,501.18	52.071	3,053.96	3,106.07
J-53	2,452.00	1.5	0.94	2,503.36	51.26	3,097.85	3,112.36
J -54	2,452.00	2.1	1.31	2,505.04	52.933	3,094.85	3,126.92
J-55	2,445.00	0.5	0.31	2,501.08	55.969	3,083.43	3,132.98
J-57	2,458.00	1.2	0.75	2,497.21	39.131	3,107.94	3,089.73
J-58	2,460.00	1.2	0.75	2,496.99	36.916	3,095.59	3,083.69
J-59	2,441.00	1.5	0.94	2,501.90	60.774	3,063.55	3,061.33

ANNEX-J Calculated and Base Demand Result

	Elevation (m)	Base Flow (l/s)	Demand (Calculated) (l/s)	Calculated Hydraulic Grade (m)	Pressure (m H2O)	X (m)	Y (m)
J- 31	2,435.00	0.82	0.51	2,491.17	56.06	2,964.91	3,122.82
J-1	2,423.00	0.42	0.26	2,503.68	80.522	3,081.61	3,009.49
J-2	2,424.00	0.57	0.36	2,503.64	79.476	3,049.94	3,021.11
J-3	2,420.00	0.45	0.28	2,503.12	82.956	3,044.24	3,011.83
J-4	2,421.00	0.81	0.51	2,502.90	81.73	3,028.95	3,014.68
J-5	2,430.00	0.49	0.31	2,502.60	72.452	3,033.91	3,030.05
J-6	2,431.00	0.99	0.62	2,502.59	71.443	3,042.91	3,024.51
J-8	2,433.00	1.66	1.04	2,501.87	68.732	3,038.32	3,045.69
J-9	2,425.00	1.22	0.76	2,503.38	78.224	3,082.54	3,034.19
J-10	2,437.00	1.74	1.09	2,502.47	65.34	3,053.84	3,037.80
J-12	2,434.00	0.86	0.54	2,502.17	68.03	3,073.42	3,043.01
J-13	2,439.00	0.83	0.52	2,502.00	62.875	3,068.65	3,052.08
J-14	2,242.00	2.03	1.27	2,501.86	259.339	3,056.43	3,069.77
J-15	2,445.00	0.94	0.59	2,501.86	56.745	3,079.38	3,084.97
J-17	2,433.00	1.47	0.92	2,499.66	66.529	3,013.09	3,055.32
J-18	2,429.00	1.07	0.67	2,493.60	64.472	3,002.07	3,040.70
J-19	2,428.00	1.6	1	2,493.29	65.157	3,009.51	3,026.04
J-20	2,425.00	2.09	1.31	2,492.04	66.91	3,017.19	3,011.90
J-21	2,426.00	1.9	1.19	2,491.51	65.38	3,010.09	3,004.22
J-22	2,428.00	2.22	1.39	2,487.16	59.046	2,980.86	3,000.38
J-23	2,430.00	1.42	0.89	2,491.79	61.661	2,969.71	3,017.43
J-24	2,433.00	1.5	0.94	2,493.05	59.928	2,969.66	3,041.02
J-25	2,440.00	1.95	1.22	2,493.46	53.355	2,956.81	3,063.93
J-26	2,436.00	1.2	0.75	2,494.78	58.66	3,000.13	3,064.77
J-27	2,439.00	1.3	0.81	2,494.70	55.59	3,008.17	3,073.67
J-29	2,430.00	0.4	0.25	2,494.23	64.102	3,007.56	3,099.16
J-30	2,433.00	1.1	0.69	2,494.10	60.981	2,988.43	3,103.41
J-32	2,440.00	0.7	0.44	2,491.17	51.07	2,944.86	3,088.91
J-33	2,435.00	0.7	0.44	2,493.33	58.216	2,919.17	3,082.92
J-34	2,433.00	1.1	0.69	2,498.66	65.525	3,012.34	3,043.77
J-36	2,430.00	0.9	0.56	2,501.62	71.474	3,026.17	3,046.92
J-40	2,445.00	1	0.63	2,501.86	56.741	3,072.34	3,078.65
J-42	2,450.00	0.4	0.25	2,501.74	51.633	3,056.31	3,084.50
J-43	2,446.00	0.5	0.31	2,501.42	55.312	3,054.49	3,095.43
J-44	2,444.00	1	0.63	2,502.07	57.956	3,074.74	3,099.72
J-45	2,435.00	1.3	0.81	2,500.68	65.549	3,031.85	3,084.61
J-46	2,437.00	1.3	0.81	2,500.36	63.234	3,031.92	3,101.46

J-47	2,450.00	1.3	0.81	2,500.16	50.055	3,031.83	3,117.47
J-48	2,452.00	1.3	0.81	2,500.12	48.023	3,031.18	3,132.62
J-49	2,448.00	0.7	0.44	2,500.78	52.671	3,055.67	3,132.54
J-50	2,427.00	1.2	0.75	2,501.14	73.994	3,054.06	3,117.63
J-51	2,449.00	1.6	1	2,501.18	52.071	3,053.96	3,106.07
J-53	2,452.00	1.5	0.94	2,503.36	51.26	3,097.85	3,112.36
J-55	2,445.00	0.5	0.31	2,501.08	55.969	3,083.43	3,132.98
J-57	2,458.00	1.2	0.75	2,497.21	39.131	3,107.94	3,089.73
J-58	2,460.00	1.2	0.75	2,496.99	36.916	3,095.59	3,083.69
J-59	2,441.00	1.5	0.94	2,501.90	60.774	3,063.55	3,061.33
J -54	2,452.00	2.1	1.31	2,505.04	52.933	3,094.85	3,126.92